Bio-Engineering for Land Stabilization

Thesis

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By

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The goal of life is living in agreement with nature

– Zeno of Elea, Greek philosopher, 490-430 B.C.

ABSTRACT

As part of the Ohio Department of Transportation's (ODOT's) ongoing effort to solve engineering problems for the Ohio transportation system through research, The Ohio State University has undertaken a Bio-Engineering for Land Stabilization study under the direction of Professor Patrick J. Fox and Professor Emeritus T. H. Wu. Bioengineering is the use of vegetation for slope stabilization and has been used with success throughout the world; however, not much work on this topic has been performed in the mid-western United States.

The aim of this study is to identify bioengineering methods to address ODOT's land stabilization needs in response to the all too common occurrence of shallow landslides. Bioengineering methods offer environmentally and economically attractive alternatives to traditional approaches to remediate and prevent such landslides. This research plans to achieve several objectives through the construction of three field demonstration projects: (1) to identify important factors that control success or failure of bioengineering methods, (2) to develop installation techniques and designs for successful application of bioengineering methods, (3) to provide thorough documentation to guide future work in bioengineering for ODOT, and (4) to develop new monitoring and testing methods that may be required for bioengineering projects.

To date, research demonstration sites have been selected in Muskingum, Logan, and Union Counties and design and construction efforts are underway. Initial results of the

project indicate that bioengineering installations, such as live willow poles, can be effective for the stabilization of shallow slides if the vegetation can be established.

To my family and friends

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FIELDS OF STUDY

Major Fields: Civil Engineering

Soil Mechanics

Geotechnical Engineering

Minor Field: Economics

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CHAPTER 1

INTRODUCTION

Bioengineering for land stabilization is the use of vegetation, alone or in combination with mechanical elements, to achieve engineering designs that will arrest erosion and/or provide slope stabilization. More specifically, the use of vegetation in combination with mechanical elements is referred to as biotechnical stabilization whereas the use of vegetation alone has been termed soil bioengineering (Gray and Sotir 1996). Both of these approaches provide cost effective and environmentally attractive alternatives to the more traditional, monolithic means of stabilizing earth slopes and preventing erosion (e.g., retaining walls and revetments). Bioengineering techniques are limited, in general, to shallow mass movements and are inappropriate for controlling deep-seated slope failures due the limited depth of plant roots.

Fundamentally, slope stabilization is achieved through soil bioengineering and biotechnical stabilization by increasing the shear strength along a potential failure surface. Vegetation can increase shear strength by mechanically interlocking the soil mass with plant roots and, also, through dewatering of the soil.

Bioengineering techniques have been employed throughout the world and, when properly implemented, have achieved good success. To date, a considerable amount of data exist which describe plant selection and design procedures for bioengineering schemes, with the European and Asian continents at the forefront of research and development. Some work has been done in the United States in recent decades to promote the use and acceptance of bioengineering methods. State DOTs, the USDA, academic researchers, consultants, and the US Forest Service have been some of the biggest proponents. The successes of these technologies have been marked by projects that have proved to be both economically and environmentally sustainable (Gray and Leiser 1982; Sotir 1995; Gray and Sotir 1996; Sotir and Christopher 2004).

We live in a changing engineering world where "green" and sustainable designs are becoming commonplace with greater awareness of mankind's influence on the environment. The motivation for this research is to obtain the necessary information which will permit the rational design of bioengineering technologies as a cost effective and environmentally attractive alternative to the traditional means for slope stabilization. Direct emphasis has been placed on the slope stabilization needs of the project's sponsor, the Ohio Department of Transportation (ODOT); specifically, the proper vegetation and methods to stabilize earth slopes throughout the state's particular geologic makeup and climate. The objectives of this research are: (1) to identify important factors that control success or failure of bioengineering methods, (2) to develop installation techniques and designs for successful application of bioengineering methods, (3) to provide thorough documentation to develop design guides for future work in bioengineering for ODOT, and (4) to develop new monitoring and testing methods that may be required for bioengineering projects.

Three field case studies will be evaluated through the course of the research effort and will provide insight to the appropriate plant selection and implementation of bioengineering technologies. One cut slope and two embankment slopes which have experienced shallow landsliding comprise the three case study field demonstration sites for this project. The first site is a cut slope drainage swale located at the infield of the interchange from U.S. Route 33 (US-33) to Ohio State Route 347 (SR-347) approximately fifteen miles west-northwest of Marysville, Ohio, in Logan County. The second demonstration site is located eighty miles east of Columbus, Ohio, near the village of New Concord in Muskingum County. An embankment supporting the onramp from Ohio State Route 83 (SR-83) to west bound Interstate 70 (I-70) is the location of the second demonstration site. An overpass embankment along U.S. Route 33 (US-33) just outside of Marysville, Ohio, in Union County, is the location of the third field demonstration site. These three sites have been selected from over forty landslide sites in Ohio that were visited and evaluated during the early stages of this project.

Each of the three demonstration sites are being monitored with extensive instrumentation which include tensiometers, inclinometers, gypsum moisture blocks, and piezometers. Additionally, subsurface investigations have been conducted at all three sites which have produced standard penetration soundings and relatively undisturbed soil samples. Laboratory testing, including triaxial shear, consolidation, classification, and soil nutrient levels, have been performed on the recovered soil samples. Through the laboratory and field monitoring efforts, the design, environmental, and performance parameters considered include soil nutrients, soil moisture, pore pressure/matric suction, soil strength, slope movements, and vegetation survivability.

The information in this thesis is current up to spring 2006. To date (spring 2006), each one of the field demonstration sites is at a different stage of the remediation process. Designs for all three sites were prepared and finalized. Vegetation harvest sources which are needed for the site construction have been secured from various locations. At the Muskingum County site, a bioengineering design was implemented where live poles and brushlayers were installed during spring 2005 to arrest shallow mass movements. Combinations of live willow poles, brushlayers, slope grading, and geosynthetics were constructed at the Logan County site during spring 2007. Live willow pole installation was also completed during spring 2007 at the Union County site.

This thesis is divided into 6 chapters and presents the efforts undertaken up to spring 2006 for the ODOT funded research project, Bio-Engineering for Land Stabilization. This chapter, Chapter 1, introduces the research project, the focus, and the research objectives. Chapter 2 contains a comprehensive literature review of bioengineering and provides a synthesis that is directly applicable to this project. The general site selection and project design considerations which are not specific to the individual demonstration sites are described in Chapter 3. The two demonstration sites in Logan and Muskingum Counties are reported in Chapters 4 and 5. Finally, Chapter 6 presents conclusions for the research effort up to spring 2006, as well as future work and recommendations for this research project.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Bioengineering for slope stability is the use of vegetation by itself or in conjunction with inert construction materials to provide resistance against slope instability. In such constructions the survival, selection, and implementation of vegetation are paramount to the success of an engineered project. Therefore, the engineer who desires successful employment of a bioengineered design requires a fundamental understanding of the growth requirements of commonly used plant species. The interdisciplinary nature of such projects requires attention on items which will be necessary for successful plant propagation like plant storage and handling, soil nutrients and pH, growing cycles and plant compatibility, plant selection, native versus foreign plant species, and other environmental variables which can increase or decrease the ability of vegetation to take root and perform its intended function.

Historically, the use of vegetation to stabilize slopes is not a modern enterprise. However, the quantification of engineering properties to correlate the ability of vegetation to increase stability is more recent. The use of vegetation to stabilize earthen slopes has "roots" dating back to ancient times. For instance, written accounts of the use of live staking date back to 1791, dike repair using soil bioengineering techniques in China date back to 28 B.C., and soil bioengineering methods were being used and codified throughout Europe by the 16th century (Finney 1993). The earliest documented application of bioengineering in the United States was the stabilization of mountainous service roads in California by the U.S. Forest Service (Krabel 1936). Excellent reviews of the history of soil bioengineering can be found in Greenway (1987) and Finney (1993).

Geotechnical analyses of slope stability which incorporate vegetation as a structural element differentiate modern engineered projects from ancient methods which derived from tradition and empiricism. Much of what has been practiced in the past has been based on trial and error where as the modern approach to soil bioengineering permits the engineer to calculate and predict the reinforcement capabilities of a design.

In recent years much effort has been directed to determining and quantifying the mechanics of root and soil interaction. Geotechnical, civil, wetland, and environmental journals and publications have witnessed an influx of articles addressing the growing use of vegetation for soil improvement by highlighting case studies and research. These efforts have produced a great deal of literature and provide the data that define the state of practice which engineers draw upon to incorporate vegetation into their designs. Much research has been conducted dealing with the various aspects and sub-disciplines associated with soil bioengineering and biotechnical stabilization for the stabilization of stream bank, wetland, riparian, and upland slopes.

A handful of books have been published addressing soil bioengineering as a viable means for slope stabilization and erosion control (Schiechtl 1980; Gray and Leiser 1982; Coppin and Richards 1990; Morgan and Rickson 1995; Gray and Sotir 1996; Schiechtl and Stern 1996; Schiechtl and Stern 1997; Barker et al. 2004). These texts provide comprehensive theoretical and practical treatment of the state of practice for biotechnical stabilization and soil bioengineering and may be used as guides for practice. In addition, agencies and other entities like the United States Department of Agriculture (USDA), U.S. Forest Service, state Departments of Transportation (DOT's), Transportation Research Board (TRB), and others, have supported extensive research projects and numerous case studies which have been presented in publications such as design manuals and technical papers (Lewis 2000; Lewis et al. 2001; WSDOT 2003; Steele et al. 2004).

It must also be recognized that soil bioengineering is a worldwide venture. From a global perspective, the bioengineering work that has been done in the United States represents a small fraction of what has been done internationally and much can be learned from our European and Asian counterparts. Nonetheless, the techniques employed in the global arena must be evaluated with a discerning eye because what may have succeeded in one particular part of the world is not guaranteed to work in another region. For instance, the tropical vegetation used for live pole stabilization in Malaysia would most certainly die in the climate of Marysville, Ohio. Moreover there is a vast geological difference between the glacial till of Ohio and the tropical soil of Malaysia. But on the other hand, the fundamental mechanics of a live pole design and analysis are not different and the Malaysian project can provide insight to the design of the Ohio project because the stabilizing benefits of vegetation, though different species, can be expected to behave similarly. Unlike typical "hard" geotechnical designs which rely on inert materials like earth, steel, wood and concrete, the survival of the chosen vegetation is critical to the success of a soil bioengineering design.

2.2 SOIL BIOENGINEERING DESIGNS

Soil bioengineering design methods range from installations that merely resist erosion to systems which provide slope stabilizing reinforcement and drainage through the strategic establishment of vegetation. Some examples of the numerous established techniques are live staking, live poles, fascines, brushlayers, vegetated geogrids, branchpacking, vegetated crib walls, live slope grating, wattle fences, furrow planting, and vegetated gabions. Comprehensive guides and design details for these methods can be found in Gray and Leiser (1982), Gray and Sotir (1996), and Schiechtl and Stern (1996). Table 2.1, summarizes some of the more common soil bioengineering systems, and, specifically, their construction and functions.

Sections 2.2.1 and 2.2.2 present in greater detail the bioengineering design schemes which have been specifically incorporated into the demonstration sites for this project; live poles and brushlayers. Guidelines for the installation and establishment of soil bioengineering projects are discussed in Schiechtl (1980), Gray and Leiser (1982), Coppin and Richards (1990), Gray and Sotir (1996), and Schiechtl and Stern (1996). The use of erosion control projects in conjunction with soil bioengineering techniques has been documented as well (Szymoniak et al. 1984; Di Pietro and Brunet 2002).

Although different hardwood species have been used successfully in bioengineered projects, the literature indicates that willow species are generally the most robust for live pole and brushlayer installations (Gray and Sotir 1996; Eubanks and Meadows 2002). Willow species in general possess good to excellent ability to root from cuttings. Additionally, they establish quickly, and are, in general, tolerant to flooding, salt, and deposition (Gray and Sotir 1996).

Table 2.1: Summary of bioengineering systems. (after Gray and Leiser 1982)

Name	Construction	Primary function(s)
1. Live stakes	Sticks are cut from rootable plant stock and tamped directly into the ground.	Live plants reduce erosion and remove water by evapotranspiration. Plant roots reinforce soil.
2. Live poles	Poles are cut from rootable plant stock and inserted into premade holes.	Same as 1.
3. Live faccine (wattling)	Sticks of live plant material are bound together and placed in a trench. They are tied to the ground by stakes.	Same as 1.
4. Brush mattress	Live branches are placed close together on the surface to form a mattress.	Same as 1. In addition, it provides immediate protection against erosion.
5. Brushlayer branchpacking	Live branches are placed in trenches or between layers of compacted fill.	Same as 1.
6. Vegetated geogrid	Live branches are placed in layers between compacted soil wrapped in geogrid.	The geogrid provides immediate stability. The plants serve the same functions as in 1.
7. Rooted plants	Rooted plants grown in a nursery or in the wild are planted.	Same as 1. In addition, roots provide buttressing.

2.2.1 Live Poles

Live pole planting is the installation of hardwood cuttings (i.e., poles) into a slope. A typical live pole installation could be one to two in. diameter willow cuttings, five to six ft. long, and placed perpendicularly into a slope on a grid pattern with spacing of two to three ft. (see Figure 2.1). Live poles have the ability to stabilize relatively steep slopes which are subject to shallow sliding and has been used successfully to stabilize highway slopes (Barker 2004; Steele et al. 2004). The live poles provide immediate mechanical stabilization similar to micropiles or soil nails. Over time, root development will provide additional mechanical reinforcement by binding the soil mass together, as well as, providing the hydrological benefits of reducing soil moisture, increasing evapotranspiration, and inducing negative pore pressures (Barker 2004).

2.2.2 Brushlayer

Live brushlayering is the placing of layers of live branches into a slope. Figure 2.2 shows a schematic brushlayer installation of a cut slope. Brushlayer designs use hardwood (e.g., willow, alder, and dogwood) branches which can extend into the slope as much as twelve ft. in some applications (Gray and Sotir 1996). This stabilization technique is applicable for the impediment of shallow sliding and provides erosion protection. Erosion protection is achieved by intercepting and reducing the velocity of runoff water which transports or erodes soil. Brushlayers can be constructed entirely of vegetative material or the design can incorporate natural or synthetic reinforcement. Typical natural reinforcement may include coir fabric and synthetic reinforcement may be achieved with geogrids.

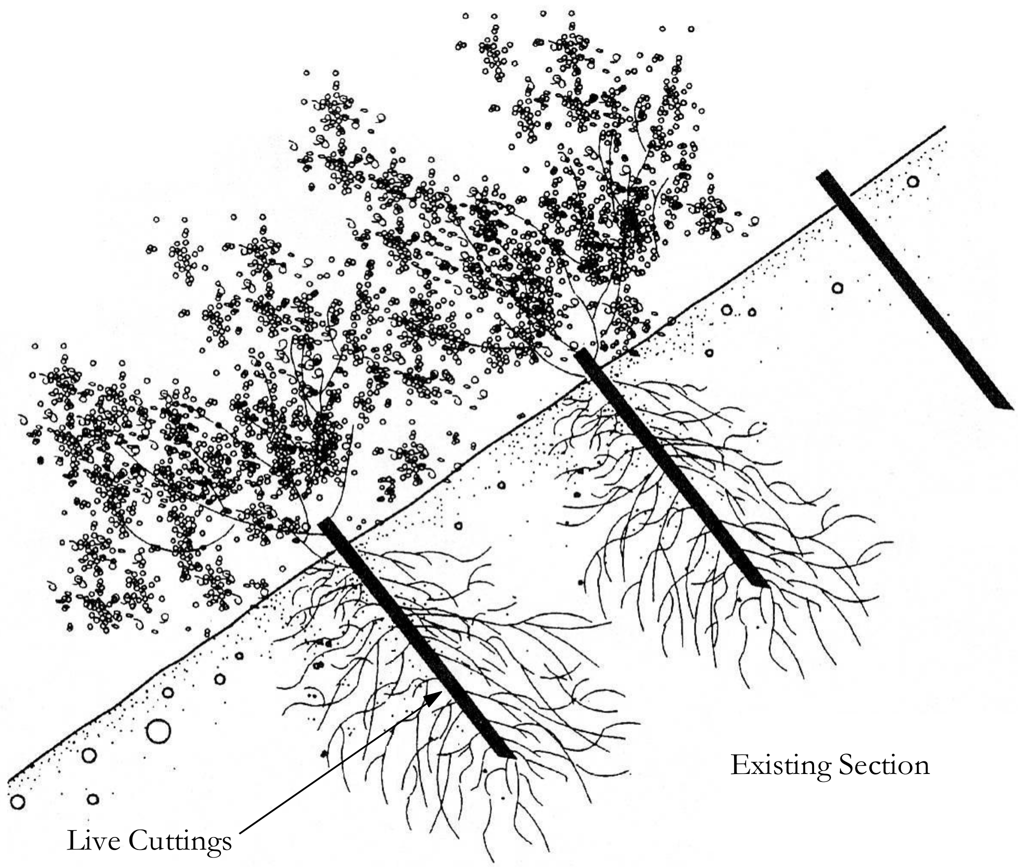
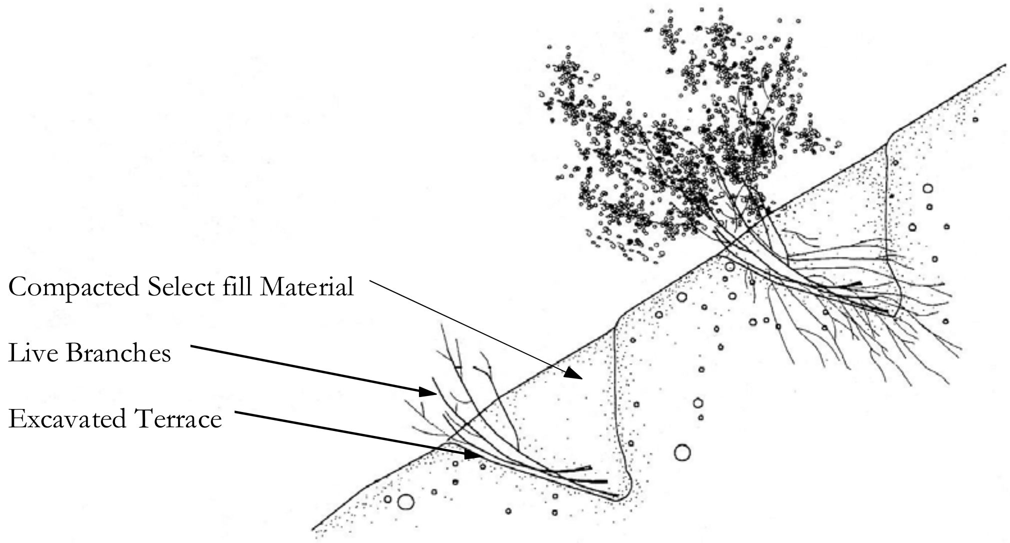
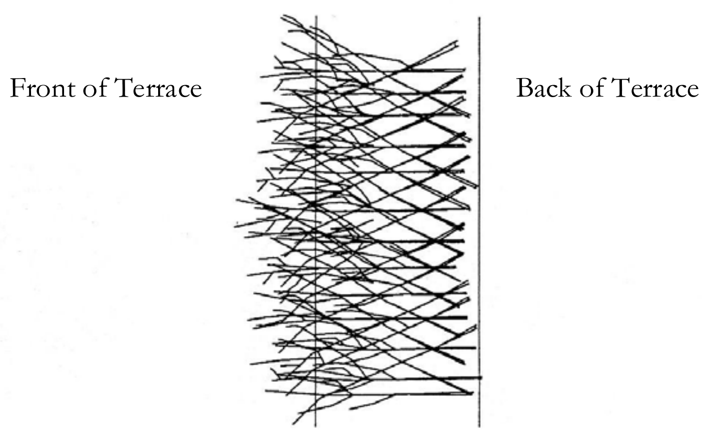


Figure 2.1: Schematic diagram of an established live pole installation. (Gray and Sotir 1996)



Section View



Plan View

Figure 2.2: Schematic diagram of an established, growing *cut slope* brushlayer installation showing alternating layers of live cut brush placed on narrow benches or terraces excavated in the slope. (Gray and Sotir 1996)

2.3 VEGETATION

Vegetation selection is an essential aspect of bioengineering design. The use of native versus exotic species and plant availability are important considerations. Hardwood species like willow, cottonwood, poplar, and dogwood have been used successfully for bioengineering construction throughout the United States (Gray and Sotir 1992; Gray and Sotir 1995; Lewis et al. 2001; WSDOT 2003) with willow being the species of choice for the majority of applications. Gray and Sotir (1996) outline the location, availability, habitat value, size/form, root type, and rooting ability from cuttings for suitable soil bioengineering plant species, as well, as plant tolerance to deposition, flooding, drought, and salt. These tables are useful as they provide a guide for the selection of bioengineering plant material. Additionally, one should consider the use of native material before introducing exotic plant species because native varieties are typically better acclimated to localized climate and environment (Gray and Sotir 1996). However, a case can be made for choosing introduced species in some instances where aesthetics or availability may be of concern (Gray and Leiser 1982).

Because the success of a bioengineering project relies on the propagation of the chosen vegetation, careful attention must be placed on environmental factors like climate and soil vitality when choosing the vegetation for a bioengineered project and a carefully monitored maintenance schedule should be followed to hedge off any potentially detrimental occurrences. The vegetation used in this project was limited to hardwood species. For this reason little or no reference has been made to various other types of plants, shrubs, grains, grasses and turfs, which are often employed for various erosion and stability ventures. The interested reader is referred to Coppin and Richards (1990) and Gray and Sotir (1996) for guidelines on the use of these other flora. Additionally, the focus of the literature presented herein has been on upland slope stabilization because of the relevance to the project demonstration sites and little has been discussed on stream bank slopes, riparian slopes, and wetland slopes. Thorough treatments on non-upland soil bioengineering and biotechnical stabilization can be found in Schiechtl and Stern (1997), Fotherby et al. (1998), and Eubanks and Meadows (2002).

2.3.1 Collection

Hardwood cuttings (willow and poplar), which are used in many bioengineering applications, must be harvested and installed in their dormancy. The dormant period is generally during the winter after a hard frost has occurred and before budding. Hardwood cuttings are generally harvested using conventional tree trimming tools like pruners, loppers, tree saws, chain saws, and brush saws. Additionally, cuttings should be taken 8 to 10 in. above the ground surface so that the host plants can regenerate (Gray and Sotir 1996).

2.3.2 Storage

In some instances, the harvest of vegetation and installation times may not coincide; therefore, it may be necessary to store the cuttings for some period. Refrigerated storage, such as commercial cooler/freezer, refrigerated truck, or barn with suitable conditions, offers a solution for allowing delayed, late spring, planting. Another alternative for storage is "heeling in" where the cuttings are temporarily planted in loose soil during the dormant season and then dug up and moved to the permanent installation later (Rowe 2005). Research into the effects of temperature, moisture, and duration of storage on hardwood cuttings has been conducted by Cram and Lindquist (1983) and Volk et al. (2004). The consensus is that the optimal environment for the refrigerated storage of hardwood cuttings is 34°F and 90% humidity (Gray and Sotir 1996).

2.3.3 Environmental Considerations

Soil pH and nutrient and metal concentrations should be within acceptable limits. In some cases it may be necessary to fertilize or otherwise treat the soil to promote favorable growing conditions (Gray and Sotir 1996). Soil texture has been reported by Schaff et al. (2003) to be the dominate factor in determining black willow cutting growth, health, and survivability with coarse-grained soils (sands) being the most conducive. The tolerance of riparian willow and cottonwood species to water table decline has been studied by Amlin and Rood (2002) and their findings suggest that a gradual water table decline tends to promote shoot and root growth and, conversely, a rapid decline induces mortality with willow being the more vulnerable of the two genera. One must also be aware of site microclimate conditions, for example, areas susceptible to drought or heavy rainfall uncharacteristic of the surrounding area's climate. Also, the aspect or slope facing direction is an environmental consideration which must not be overlooked. Slopes receive substantially less sunlight on north facing slopes in the northern hemisphere than south facing slopes. The amount of sunlight can have an effect on the stability of an earthen slope by influencing both plant survivability and the near surface groundwater regime.

Pre-planting soaking has been shown to be beneficial for the survival of hardwood cuttings and Schaff et al. (2002) recommends a ten day pre-planting soaking for black willow cuttings. Gray and Sotir (1996) state that live cuttings must be protected from drying up and should be heeled into moist soil or kept in water prior to planting.

2.4 SLOPE STABILITY

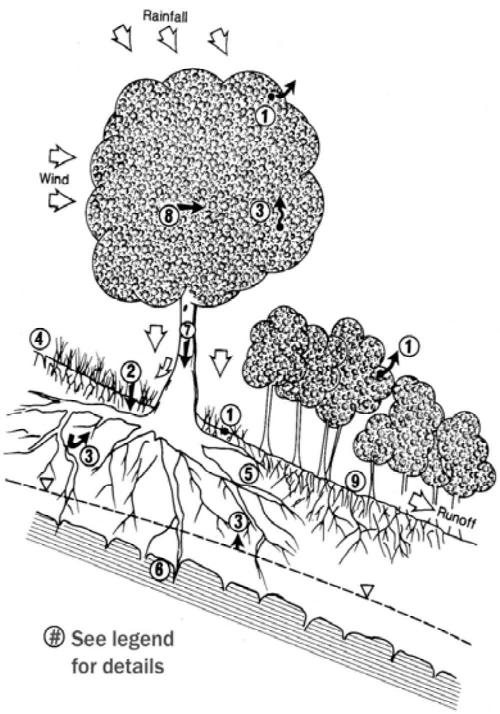
A comprehensive review of the state of practice for slope stability analysis for vegetated slopes can be found in Gray and Leiser (1982), Wu (1995), and Gray and Sotir (1996). Wu (1995) gives a thorough treatment of the mechanics for vegetated slope stability analysis, the determination of the vegetative contribution to stability, soil root interaction, and reliability analysis for slope stability calculations.

The key factors that reduce the stability of a slope by either contributing to high shear stress or to low shear strength and, consequently, reduce the factor of safety against sliding, are outlined in Table 2.2. Attention paid to these factors is important for the analysis of both bioengineered and traditional or non-bioengineered slope stability designs.

Greenway (1987) has outlined the hydrological and mechanical effects of vegetation on a typical slope (Figure 2.3). The importance of this graphic is that it shows physically the mechanisms at work on a slope. It depicts the benefits and limitations that must be considered when choosing a soil bioengineering method and when analyzing the stability. Vegetation may have an overall stabilizing or destabilizing effect on the slope and this can change over time due to seasonal variances and other perceivable factors. For example, a seemingly stable vegetated slope may be undermined by excessive wind or unusually heavy rainfall.

Table 2.2: Factors contributing to instability of earth slopes. (after Varnes 1958) (from Gray and Leiser 1982)

Factors That Contribute to High Shear Stress	Factors That Contribute to Low Shear Strength
A. Removal of lateral support	A. Initial state
1. Erosion – bank cutting by streams and rivers	1. Composition – inherently weak materials
2. Human agencies – cuts, canals, pits, etc.	2. Texture – loose soils, metastable grain structures
B. Surcharge	3. Gross structure – faults, jointing, bedding, planes, varving, etc.
1. Natural agencies – weight of snow, ice, and rainwater	B. Changes due to weathering and other physico-chemical reactions
2. Human agencies – fills, buildings, etc.	1. Frost action and thermal expansion
C. Transitory earth stresses – earthquakes	2. Hydration of clay minerals
D. Regional tilting	3. Drying and cracking
E. Removal of underlying support	4. Leaching
1. Subaerial weathering – solutioning by groundwater	C. Changes in intergrannular forces due to pore water
2. Subterranean erosion – piping	1. Buoyancy in saturated state
3. Human agencies – mining	2. Loss in capillary tension upon saturation
F. Lateral pressures	3. Seepage pressure of percolating groundwater
1. Water in vertical cracks	D. Changes in structure
2. Freezing water in cracks	1. Fissuring of preconsolidated clays due to release of lateral strain
3. Swelling	2. Grain structure collapse upon disturbance
4. Root wedging	



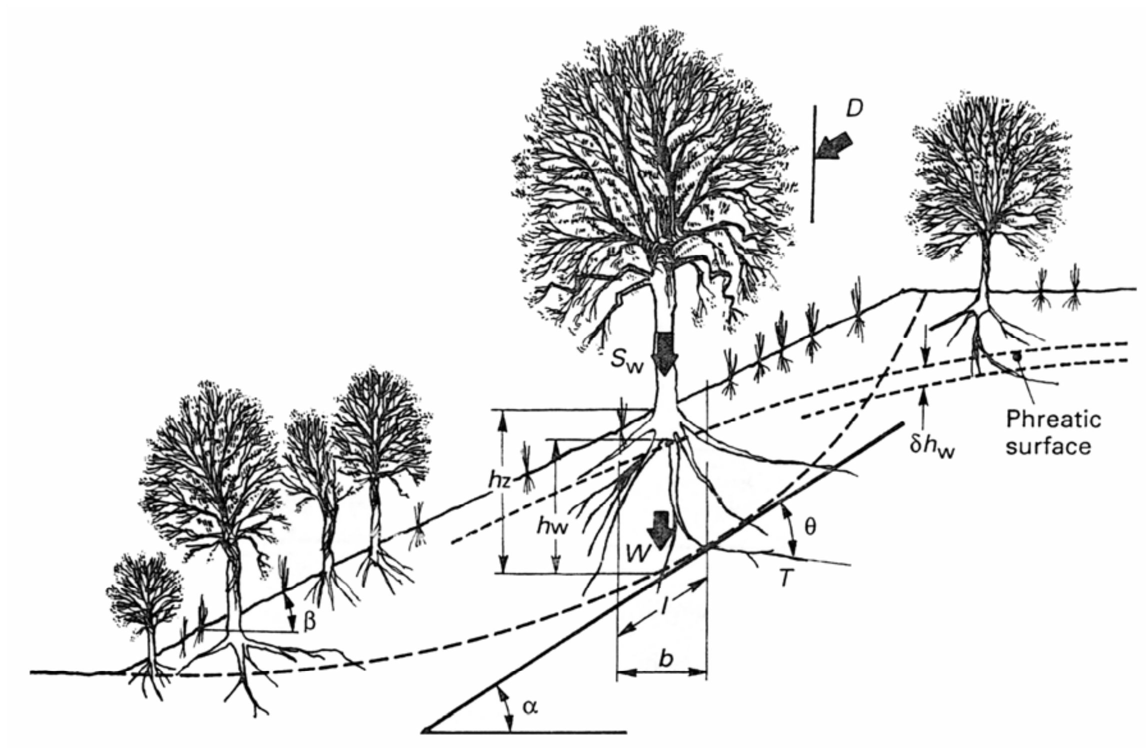
Hydrological Mechanisms	***Influence***
1. Foliage intercepts rainfall, causing absorptive and evaporative losses that reduce rainfall available for infiltration.	Beneficial
2. Roots and stems increase the roughness of ground surface and permeability of the soil, leading to increased infiltration capacity.	Adverse
3. Roots extract moisture from the soil which is lost to the atmosphere via transpiration, leading to lower pore-water pressures.	Beneficial
4. Depletion of soil moisture may accentuate desiccation cracking in the soil, resulting in higher infiltration capacity.	Adverse

Mechanical Mechanisms	
5. Roots reinforce the soil, increasing shear strength.	Beneficial
6. Tree roots may anchor into firm strata, providing support to the upslope soil mantle through buttressing and arching.	Beneficial
7. Weight of trees surcharges the slope, increasing normal and downhill force components.	Adverse/ beneficial
8. Vegetation exposed to wind transmits dynamic forces into slope.	Adverse
9. Roots bind soil particles at the ground surface, reducing susceptibility to erosion.	Beneficial

Figure 2.3: Slope-vegetation interactions influencing stability. (after Greenway 1987)

2.4.1 Stability Analysis

The ability to quantify the stability of earthen slopes is of paramount importance to the geotechnical engineer. Stability problems involving shallow movement are typically analyzed using limit equilibrium approaches like infinite slope models (Taylor 1948) or the many circular or non-circular analysis methods (Bishop 1955; Janbu et al. 1956; Morgenstern and Price 1965; Spencer 1967). An overall comparison of slope stability methods can be found in Fredlund and Krahn (1977). Chok et al. (2004) have presented the use of finite element slope stability analysis for vegetated slopes. A comparison of limit equilibrium to an energy approach stability method, both taking account for vegetation, can be found in Ekanayake et al. (2004). Unlike the limit equilibrium approach, the energy approach (EA) (Ekanayake and Phillips 1999; Ekanayake and Phillips 2002) attempts to integrate the shearing resistance of the root-enhanced soil mass. Discussion on the validity of the EA's fundamental assumptions (Ekanayake and Phillips 2003; Wu 2003) has highlighted some of the model's shortcomings which need to be addressed before this approach can be considered bona fide. However, this method shows promise as a tool for the stability analysis of vegetated slopes. Additionally, thorough treatments on theory and application of slope stability analysis can be found in Lambe and Whitman (1969), Duncan (1996), and Abramson et al. (2002). Figure 2.4 shows the parameters applied in slope stability analysis where the influences of vegetation are taken into account.



W	Total weight of soil slice
c', ϕ'	Effective strength parameters at slip surface
l	Length of slip surface with slice
u	Pore-water pressure at slip surface
u_v	Decrease in pore-water pressure to evapotranspiration by vegetation at slip surface
c'_R	Enhanced effective soil cohesion due to root matrix reinforcement by vegetation along slip surface
c'_S	Enhanced effective soil cohesion due to soil suction due to evapotranspiration by vegetation at slip surface
S_w	Surcharge due to weight of vegetation
D	Wind loading force parallel to slope
T	Tensile root force acting at base of slice
h_z	Vertical height of surface of soil layer above slip plane
h_w	Vertical height of phreatic surface or water table above slip surface

Due to vegetation

Figure 2.4: Parameters applied in slope stability analysis. (after Coppin and Richards 1990)

2.4.1.1 Infinite Slope Analysis

Analysis methods which quantify the effect of vegetation on slope stability have been published by Wu (1984), Barker (1986), Gray (1994), Wu (1995), Ekanayake and Phillips (1999), Wu and Watson (1999), and Ekanayake and Phillips (2002). Barker (1986) presents an infinite slope model for slope stability analyses which includes the stabilizing/destabilizing effects of vegetation. In this analysis, poles of live plant stock are modeled as micropiles which provide lateral resistance to the potential sliding mass.

The classical "infinite slope" analysis procedure (Taylor 1948) is appropriate for analyzing the stability for shallow, transitional slides. This method is suitable for slopes where the slip surface can be assumed to be parallel to the ground surface and the depth to length ratio of the sliding mass is small. In other words, the infinite slope approach is suitable for the sliding of a long shallow mass of soil. The geometry of the infinite slope simplifies the analysis to that of a single element where the forces acting on the element's sides are equal, opposite and collinear, and the overall end effects in the sliding mass can be ignored. Because this approach is relatively simple, one may incorporate nearly every conceivable force which may act on a slope. For this reason, the infinite slope analysis can assume many forms and may be analyzed using drained or undrained shear strength parameters provided one is consistent in using each approach with regard to the groundwater conditions; the two basic approaches are (1) total soil unit weight and boundary pore water pressure or (2) buoyant soil unit weight and seepage forces.

The infinite slope analysis uses force equilibrium where the ratio of the stabilizing and the destabilizing forces acting on the element are identified and compared to yield a factor of safety. Two examples of the infinite slope procedure are presented in Gray and

Sotir (1996); (1) a general form of the infinite slope analysis for determining the factor of safety against sliding for a slope with surcharge and water table, and (2) a modified infinite slope model which accounts for seepage and seepage direction, root contributions to increased soil shear strength or root cohesion, and vertical surcharge. The factor of safety for the first case, a slope with surcharge and water table, is determined by Equation 2.1 and Figure 2.5 shows a schematic representation.

$$FS = \frac{[c'/\cos^2\beta\tan\phi' + (q_0 + \gamma H) + (\gamma' - \gamma)H_W]\tan\phi'/\tan\beta}{(q_0 + \gamma H) + (\gamma_{SAT} - \gamma)H_W} \tag{2.1}$$

where: ϕ' = effective angle of friction

c' = effective cohesion intercept

β = slope angle of natural ground

γ = moist density of soil

γ' = buoyant density of soil

γ_{SAT} = saturated density of soil

H = vertical thickness (or depth) of sliding surface

H_W = piezometric height above sliding surface

q_0 = uniform vertical surcharge stress on slope

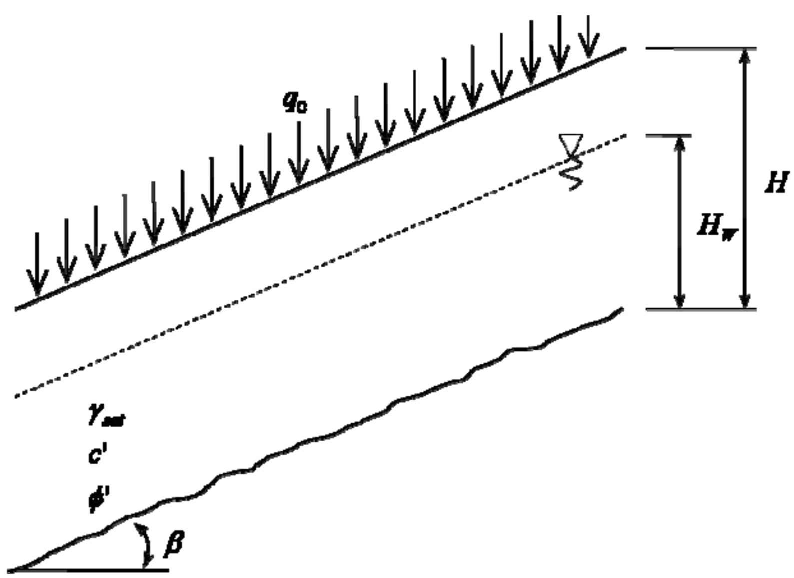


Figure 2.5: Schematic representation of infinite slope with surcharge and water table. (after Gray and Sotir 1996)

The effects of vegetation may be readily incorporated into the infinite slope analysis. The factor of safety for the case where seepage and seepage direction, root cohesion, and no uniform vertical surcharge, is determined by Equation 2.2 and Figure 2.6 shows a schematic representation.

$$FS = \frac{A \tan \phi}{\tan \beta} + \frac{B(c + s_r)}{\gamma H} \tag{2.2}$$

$$A = \frac{1 - r_u}{\cos^2 \beta} \tag{2.3}$$

$$B = \frac{1}{\cos \beta \, \sin \beta} \tag{2.4}$$

$$r_u = \frac{\gamma_w}{\gamma + \gamma \tan \beta \, \tan \theta} \tag{2.5}$$

where: β = slope angle of natural ground

θ = seepage angle (with respect to horizontal)

ϕ = angle of internal friction

c = soil cohesion

s_r = root cohesion

γ = soil density

γ_w = density of water

H = vertical thickness (or depth) of sliding surface

The root cohesion term, s_r, takes into account the influence of root reinforcement and may be determined based on experience, from published values, or from either laboratory or in situ shear strength tests. This version of the infinite slope analysis also allows the engineer to incorporate the vegetation effect on the soil moisture regime. The moisture content and pore pressure/matric suction at depth within a slope can be accurately measured with instruments such as tensiometers, piezometers, time domain reflectometry (TDR), and porous blocks. The seepage direction, θ, is determined by identifying pore pressure/matric suction gradients within the slope. Because vegetation removes water from the soil, vegetation will have an effect on the seepage forces as well as soil density, γ.

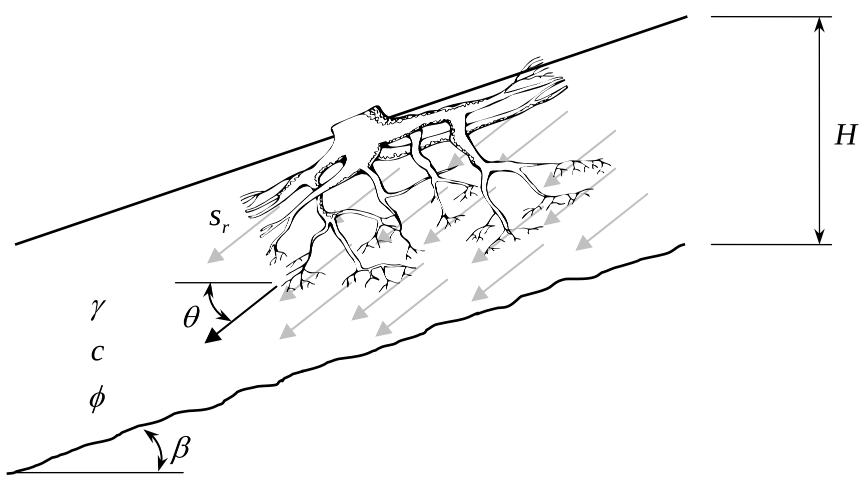


Figure 2.6: Schematic representation of infinite slope with seepage and root cohesion. (after Gray and Sotir 1996)

2.4.1.2 Root Reinforcement

A root reinforcement model must also be incorporated into the overall stability analysis. An appropriate slope stability analysis will include the beneficial and adverse effects of vegetation, satisfy the site geometry and include reasonable assumptions. The appropriate model/analysis depends on the problem-specific geometry and model assumptions and may assume varying degrees of complexity, which will be dictated by the available information, resources, and experience of the engineer. Comprehensive reviews of soil-root models which quantify the contribution of roots to shear strength may be found in Gray and Leiser (1982), Choppin and Richards (1990), Morgan and Rickson (1995), and Gray and Sotir (1996). Proposed by Wu et al. (1979), Equation 2.6 is an approach to calculate the increase in shear strength attributed to the root reinforcement of soil, s_r. Wu et al. (1979) found that s_r is nearly constant and would have the characteristic of cohesion for the range of shear distortion, θ, from 48° to 72°. For clarification; throughout the literature, the root's contribution to shear strength, s_r, is often referred to as "root cohesion" and represented with the notation c_R.

$$s_r = 1.2\sigma_t A_r \tag{2.6}$$

where: σ_t = tensile strength of the roots

A_r = root area ratio

Typical values of, s_r, are presented in the following table (Table 2.3).

Table 2.3: Published values of "root cohesion", s_r. (after Schmidt et al. 2001)

s_r (kPa)	Vegetation type	Location	Source
Measurement of root diameter and thread strength			
3.5-7.0*	Sphagnum moss	Alaska	Wu 1984a
5.6-12.6*	Hemlock, sitka spruce, yellow cedar	Alaska	Wu 1984b
5.7†	Sugar maple	Ohio	Riestenberg and Sovonick-Dunford 1983
6.2-7.0	Sugar maple	Ohio	Riestenberg and Sovonick-Dunford 1983
5.9*	Alaska cedar, hemlock, spruce	Alaska	Wu et al. 1979
7.5-17.5	Douglas-fir	Oregon	Burroughs and Thomas 1977
In situ direct shear test			
1.0-5.0†	Japanese cedar	Japan	Abe and Iwamoto 1986
2.0-12.0†	Alder nursery	Japan	Endo and Tsuruta 1969
3.0-21.0†	Lodgepole pine	California	Ziemer 1981
3.7-6.4†	54-month-old yellow pine	Laboratory	Waldron et al. 1983
~5†	52-month-old yellow pine	Laboratory	Waldron and Dakessian 1981
6.6†	Beech	New Zealand	O'Loughlin and Ziemer 1982
Back-calculation			
1.6-2.1†	Grasses, sedges, shrubs, sword fern		
2.6-3.0†	Red alder, hemlock, Douglas-fir, cedar	Washington	Buchanan and Savigny 1990
2.02†	Bluberry, devil's club	Alaska	Sidle and Swanston 1982
2.8-6.2†	Ponderosa pine, Douglas fir, Engelmann spruce	Idaho	Gray and Megahan 1981
3.4-4.4†	Hemlock, spruce	Alaska	Swaston 1970

* Root cohesion representing lateral reinforcement. † Root cohesion representing basal reinforcement

The stabilizing or reinforcing properties of plant roots has been evaluated and has been well established (Wu et al. 1979; Wu 1984; Wu and Watson 1998; Wu and Watson 1999; Kirsten 2001; Goldsmith 2006). These studies include laboratory testing, field-testing, and the observance of the effects from clear felling and timbering on slope stability. Wu et al. (1979) explored the stability of slopes on Prince of Wales Island, Alaska, prior to and after the removal of forest cover. Numerous slope failures were observed following the clear cutting of timber and this study concluded that tree roots contributed to an increase of around 5.9 kPa in the shear strength of the soil. Also, direct shear tests results published by Goldsmith (2006) show that the relative strength increase (i.e., soil shear resistance) of vegetated as compared to unvegetated slopes at controlled horizontal displacements increased by 445% and 216% for black willow *(Salix nigra)* and common cottonwood *(Populus deltoids)*, respectively. For black willow root permeated soil the reported shear strength increased to 36.0 kPa in comparison to the unvegetated soil shear strength of 6.6 kPa at displacements of 7 cm. For common cottonwood root permeated soil the reported shear strength increased to 20.9 kPa in comparison to the unvegetated soil shear strength of 6.6 kPa at displacements of 7 cm.

2.5 SELECT CASE HISTORIES

This section presents three case histories which highlight the utilization of bioengineering technologies and which have direct relevance to the methods explored for this research project. For this reason, emphasis has been placed on projects involving upland rather than stream bank or riparian ecology and designs calling specifically on live pole and live brushlayer approaches. Nonetheless, numerous case histories have been published covering the entire gamut of bioengineered projects. For example, case histories detailing stream bank, wetland, and riparian installations have been presented by Sadlon (1993), Duncan et al (1998), Fotherby et al. (1998), and Sotir (1998), to name a few. Additionally, upland projects drawing on methods other than live brushlayer and live pole installations may be found in Gray and Sotir (1992), Coppin et al. (1995), Gray and Sotir (1995), Lewis (2000), and Lewis et al. (2001).

2.5.1 United Kingdom Live Willow Pole Trial – Iwade

The live willow pole installation in Iwade was the first trial of the "pole" technique in the United Kingdom and much has been learned from the shortcomings of this effort (Barker 1997; Steele et al. 2004). The lessons learned from this project have clearly led to improving the technique and this insight has led to improved installation methodology that have been adopted and implemented in the 2000-2001 live willow pole trials (see section 2.5.2), which in comparison, was a much more successful effort.

The Iwade project was riddled with problems leading to an overall survival rate of the installed poles to a mere 15%. Several of the factors which led to poor pole survival were; limited willow stock, delayed/late season installation, contractor inexperience, and poor installation procedures (Steele et al. 2004). It was reported that in 2003 some poles had grown in excess of 4 to 5 m indicated that live willow poles have the potential for continued long term survival (Steele et al. 2004) despite the overall poor survival rate. On a more positive note, an exhumed pole from the site after three and a half years had root growth down to 2 m (Steele et al. 2004) which indicates the potential for slope stabilization to such depths.

2.5.2 United Kingdom Live Pole Trials (2001-2002)

A compressive research effort investigating the validity of bioengineering techniques, similar in scope to the project presented in this thesis, was conducted in the United Kingdom in the early 2000s. The installation and establishment of live willow poles on four highway slopes in the UK has been documented in TRL Report TRL619 (Steele et al. 2004). Some 900 live willow poles were installed in 2000-2001 at the following sites: A10 Hoddesdon, M1 Toddington, A5 Milton Keynes, and M23 Gatwick. The four sites which were selected for the study had a general history of shallow transitional failures. Table 2.4, outlines the details of the field sites.

Table 2.4: UK live willow pole trials. (after Steele et al. 2004)

Site	General soil conditions	Type	Slope height (meters)	Slope (H:V)	Facing	No. of poles	Comments
A10 Hoddesdon	Boulder Clay overlying deposits of London Clay	cut	5 to 6	3:1	WNW	67	
M1 Toddington	Gault Clay	fill	6 to 7	2:1	SW	126	
A5 Milton Keynes	Oxford Clay	cut	10	4:1	SSW	72	
M23 Gatwick	Weld Clay	fill	3.5	2:1	W	625	Mycorrhizal treatment was used on 211 poles

This series of trials defines the second generation of live willow pole installation in the UK. The shortcomings of a previous trial in Iwade (see section 2.5.1) were drawn upon to refine the live willow pole installation technique to maximize the survival rate and success of such projects. Special attention was placed on controlling as many factors as conceivably possible (e.g., pole size, installation time, species selection, hole formation method, pole preparation, installation, backfilling, above ground protection, and mycorrhizal (antifungal) treatment) to encourage high survival rates. Draft specifications are included in the appendix of this report and are an invaluable guideline for the installation of live willow poles. After three years of monitoring, an overall survival rate of the 900 installed live willow poles of 91% was achieved and the slopes were stable.

2.5.3 Brushlayer Fill – Cut Highway Slope Colrain, Massachusetts (Gray and Sotir 1992; Gray and Sotir 1995; Sotir 1995; Gray and Sotir 1996)

To preserve the scenic character, a bioengineering approach was chosen to stabilize a cut slope along Greenfield road near the village of Colrain in Massachusetts in 1989 (Figure 2.7). Road improvements and widening along this road resulted in an unstable cut slope in residual silty sand overlying fractured quartz-mica schist bedrock. The 1.5H:1V unstable cut measured approximately 1,200 ft. in length with heights ranging between 20 to 60 ft. Failures in this slope generally consisted of small slipouts and slumping and, in addition, a substantial amount of groundwater seepage flowing from the cut was observed.

Figure 2.7: Brushlayer fill during first growing season – Greenfield road near the village of Colrain in Massachusetts. Brushlayers have rooted and leafed out. (Gray and Sotir 1996)

To address the scenic character, global stability, shallow stability, and groundwater seepage, a soil biotechnical solution was applied which consisted of a composite rock toe and earthen brushlayer buttress fill. The stability analyses showed that the rock buttress at the bottom intercepted the critical global failure surface, which passed through the toe of the slope, resulting in an increase in the factor of safety from 1.0 (i.e., failure condition) to 1.5 as calculated by the simplified bishop method (Gray and Sotir 1996).

Live brushlayers and live fascines constructed from willow cuttings were used to successfully mitigate the stability issues associated with the over-steepening of this highway slope. Only minimal erosional problems associated with surface runoff were experienced following construction and were addressed by the construction of a brow ditch. It is reported by Gray and Sotir (1996) that three years following the construction the slope was stable, well vegetated, and had assumed a natural and pleasing appearance.

CHAPTER 3

PROJECT OVERVIEW

3.1 INTRODUCTION

This chapter describes the activities completed for this research project, including the selection criteria and identification of candidate sites, the demonstration site investigation/characterization, site instrumentation and monitoring, sources of vegetation, laboratory testing, stability analysis and design, and demonstration site construction.

3.2 SITE SELECTION CRITERIA AND IDENTIFICATION OF CANDIDATE SITES

The field demonstration sites for this research project allow the implementation of bioengineering methods to be studied in depth to further understand the value and limitations of such designs. As an initial step, specific selection criteria were established to determine the appropriateness of potential demonstration sites. Potential sites were then located, investigated, and graded using the established criteria.

In order to properly implement the bioengineering methods for this research study, the limitations and the appropriate use of vegetative materials as a means for slope stabilization were identified by the project team. Bioengineering designs can only provide stabilization against shallow movements where the sliding surface does not extend beyond the depth of the root systems. For this reason, only sites clearly undergoing shallow slope failures less than 5 ft. deep were considered for demonstration site construction. The search was further limited to sites capable of hosting side-by-side comparisons to evaluate the effectiveness of conventional, bioengineering, and a combination of bioengineering and geosynthetic stabilization techniques. Additionally, it was decided by the research team that overly complicated sites should be avoided, as complicated site conditions would introduce uncertainty into the design process and the subsequent interpretation of design performance. Other site selection factors applied by the research team during the candidate site selection include construction/repair priority to ODOT, slope geometry, failure mode(s), and our understanding of the site conditions (i.e., soil properties and groundwater and surface water conditions).

To facilitate the selection of suitable field demonstration sites, communication between the research team and the ODOT districts was initiated. During the selection process over 40 individual sites located throughout the state of Ohio were visited and evaluated. These sites covered the gamut of landslides including cut, fill, and stream banks with instabilities ranging from surficial erosion to deep-seated rotational failures. A summary of the landslides visited during the selection process is presented in Table 3.1. Following the extensive inventory of the known landslides identified by the ODOT district offices, three field demonstration sites were chosen: US-33/SR-347 in Logan County, I-70/SR-83 in Muskingum County, and US-33 in Union County near Marysville.

Table 3.1: Landslide site evaluation inventory.

Date	Site location	Observations/notes
		Slope/Slide geometry/Soil
8/2/2004	Logan Co. (Dist.7): LOG-33/347 Infield of 347 ramp to 33 EB	Erosion and shallow slips observed. Environmentally sensitive area: Big Darby Creek.
		Cut slope /2H:1V, Shallow and deep slide(s): 225 ft. long (main slip area 75 ft. to 225 ft.); 20 ft. high
8/9/2004	Perry Co. (Dist. 5)	Toe erosion.
		Stream bank
8/9/2004	Muskingum Co. (Dist. 5): North of SR-666 and Muskingum Valley	Small slide.
		Cut slope/clay
8/9/2004	Muskingum Co. (Dist. 5): SR-83/I-70 overpass	Cracks observed in slope area separating overpasses. Scour observed under concrete drainage channel and overpass. Over five years old.
		Cut slope/2.9H:1V, slide may be deep, 70 ft. long
8/9/2004	Muskingum Co. (Dist. 5): SR-83 to westbound I-70 ramp	Slumps observed on slope. Grass observed on slope.
		Fill slope/2H:1V, shallow slide, 57 ft. long/Shale
8/9/2004	Muskingum Co. (Dist. 5): Westbound I-70, east of SR-83	Red clay slide.
		Cut slope/Deep slide/Clay
8/9/2004	Guernsey Co. (Dist. 5): Across from rest area on I-70	Slide repair a year ago in slope in median.
		Fill slope
8/10/2004	Noble Co. (Dist. 10): Belle Valley Interchange SR-821 & I-77	
		Cut slope/2.5H:1V
8/10/2004	Noble Co. (Dist. 10):	Erosion and sloughing not threatening the road.
		Cut slope
8/10/2004	Noble Co. (Dist. 10): North of Caldwell	Emergency slip to be repaired Summer 2004.
		Fill slope/2H:1V/Shale
8/10/2004	Noble Co. (Dist. 10): Exit 25 on I-77, Caldwell exit	Erosion.
		Steep slope/Shale

Continued

Table 3.1 (Continued)

Date	Site location	Observations/notes: Slope/Slide geometry/Soil
8/10/2004	Noble Co. (Dist. 10): Mile marker 21 on SB I-77; mile 20.8	Slide along guardrail.
8/10/2004	Noble Co. (Dist. 10): I-77 beginning of narrow median	Emergency failure along I-77. Fixed with drilled shafts.
8/10/2004	Noble Co. (Dist. 10): I-77 Before mile marker 19 across from exposed rock	Small slides and erosion.
		Cut slope
8/10/2004	Noble Co. (Dist. 10): I-77, Noble/Washington Co. line	Stable.
		Fill slope/2H:1V
8/10/2004	Washington Co. (Dist. 10): I-77 Macksberg Exit	Many small slides and erosion observed.
		Cut slope/Shale
8/10/2004	Washington Co. (Dist. 10): I-77	2 year old fix w/ jute mat, ODOT pleased with performance.
		Fill slope/2H:1V
8/10/2004	Washington Co. (Dist. 10): Eastside of I-77, mile marker 15	Exposed geology. Small rock & shale sliding towards road.
		Cut slope
8/10/2004	Washington Co. (Dist. 10): STA 577+00, just south of mile marker 11	50 ft. slide area between two old fixes using geotextile reinforcement.
		Slide 50 ft. long
8/12/2004	Warren Co. (Dist. 8): WAR-123-20.70 Left side of SR-123 just north of Greentree Road	Undercutting erosion on stream bank bend past existing concrete revetment below SR-123. Resident claims that water level reaches 8 ft. above current height during peak rainfall events.
		Stream bank
8/12/2004	Warren Co. (Dist. 8): WAR-123-16.75, westbound right side	Stream bank erosion undercutting SR-123. Armor rock has been placed to reduce erosion. Rock is falling, not staying in place.
		Stream bank

Continued

Table 3.1 (Continued)

Date	Site location	Observations/notes Slope/Slide geometry/Soil
8/12/2004	Hamilton Co. (Dist. 8): HAM-126-7.27, eastbound at the 3.4 Artimis marker	Scarp observed below the noise wall on slope. Rock has been placed per ODOT standard fix for "spring areas". Cut slope/Till
8/12/2004	Hamilton Co. (Dist. 8): HAM-126-7.73, eastbound at the 3.95 Artimis marker	Similar to HAM-126-7.2. Scarp observed below noise wall on slope. Cut slope/Scarp approx. 80' long/Till
8/12/2004	Hamilton Co. (Dist. 8): HAM-275-15.27, westbound, near Pebble Creek GC	Small slide above Bridge No. HAM-275-1527. Fill slope/Shale, soil, rock
8/12/2004	Hamilton Co. (Dist. 8): HAM-74-14, southside of I-74	Soft moist ground. Fill slope
8/12/2004	Hamilton Co. (Dist. 8): HAM-29-13	Asphalt cracking along shoulder.
8/12/2004	Hamilton Co. (Dist. 8): HAM-50-32.2	Slides washing debris onto SR 74 during rainfall events. Two large gullies have been formed from erosion/slides in washout area. Cause: possibly blocked culverts up slope.
8/12/2004	Clermont Co. (Dist. 8): CLE-50-7.20	Scarp along guardrail. Cracking observed on roadway. Site observed to be heavily vegetated.
8/12/2004	Clermont Co. (Dist. 8): CLE-50-7	Scour under culvert. Stream bank
8/18/2004	Brown Co. (Dist. 9): BRO-52-10.4, slope between SR-32 and Ohio River	Possible rapid drawdown? Locust trees growing on slope. Road patched several times a year. Extensive settlement in road and guardrail for a distance of approx. ¼ mile in length. Slip starting as far in as the center of SR-32. Large mature trees present in worst parts of slide area. Stream bank/Deep slide; ¼ mile long

Continued

Table 3.1 (Continued)

Date	Site location	Observations/notes Slope/Slide geometry/Soil
8/18/2004	Brown Co. (Dist. 9): BRO-68, ¼ mile north of Ripley SR-52 exit. East side of road	Site fixed in 2003: upslope road and guardrail replaced after slide. No distress visible at top; active erosion. Ditch debris removed after slide event caused by heavy rain. Cut slope/1H:1V/Rock and slate
8/18/2004	Adams Co. (Dist. 9): ADM-32, near bridge 11.32	Site has been fixed two/three times. Guardrail buckling observed. Rock blanket and wheat planted, fix, a year ago; guardrail hanging after slip. Site is not too big; appears to manageable for biostabilization. Fill slope/2H:1V/Slide appears shallow; 30 ft. high/Soil appears good for vegetation
8/18/2004	Adams Co. (Dist. 9): ADM-32-24, Union Hill	Westbound lanes taken out by slide 10 to 15 years ago. Monitoring wells and inclinometers on site. Possible site to introduce biostabilization scheme during construction. Slated for reconstruction of Highway and embankment Spring/Summer 2005 with ODOT "Slip Fund".
8/18/2004	Adams Co. (Dist. 9): ADM-32-5, ¾ miles west of Dever Rd. exit	Series of shallow slips. Appears to be one deep failure. Deep slide

3.3 DEMONSTRATION SITE INVESTIGATION/CHARACTERIZATION

Field exploration (drilling and sampling) and field and laboratory testing was completed for each of the demonstration sites to determine subsurface conditions. Of specific interest was the determination of the groundwater regime (pore pressure/suction and location of the groundwater table), slope movement (location, extent, and amount of sliding), subsurface profile (location and extents of geologic units), and engineering parameters of the site soils.

3.4 SITE INSTRUMENTATION AND MONITORING

Regular monitoring and instrumentation programs have been implemented at the demonstration sites to document: (1) surface erosion and slope movements, (2) pore pressure, soil suction, and soil moisture content, and (3) plant growth and survivability. These measurements and observations are critical to define the baseline site conditions, which are needed for design purposes, and to provide a means for documenting the performance of the various stabilization methods.

3.4.1 Surface Erosion and Movements

Beyond the observations noted during the initial site selection visits, regular visits were made to measure the surface erosion and slope movements at the sites. Erosion activity has been documented primarily through visual observation, field measurements and notes, and digital photography. Inclinometer casing surveys, shallow slope inclinometers, direct measurements of site features, and topographic surveys were used to define the geometry, depth, and extent of the shallow and deep-seated displacements. Details pertaining to surface erosion and movement monitoring at the field demonstration sites are described in their respective sections.

3.4.1.1 Inclinometers Surveys

Inclinometer casings manufactured by Slope Indicator (Mukilteo, Washington) were installed in select boreholes at the demonstration sites and monitored regularly. The inclinometer system is comprised of slotted inclinometer casing, an inclinometer probe and cable, and an inclinometer readout unit. The system permits the measurement of the depth, magnitude and rate of slope movement. This data is used for identifying and quantifying failure surfaces within an unstable slope. Perhaps the two most useful presentations of inclinometer survey data are plots of cumulative and incremental displacement. A cumulative displacement plot is useful for identifying shear movements as it graphs the sum of the incremental displacements from the bottom or reference point of the casing. A graph of cumulative displacement shows how subsurface movement relates to movement at the surface (Slope Indicator 2003). An incremental displacement plot shows displacements at discrete depths where a growing "spike" indicates movement location (Slope Indicator 2003).

The information produced from inclinometer surveys is valuable for determining the geometry of a slide mass or masses and deciding the appropriate type of stability analysis and the design and remediation measures. Correlations between environmental factors like rainfall and slope movement can be made. The slope inclinometer is also used to evaluate the long term performance of an unstable slope. In this scenario, the before and after movements of a landslide that has been mitigated using a bioengineered design can be compared.

3.4.1.2 Shallow Slope Inclinometers

An unconventional approach was specially designed to measure the shallow slips at the project sites. Similar to the inclinometer casings, copper pipes and flexible Tygon® tubing have been installed into shallow holes drilled using a portable gas-powered auger. Near-surface slope movements are periodically measured by surveying the top portion of the tube or pipe sticking above the ground surface. During the life of the installation, the copper pipe or Tygon® tube will bend and subsequently record the slope movement. Although labor intensive, careful exhumation of such installations with hand-tools provides direct measurement of shallow slope movements. Prior to exhumation, the Tygon® tubes are filled with wax or other hardening agent to preserve the deformation of the tubing. Typical installations are limited to depths of about five ft. as it becomes increasingly difficult to auger the holes and exhume the tubes/pipes with increasing depth. These installations provide a means of measuring shallow slope movements without incurring the expense of mobilizing a drill rig and crew. In addition, such installations are possible on side slopes that are inaccessible to a truck-mounted drill rig.

3.4.2 Pore Pressure, Soil Suction, and Soil Moisture Content

Measurements of in situ pore pressure, soil suction, and soil moisture content have been recorded at the demonstration sites from the time of initial selection. These measurements are directly related to soil shear strength, and thus provide valuable information regarding the stability of the subject slopes throughout the year and also before and after the bioengineering installations are completed. Pore pressures are measured with piezometers, soil suctions are measured with tensiometers and gypsum moisture blocks, and soil moisture contents are measured by direct sampling and through calibration with measured soil-moisture characteristic curves. Pore pressure/suction measurements from these instruments can be used directly in stability analysis. For example, small or zero suction measurements justify the use of zero suction in the stability analysis. In addition, this data is important not only for stability analysis, but for defining the groundwater conditions at a site which will undoubtedly influence vegetation survivability.

3.4.2.1 Tensiometers

Jet fill tensiometers manufactured by Soilmoisture Equipment Corp. (Santa Barbara, California) were used for this project to obtain pore water suction measurements at the demonstration sites. The jet fill tensiometer consists of a plastic tube connected to a high air entry ceramic cup which has intimate contact with the surrounding soil at the depth of interest and a bourdon vacuum gage to measures the soil suctions. The installation and operating procedures are outlined by the manufacturer (Soilmoisture Equipment Corp., 1997) and also in Fredlund and Rahardjo (1993).

When using tensiometers, it is important to realize that their ability to function as intended is limited by the mechanical properties of water. That is the suction measurements are limited to approximately -13 psi (-90 KPa) due to the possibility of cavitation. In addition, freezing temperatures will render the instruments inoperable if not ruin them altogether. One additional note; the measured suction must be corrected for the elevation head of the standing column of water between the porous cup and the measuring device or gage. This correction results in a larger negative water pressure being measured by the Bourdon gage than exists in the soil (Fredlund and Rahardjo 1993). Accordingly, positive pressures are reported where the correction for the standing column of water in the tensiometer is greater than the in situ suction.

3.4.2.2 Gypsum Moisture Blocks

Gypsum moisture blocks (G-Blocks) manufactured by the Soilmoisture Equipment Corp. have also been used to monitor the subsurface moisture regime. Using the manufacturer's G- Block relationships (see Figure 3.1), soil suction values have been indirectly measured through correlations to the electronic resistance measured across each G-Block (Soilmoisture Equipment Corp. 2000).

3.4.2.3 Piezometers

In addition to tensiometers and G-Blocks, piezometers have been used to investigate groundwater conditions. These instruments provide a direct measure of the piezometric head of groundwater within a slope.

3.4.3 Plant Growth and Survivability Surveys

Clearly, plant growth provides a measure of the success of each bioengineering installation. Regular visits have been made to evaluate the survivability of the live pole and brushlayer installations at the Muskingum County site. During these visits the condition of the vegetation was inventoried to assess the survival rates for the different species as well as their respective installation technique. The details of the growth and survivability surveys for the Muskingum County site are discussed in section 5.7.

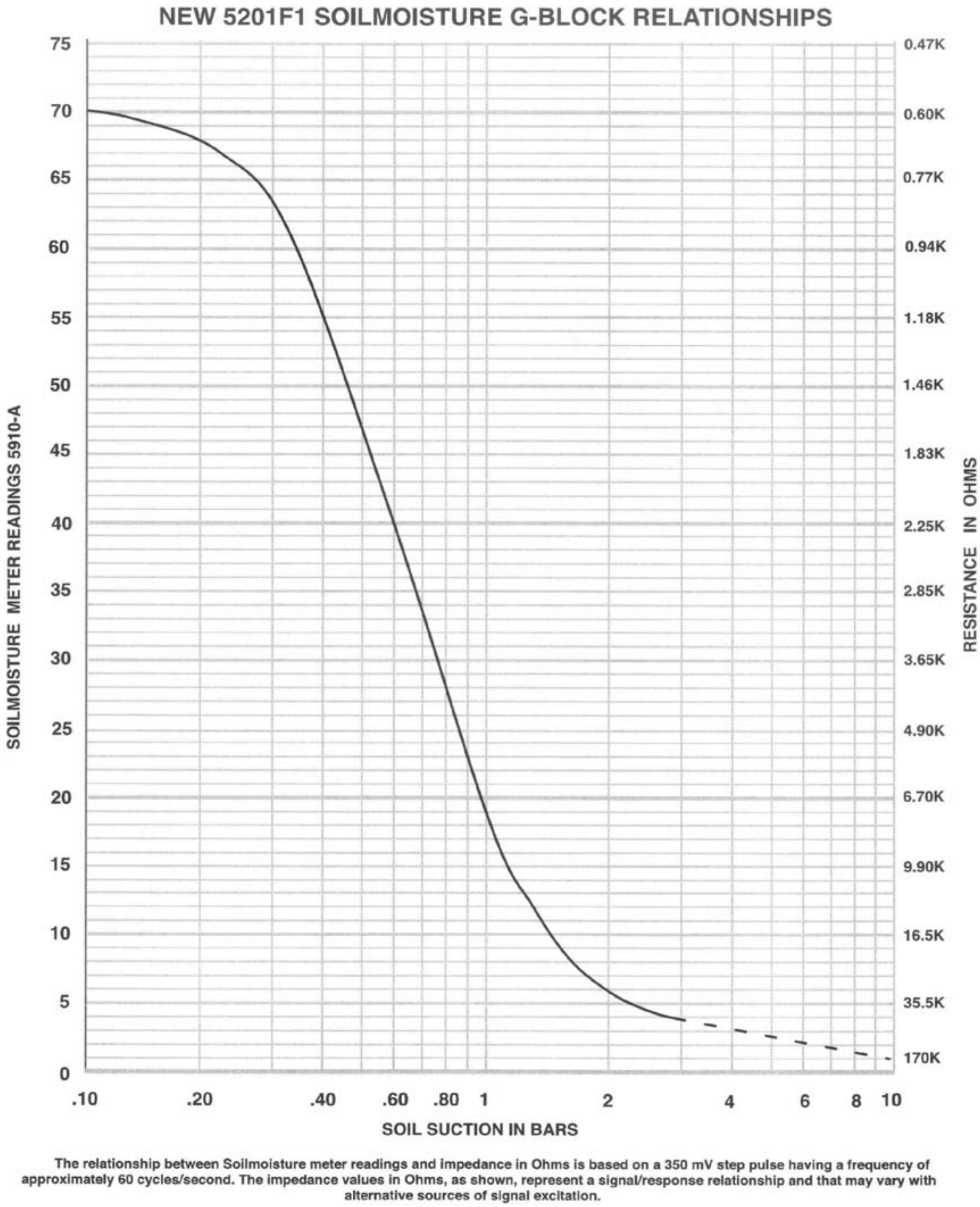


Figure 3.1: G-Block soil suction relationships. (from Soilmoisture Equipment Corp. 2000)

3.5 VEGETATION SOURCES

Unlike most construction projects where inert materials like concrete and steel are generally standard and easily obtained, bioengineered projects rely on site-specific living vegetation which may or may not be locally or readily available. A healthy stock of appropriate vegetation must be secured in order to construct a bioengineered design. Commercial nurseries can provide a reliable source for vegetative materials; however, they typically charge a premium which can be costly. Often willow and other hardwood species can be secured for a project, free of charge, with a little resourcefulness. Local farmers, landowners, business, and public lands are possible sources for suitable vegetation. The clearing of willows, for example, along a ditch line or drainage swale can be mutually beneficial for both parties. On one hand, the vegetative needs of the bioengineering project are fulfilled, and on the other hand a maintenance issue is addressed for a landowner who intended on cutting back the overgrown vegetation. Persistence is the key to successfully securing free vegetation for a bioengineering project. Communication with local wildlife, forestry, and natural resource agents may yield potential sources. Furthermore, vegetation requests for environmentally friendly projects are generally well received.

Vegetation sources for this project were secured by placing numerous phone calls throughout the state to various parties like the Ohio Department of Natural Resources (ODNR), land managers, and foresters. Honda of America, Inc. allowed the harvesting of willow and other hardwood species growing in the drainage channels throughout their complex in Marysville, Ohio. This is a reliable source as Honda has permitted harvesting in the past for similar ODOT projects and they permit up to 25% of the vegetation in their drainage channels to be harvested annually. ODNR has also permitted the harvesting of willows on their wetlands in Delaware County. This source is plentiful and ODNR is readily cooperative in light of the fact that they routinely clear cut much of the willow growth as part of their wetland maintenance. Table 3.2 lists the various vegetation sources that have been investigated for this project.

Table 3.2: Bioengineering vegetation sources.

Source	Contact Information	Comments
Honda of America Mfg., Inc. Marysville, Ohio	Sharon Wagner (937) 644-6644 Sharon_Wagner@ham.honda.com	1+ miles of drainage ditch with extensive growth of willow and poplar. 25% of all material may be harvested annually.
ODNR: Delaware Wildlife Area	Tim Davis (740) 499-3019	Various areas in the wetland have vast willow sources.
18319 Delaware County Line Rd, Ostrander, Ohio 43061	Jim Gates (614) 666-5604 JMGates@Columbus.gov	At least 50 willow poles on private residence.
Farm in Southern Gallia County, Ohio	Buzz Mills, ODOT District Technician	Abundant supply of willows. This source has been used in the past for a willow/stream bank demonstration project.
Ernst Crownvetch Farms, NW Pennsylvania	1-800-873-3321	Can prepare species for bioengineering planting material such as stake/pole, brushlayer, and live facine.
Envirotech Consultants, Inc. 5380 Twp 143 NE, Somerset, Ohio 43783	(740) 743-1669 info@envirotechcon.com www.envirotechcon.com	Nursery specializing in native wetland plants.

3.6 LABORATORY TESTING

Soil samples obtained from the subsurface investigations were transported to either the OSU Soil Mechanics Laboratory or the ODOT Geotechnical Laboratory and examined to confirm or modify field classifications, as well as to evaluate engineering properties. Representative samples were selected for laboratory tests including moisture content, Atterberg limits, consolidation, and triaxial shear tests. The tests were performed in general accordance with standard methods of the American Society for Testing and Materials (ASTM) or other applicable procedures. Soil nutrient testing was conducted by Calmar laboratories of Westerville, Ohio.

3.7 LIVE POLE VERTICAL PULLOUT TESTS

A device was designed to pullout live poles, which permitted the measurement of the force required to pullout or mobilize the pole in the upward or vertical direction. The pullout apparatus consists of a lever with a dynamometer connected to the pole as shown in Figure 3.2. The dynamometer allows the measurement of the force required to pullout or mobilize the pole in the upward or vertical direction.

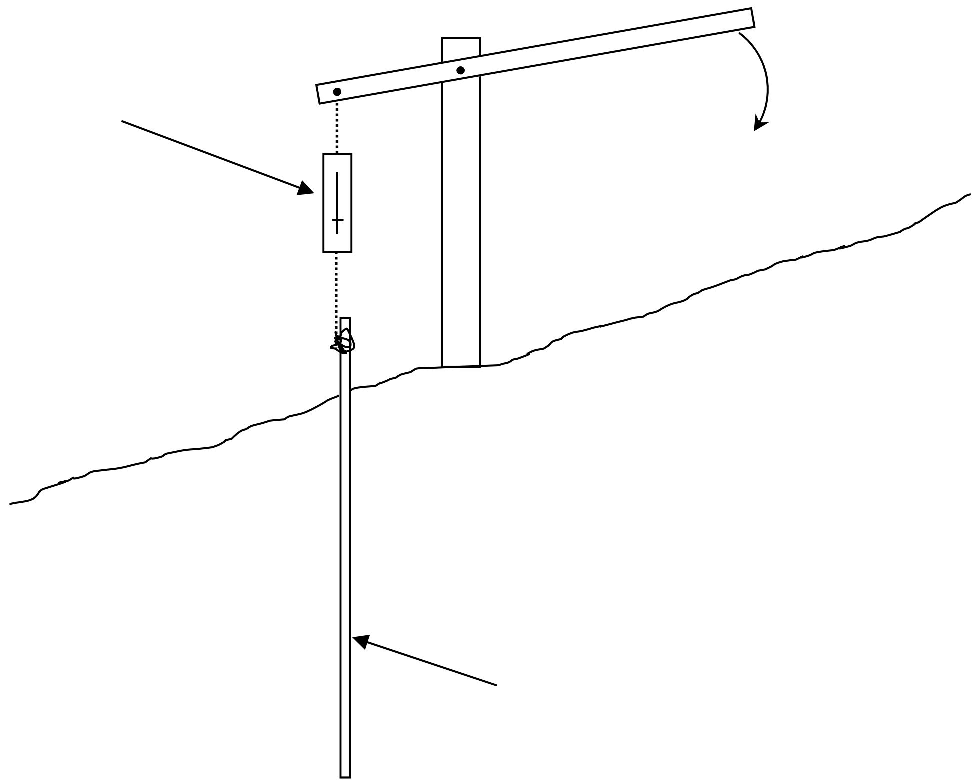


Figure 3.2: Schematic of live pole vertical pullout test.

The upward, vertical, resistance can be used to estimate the undrained shear strength of the soil by applying pile design analysis methods. For example, the α-method (Tomlinson 1971) is commonly used for total stress analysis of piles where the unit side resistance, f_s, is equal to the soil undrained shear strength, s_u, multiplied by α which is an empirical adhesion factor (see equation 3.1).

$$f_s = \alpha\, s_u \tag{3.1}$$

For s_u > 1,500 psf the American Petroleum Institute (API 1984) proposed the relationship:

$$\alpha = 0.5 \tag{3.2}$$

The pullout resistance data can be used to conduct a more rigorous approach to live pole design and may be accomplished by modeling the poles as piles using approaches for latterly loaded piles (Broms 1964; Broms 1965; Rao et al. 1996; Poulos 1999), among other methods. These methods model the mechanics of small diameter piles and the flow effects in cohesive soils.

3.8 STABILITY ANALYSIS, PLANS, AND SPECIFICAITONS

The design recommendations and plans for the Muskingum County and Logan County sites were produced and submitted to ODOT for review and bid process. Two slope stability analyses have been completed as part of the design process for this project – infinite slope and method of slices for circular slip. These methods were chosen because they most closely model the geometry and conditions of the respective sites. Section 3.8.1 and 3.8.2 summarize these analyses. Specific design details and recommendations for the Logan and Muskingum County sites are discussed in Chapters 4 and 5.

3.8.1 Infinite Slope Stability Analysis

The infinite slope stability analysis, presented in section 2.4.1.1, was used to estimate the surface stability of the Logan County slopes and embankment at the Muskingum County site before and after the installation of willow poles. This method was chosen because it was concluded from the site investigation that the slip surface(s) at this site were shallow and approximately parallel to the surface.

For preliminary design, a factored value of 232 psi (1,600 kN/m^2) was used to estimate the ultimate shear stress of live poles as suggested by Steele et al. (2004). By assuming an average live pole diameter of 2 in., the increase in shear strength attributed to the root reinforcement of soil, ("root cohesion"), s_r, can be determined per unit area or live pole grid spacing. Accordingly, s_r equal to 182 psf and 81 psf was used for preliminary design for live pole grid spacing equal to 2 ft. by 2 ft. and 3 ft. by 3 ft., respectively.

3.8.2 Circular Slip Analysis: SLOPE/W

The commercially available slope stability analysis software, SLOPE/W, manufactured by Geo-Slope International (Calgary, Alberta, Canada), was used for preliminary stability analysis of the current (i.e., failure) condition for the Logan County site. Using the Janbu (Janbu et al. 1956; Janbu 1973) and Morgenstern and Price (Morgenstern and Price 1965) circular slip methods, baseline soil strength parameters were determined by back-calculation where failure is imminent (i.e., factor of safety, $F_s \approx 1$). For this analysis, the slope geometry was determined from the survey of the site topography prepared by ODOT, the failure surface(s) was deduced from field observations of scarp and bulge locations and inclinometer data, and piezometer data was used to estimate the groundwater level. Using this data, a model for the site prior to repair was developed to provide soil strength data to be used for the design of the bioengineered remediation of the unstable slope(s). The use of back-calculated soil strength is an accepted practice for geotechnical design and Coduto (1999) states that back-calculated soil strengths are generally very reliable, because they are based on the full shear surface, not on small samples.

3.9 DEMONSTRATION SITE CONSTRUCTION

The bulk of the labor necessary for construction at the Muskingum County demonstration site was completed by crews of OSU undergraduate and graduate students. These efforts consisted of harvesting, transporting, preparing, and installing hardwood cuttings for brushlayer and live pole installation.

CHAPTER 4

DEMONSTRATION SITE: LOGAN COUNTY US-33/SR-347

4.1 PROJECT DESCRIPTION

This field demonstration site is located in Logan County, Ohio, approximately 15 miles west-northwest of Marysville, Ohio. The location is shown in Figures 4.1 and 4.2. The need to mitigate ongoing slope failures, in addition to environmental considerations, makes this site ideal for field demonstration of bioengineering. This site is within an environmentally sensitive ecosystem because it drains into the Darby Creek Watershed and it lies within a prairie restoration zone. In order to address the environmental, landslide, and erosion issues, using both biotechnical and bioengineering techniques, a design has been prepared and construction at this demonstration site was completed during spring 2007. This chapter focuses on the site conditions and research efforts completed prior to construction.

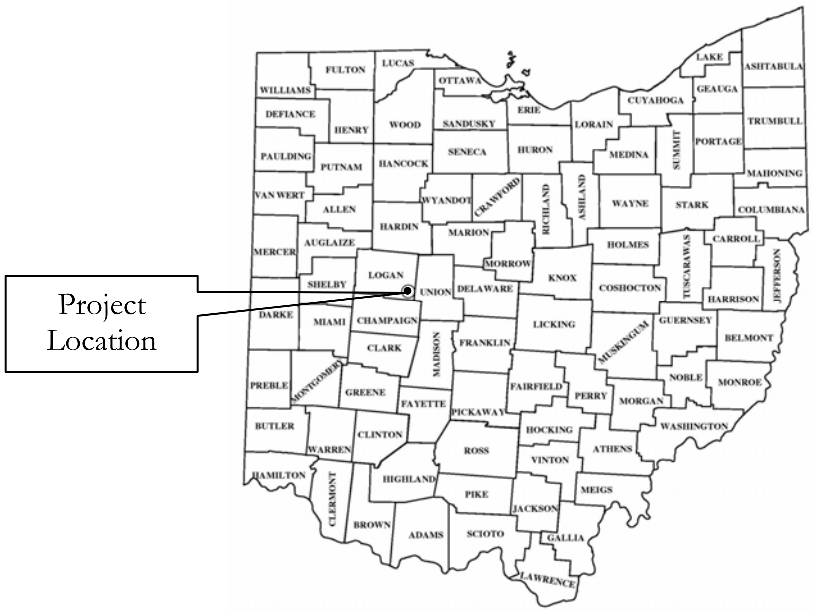


Figure 4.1: Vicinity Map: Logan County US-33/SR-347 field demonstration site.

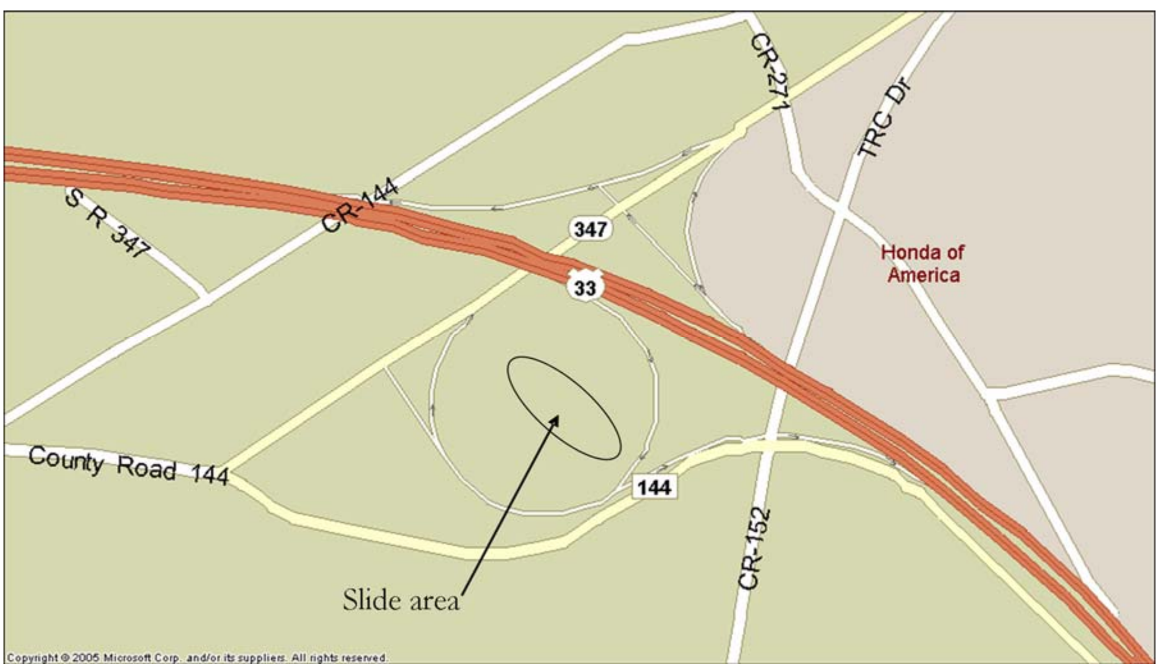


Figure 4.2: Site Plan: Logan County US-33/SR-347 field demonstration site.

4.2 SITE CONDITIONS

4.2.1 General Conditions

Cut slopes form the inclined sides of a 10 to 30 ft. high ⅛ mile long channel sited at the infield of the interchange from U.S. Route 33 (US-33) to State Route 347 (SR-347). The channel sides are approximately 2H:1V and were cut from the native soils. Figure 4.3 shows a picture of the drainage channel. The channel slopes are generally vegetated with tall grasses. However, both sides of the channel are experiencing surficial erosion, as well as shallow translational and deep-seated rotational landslides, which are marked by numerous scarps and bulges. Scarps and erosion are shown on Figures 4.4 through 4.7 for the northeast slope and Figures 4.8 and 4.9 for the southwest slope.

In 2004 the erosion and slide activity at this site became a concern to ODOT. It was suspected that the runoff water collected in this swale contained eroded silt that could potentially impact the ecology of the Darby Watershed. The specific subwatershed at this site is the Ohio EPA subwatershed 1901: Big Darby Creek Headwater to Above Flat Branch Subwatershed. Silt fences were installed by ODOT at this time as a temporary effort to prevent the transport of silt from this site into the Big Darby Creek.

Figure 4.3: Logan County 33/347 drainage channel (facing NE), August 2004.

Figure 4.4: Scarps and erosion on the northeast slope, August 2004.

Figure 4.5: Scarps and erosion on the northeast slope, February 2005.

Figure 4.6: Scarps and erosion on the northeast slope, November 2005.

Figure 4.7: Scarps and erosion on the northeast slope, November 2005.

Figure 4.8: Scarps and erosion on the southwest slope, November 2005.

Figure 4.9: Scarps and erosion on the southwest slope, November 2005.

4.2.2 Regional Geology

The following geologic information for Logan County is reproduced from the *Soil Survey of Logan County* (Waters 1979): "Logan County is geologically complex due to the fact that it has been covered by continental glaciers at least twice throughout history. The most recent geologic deposits are of alluvium in stream valleys and flood plains from eroded upland and terrace soils. Earlier layers are wind blown silt-sized particles of glacial drift which are deposited, immediately after the glacial period, as much as eighteen in. of silty material. Throughout the county exists the present day drainage channels which were formed by the melt water during the glacial retreat. Glacial till enriched with a high percent of limestone and dolomite pebbles and fine material was deposited throughout the county by the movement of glaciers over bedrock".

4.2.3 Climate

Historical climate data for the period between 1971 and 2000 have been collected for Bellefontaine, Ohio, which is approximately 12 miles west-northwest of this site, by the Midwestern Regional Climate Center (MRCC). The average annual rainfall over this thirty year period is 37.42 in. February has typically been the driest month with an average monthly precipitation of just 2.02 in. and June is on average the wettest month producing 4.11 in. of rain. The average maximum, minimum, and mean annual temperatures are 59.7°F, 40.0°F, and 49.9°F, respectively, with January being the coldest (23.8°F mean avg.) and July being the warmest (72.7°F mean avg.) months on average. Over 29 in. of rainfall was recorded in the winter and spring months of November 2004 through April 2005 in Bellefontaine. Historically (1971 through 2000), on average 16.5 in. was recorded for the same time period. Nearly 11 in. of precipitation was recorded for November 2005 compared to the historical average of 2.28 in. Temperature, rainfall, and growing season tables are included in Appendix A.

It is also worth noting that the two slopes should be expected to have differing microclimates, as one is predominantly north-facing while the other predominantly south-facing. The south-facing slope will receive abundant direct sunlight and can be expected to be increasingly susceptible to drought with the converse to be expected for the north-facing slope.

4.2.4 Subsurface Conditions

The site specific soil profile has been inferred from the subsurface investigation. In general, the profile consists of a top layer of brown glacial till. Underneath is gray to brown sandy silty clay deposits. Thin layers, less than eight in., of gray clay sometimes grading into gray clay with small stones are present between the till and silty clay layers. Figure 4.10 shows a typical subsurface profile at the site. Figure 4.11 shows the plan view of the site including the boring and instrument locations.

4.2.5 Groundwater Conditions

The piezometer data shows several trends in the groundwater conditions at this site. It has been observed that at the mid-slope (piezometers P-9, P-11, P-13, and P-15) on both sides of the channel, high piezometric levels, approximately at the level of the ground surface, have been measured at the depth of seven and nine ft. below the surface. Therefore there is at times as much as eight ft. of pressure head on the slip surface at approximately mid-slope. The piezometric levels at mid-slope of the NE slope (piezometers P-9 and P-11) are slightly less than their counterparts of the south slope (P-13 and P-15). Piezometer locations are shown on Figures 4.10 and 4.11 and the data is plotted in Figures 4.13 through 4.18.

It is observed from tensiometers data that the soil suction in the upper 7 ft. of the slopes is generally less than -5 psi. The low soil suctions indicate that during the non-winter months, significant negative pore water pressures are not present. It is likely that during the winter months, although not directly measured, negative pore water pressures are not developed because the slopes are generally saturated and positive pore pressures are recorded during wet seasons. Tensiometer locations are shown on Figures 4.10 and 4.11 and the data is presented in Table 4.4 and Figures 4.19 through 4.22.

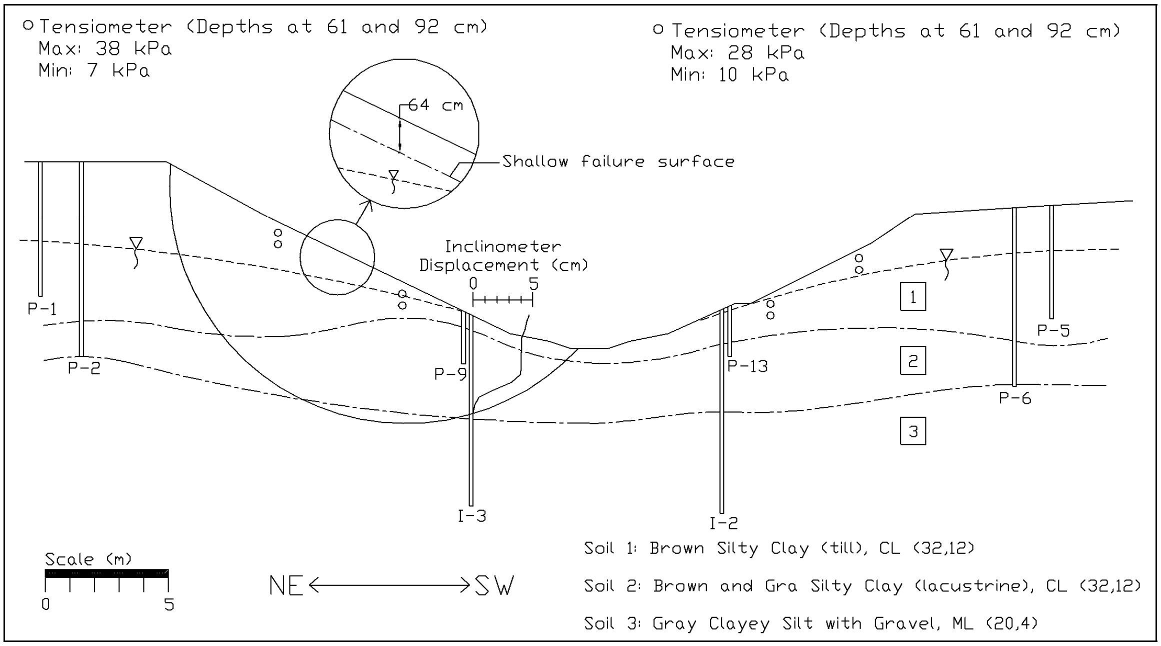


Figure 4.10: Typical subsurface profile for the Logan County site (Wu et al. 2008).

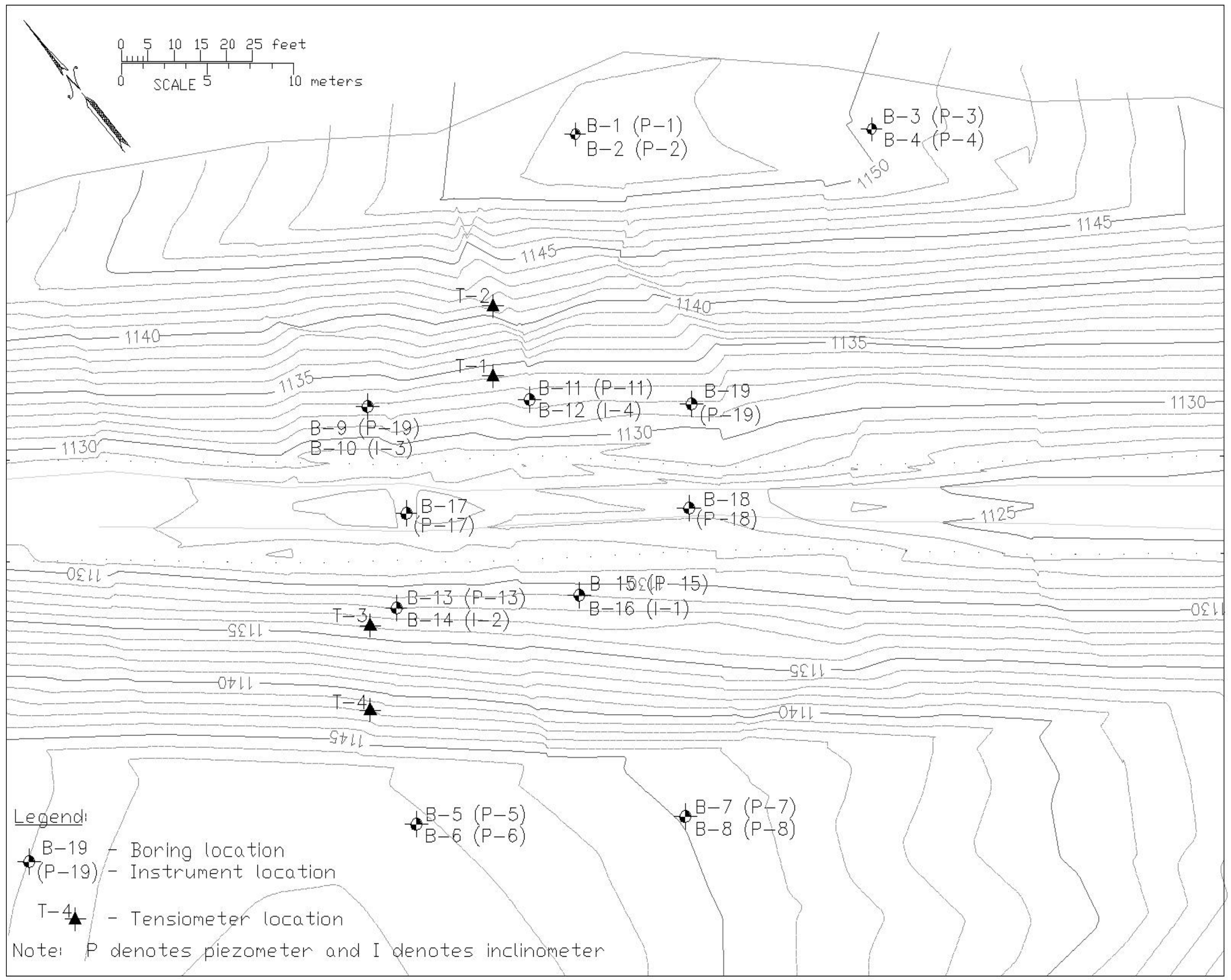


Figure 4.11: Site plan showing topography, boring, and instrumentation locations at the Logan County site.

4.3 FIELD EXPLORATION AND LABORATORY TESTING

4.3.1 Field Exploration

Nineteen soil borings, designated as B-1 through B-19, were drilled under the supervision of ODOT at the locations indicated on the typical profile, Figure 4.10, and site plan, Figure 4.11. Borings B-1 through B-8 were advanced using a truck mounted drill rig owned and operated by ODOT. Borings B-9 through B-19 were advanced using a limited access drill rig owned and operated by FMSM Engineers. The borings were drilled using hollow stem augers on October 25th through 28th, 2005. Standard penetration tests (SPT) were conducted using a 140-pound hammer falling 30 in. to drive a 2-in. O.D. split barrel sampler for 18 in.

Soil samples obtained from the SPTs were visually classified in the field, preserved in plastic tubs, and classified at the OSU and ODOT laboratories. Relatively undisturbed soil samples were obtained using Shelby tube samplers and preserved with paraffin/petroleum jelly seals. Boring logs and piezometer installation logs prepared by ODOT for borings B-1 through B-8 are included in Appendix A (logs were not prepared for borings B-9 through B-19 completed by FMSM).

4.3.2 Laboratory Testing

4.3.2.1 Physical properties

Laboratory tests to determine the engineering properties of the soil samples from the site included triaxial shear tests, consolidation tests, and classification tests. Triaxial shear tests performed at the OSU Soil Mechanics Laboratory on relatively undisturbed specimens included four multistage consolidated-undrained (CU), and two multistage unsaturated consolidated-drained (CD) tests. Three consolidation tests were also performed. Classification, Atterberg limits, sieve analysis and moisture content determinations were performed by the ODOT Geotechnical Laboratory and the results are presented in Appendix A.

Based on the consolidation test results, the preconsolidation pressure, P_c, ranged from about 1,450 to 3,500 psf (70 to 168 kPa) following the Casagrande method (Casagrande 1936). Accordingly, the overconsolidation ratio (OCR), defined as the ratio between the preconsolidation pressure and existing effective overburden pressure, $OCR = P_c / \sigma_v'$, ranges between 2.4 and 9.2. The range in calculated OCR fluctuates with the seasonal variation of the groundwater levels as measured in the piezometers, in other words, as the piezometric level rises, the effective overburden, σ_v', decreases, resulting in a higher calculated OCR. Table 4.1 summarizes the results from the consolidation tests including the moisture content, *w*, compression indices C_c and C_r, and the calculated P_c and OCR values. Calculations for the C_c, C_r, P_c and OCR are included in Appendix A.

Table 4.1: Results of consolidation testing for the Logan County site.

Boring	Depth H (ft.)	Soil Type	Water Content w (%)	Compression Index C_c	Recompression Index C_r	Pre-consolidation Pressure P_c (psf)	Over-consolidation Ratio OCR
B-16	6	Brown/gray clay w/ trace sand	18	0.104	0.028	1,450	2.4 to 4.2
B-10	6	Brown/gray Clay	20	0.114	0.032	3,500	4.8 to 9.2
B-10	6	Brown/gray Clay	17	0.176	0.042	3,030	4.0 to 7.2

Table 4.2 presents the shear strength parameters c' and ϕ' determined from the CU triaxial testing. Test reports have been included in Appendix A and the location of the boreholes are shown on Figure 4.11. The CU shear strength parameters are in general agreement with the values determined from the slope stability back analysis (section 4.5.1.2) which are also presented in Table 4.2.

Table 4.2: Results of shear strength parameters from CU triaxial testing for the Logan County site.

Soil Type	c' (psf)	ϕ' (degrees)
Brown Clay w/ stones <TILL>; w = 16%	560	24.9
Gray Clay; w = 23%	0	33.0
All CU data points	0	32.2
Back Calculated (section 4.5.1.2)	50	30

An extended Mohr-Coulomb failure envelope has been used to interpret the results of the multistage CD tests conducted on unsaturated soil samples. The extended failure envelope differs from the traditional Mohr-Coulomb failure envelope for saturated soils in that a third axis is introduced where the increase in shear strength resulting from increased matric suction is plotted. The result is a planar failure surface defined by the angles ϕ' and ϕ^b which characterize the increase in shear strength due to increase in net normal stress and matric suction, respectively, and their respective intercepts c'. Figure 4.12 shows the extended Mohr-Coulomb failure envelope.

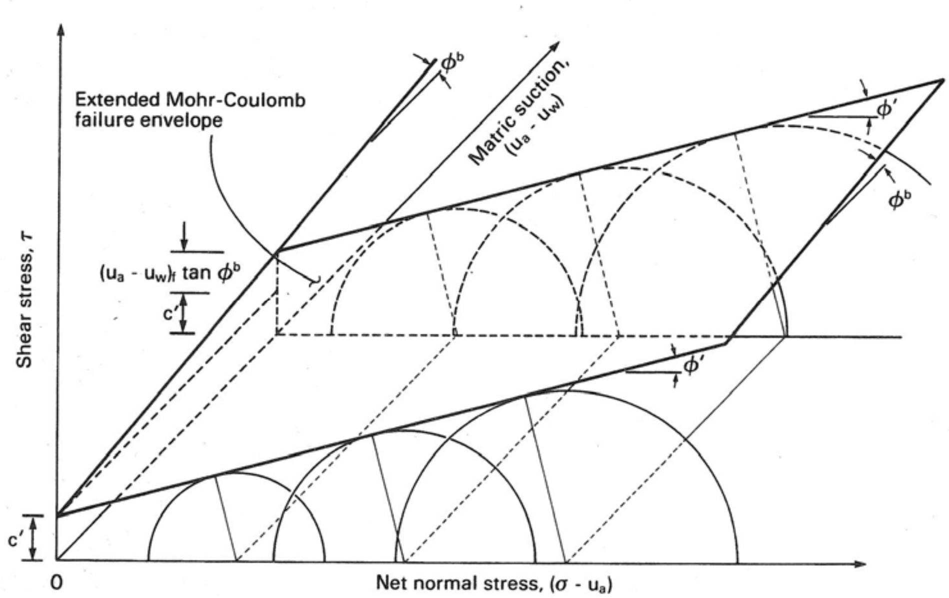


Figure 4.12: Extended Mohr-Coulomb failure envelope for unsaturated soils. (after Fredlund and Rahardjo 1993)

When multiple CD test are conducted on like unsaturated soil samples, ϕ^b can easily be determined by plotting the Mohr circles as shown in 4.12 (ϕ^b is the angle of inclination of the failure plane along the shear stress, τ, versus matric suction, $(u_a - u_w)$ axis). Due to the lack of multiple tests conducted at the same matric suction, $(u_a - u_w)$, ϕ^b was estimated theoretically by employing several assumptions. First, it is assumed that the shear strength parameters determined from the CU tests on like saturated samples (i.e., effective friction angle, ϕ', and effective cohesion, c') are the same for the unsaturated case. Second, the extended Mohr-Coulomb failure envelope is planar rather than curved (i.e., nonlinearity in the shear strength versus matric suction has been neglected). Lastly, the soil sample has not undergone excessive deformation during the multi stage test, which could result in the measured shear strength decreasing with successive stages toward residual conditions. Table 4.3 presents the results of the multistage unsaturated CD test. Calculations for determining ϕ^b along with the CD test reports are included in Appendix A.

Table 4.3: Results of shear strength parameters from CD triaxial testing for the Logan County site.

Soil Type	c' (psf)	ϕ' (degrees)	ϕ^b (degrees)
All CD data points (average values)	0	32.2	16.7

4.3.2.2 Chemical properties

Soil nutrient testing was performed by Calmar, Inc., in Westerville, Ohio, on a composite soil specimen recovered from several locations across the upper two ft. of soil from the landslide areas. The soil nutrient report for this site can be found in Appendix A. Based on the test results provided by Calmar, it has been determined that the pH at this site is higher than optimum and the nitrogen, phosphorus, potassium, and organic matter levels are lower than optimum for the growth of hardwood trees and shrubs and bushes. As per Calmar's recommendation, 2.8, 3.4, and 4.6 pounds per 1,000 ft^2 of nitrogen, phosphorus, and potassium, respectively, should be added to the soil annually. Additionally, 3, 5, 8, and 11 pounds per 1,000 ft^2 of sulfur should be added seasonally for soil depths of 3, 6, 9, and 12 in., respectively, to achieve ideal nutrient conditions for hardwood trees and shrubs and bushes at this site.

4.4 SITE INSTRUMENTATION

As part of the ongoing site investigation, groundwater and slope stability conditions have been regularly monitored. Tensiometers and piezometers have provided insight into the groundwater conditions. Shallow and deep inclinometers have also been used to measure slope movements. Figure 4.11 shows the location and type of instruments that have been installed at this site.

4.4.1 Piezometers

Twelve piezometers were installed at this site during the subsurface investigation. These piezometer installations have been used to measure the groundwater conditions at this site over time. Figures 4.13 through 4.18 show the piezometric levels at each piezometer versus time since they were installed up until February 1, 2006. Each piezometer has been given a designation of the letter "P" for piezometer followed by the borehole number in which it was installed. For example, P-9 indicates the piezometer installed in borehole number 9. The piezometer data is discussed in section 4.2.5, Groundwater Conditions.

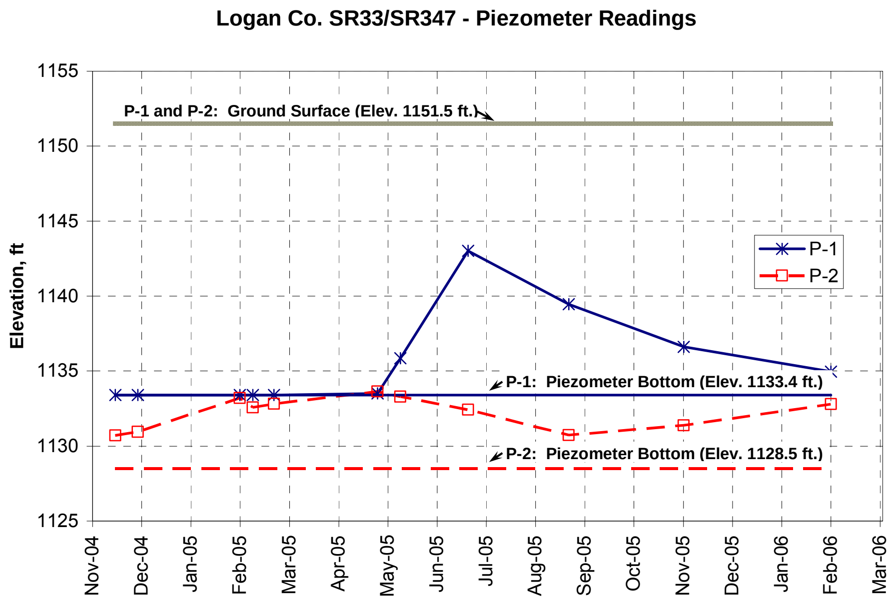


Figure 4.13: Piezometer data for P-1 and P-2.

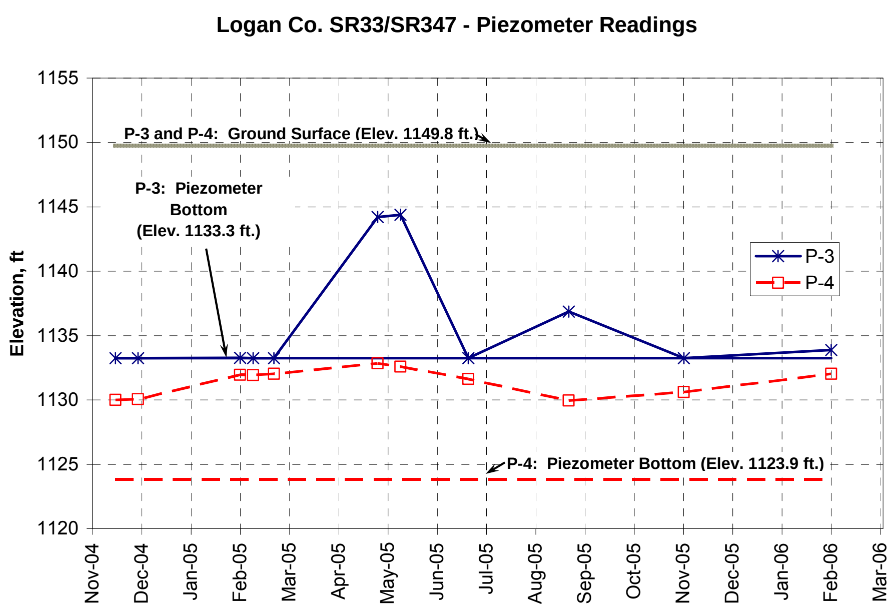


Figure 4.14: Piezometer data for P-3 and P-4.

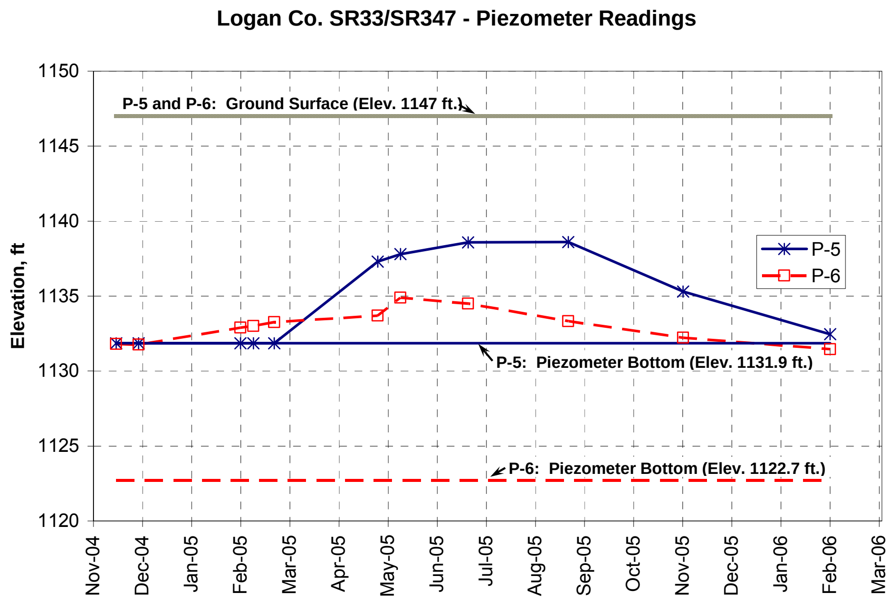


Figure 4.15: Piezometer data for P-5 and P-6.

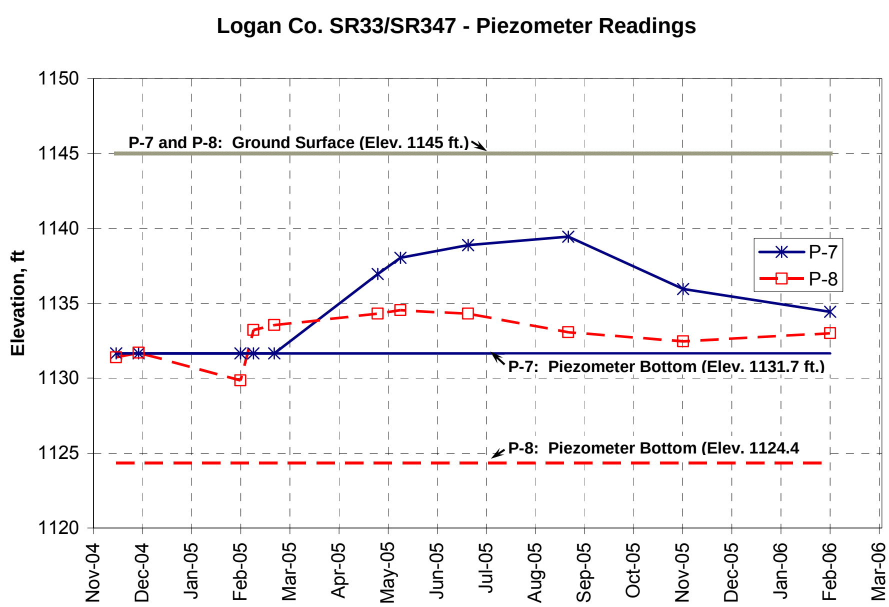


Figure 4.16: Piezometer data for P-7 and P-8.

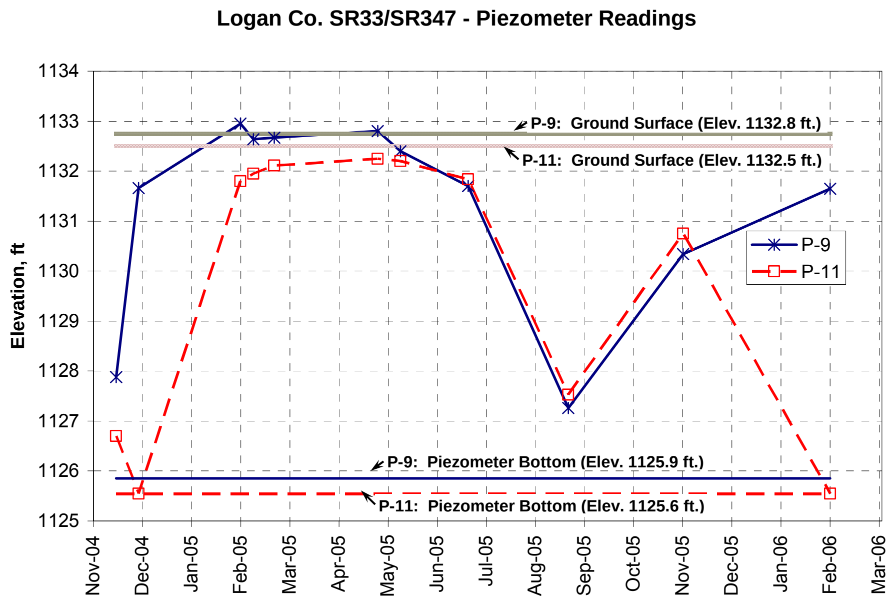


Figure 4.17: Piezometer data for P-9 and P-11.

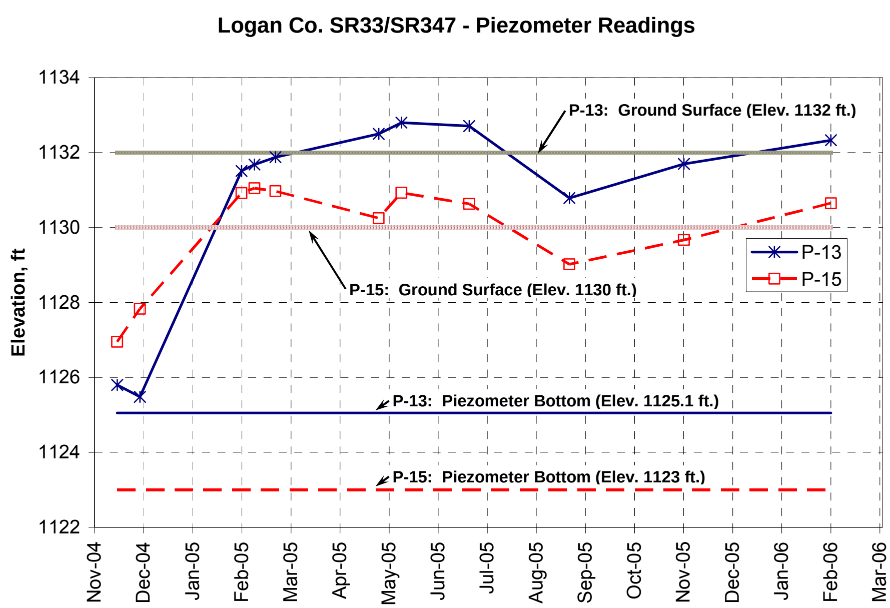


Figure 4.18: Piezometer data for P-13 and P-15.

4.4.2 Tensiometers

Jet fill tensiometers were used to measure the soil suction or negative pore water pressure in the upper six ft. of the unstable slope. In total, 11 tensiometers, two 2-ft., four 3-ft., four 5-ft., and one 7-ft. deep, were installed and monitored between August and November 2005. Their locations are shown in Figure 4.11 and are designated with the letter "T". Unfortunately, tensiometers data is not available for the winter season, which had the largest landslide activity as freezing temperatures ruin these instruments. The tensiometers have been read and serviced on a regular basis between June and November 2005 and the collected data is presented in Table 4.4 and Figures 19 through 22. Positive pressures are reported where the correction for the standing column of water in the tensiometer is greater than the in situ suction. The tensiometer data is discussed in section 4.2.5, Groundwater Conditions.

Table 4.4: Tensiometer data for Logan County site.

	Pore Water Pressure in Soil* [psi] (negative = suction)										
Tensiometer →	T-1				T-2		T-3			T-4	
Depth (ft.) →	2	3	5	7	3	5	3	5	7	3	5
Reading date and time											
6/22/05	-1.5	-0.7	0.5	--	-1.4	--	-1.2	--	--	-2.4	-1.0
8/23/05	-10.4	-9.4	-1.8	--	-1.2	-0.8	-0.4	0.5	--	-0.7	-0.1
9/14/05	-6.3	-0.4	0.8	--	-1.2	-0.1	-0.5	0.8	--	-0.1	-0.1
9/15/05	-10.1	-0.7	0.1	--	-2.1	-0.1	-0.7	0.6	--	-0.9	-0.1
9/16/05	-0.8	1.1	-0.2	--	0.2	1.4	0.2	0.8	--	0.2	1.1
10/12/05	-0.5	0.9	1.7	--	-0.4	0.8	0.2	1.1	--	-0.1	0.5
11/1/05	0.1	0.8	--	2.2	-0.1	1.1	0.2	0.9	2.4	-0.1	0.2
11/2/05	-0.1	0.5	1.2	1.9	-0.4	0.5	-0.1	0.8	1.7	-0.2	0.2
11/3/05	0.1	0.5	1.2	2.1	-0.2	0.8	-0.1	0.8	1.7	-0.1	0.2
11/10/05	0.4	1.1	1.8	2.4	-0.1	0.9	0.8	1.5	2.1	0.1	0.8
Average	**-2.9**	**-1.3**	**0.5**	**2.2**	**-0.8**	**0.5**	**-0.2**	**0.9**	**1.9**	**-0.5**	**0.1**

*corrected for elevation corresponding to the water column in the tensiometer

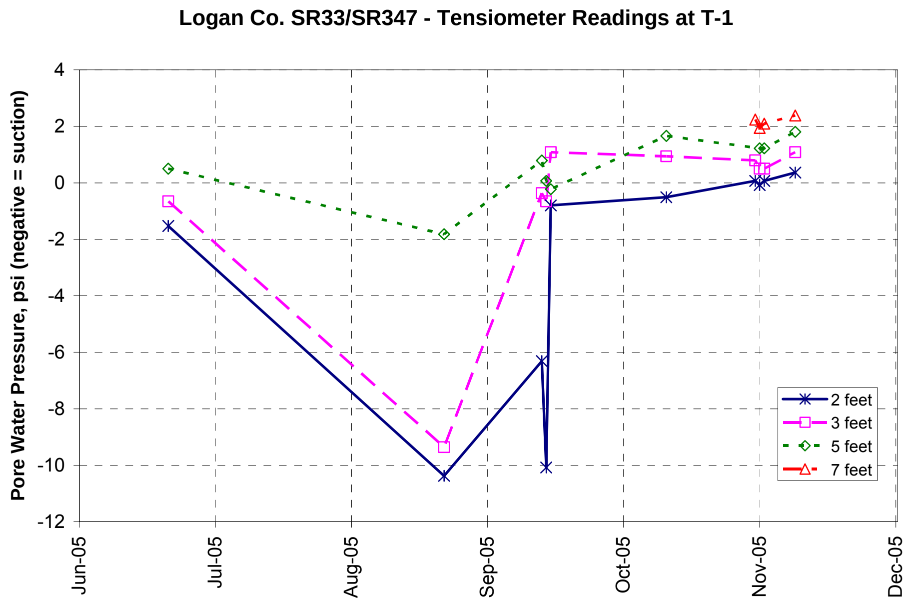


Figure 4.19: Tensiometer data for T-1.

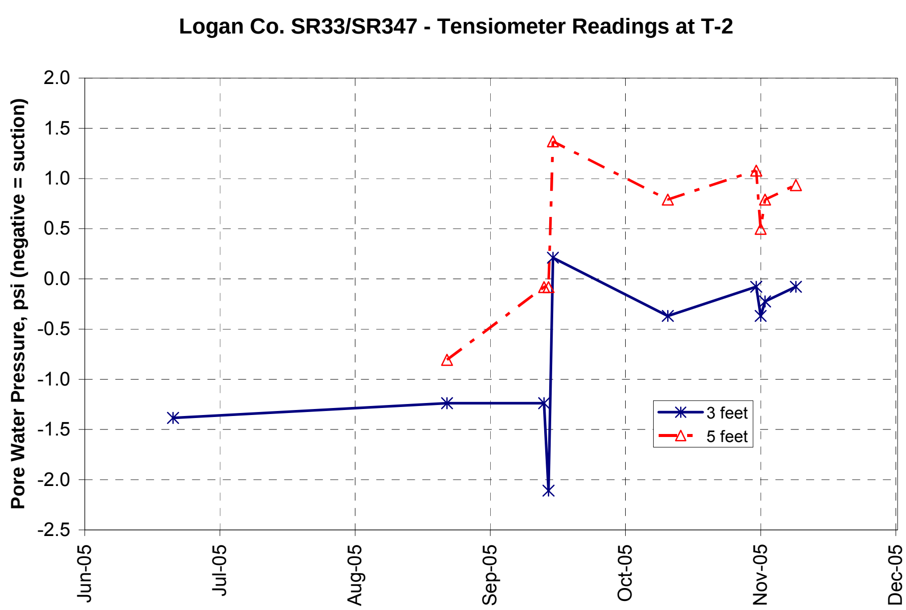


Figure 4.20: Tensiometer data for T-2.

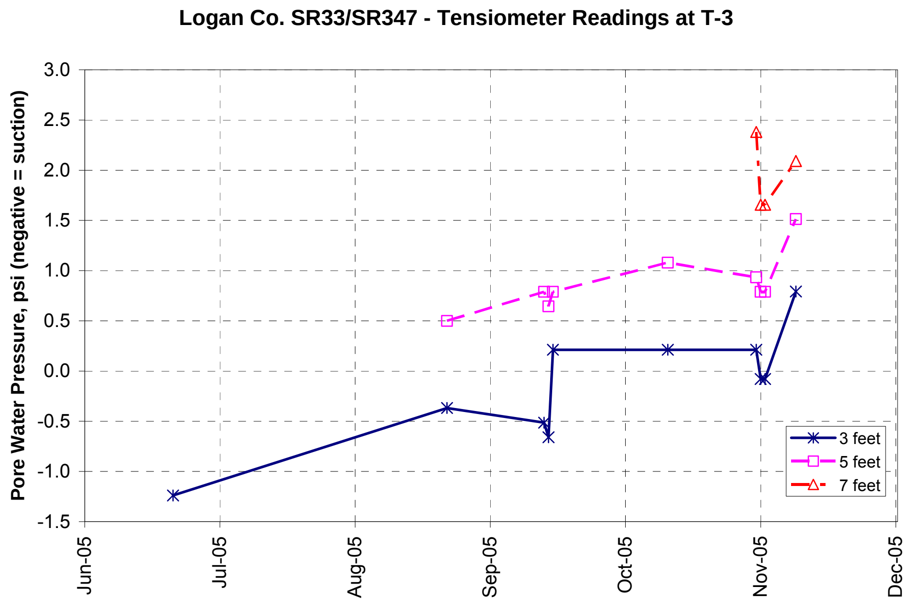


Figure 4.21: Tensiometer data for T-3.

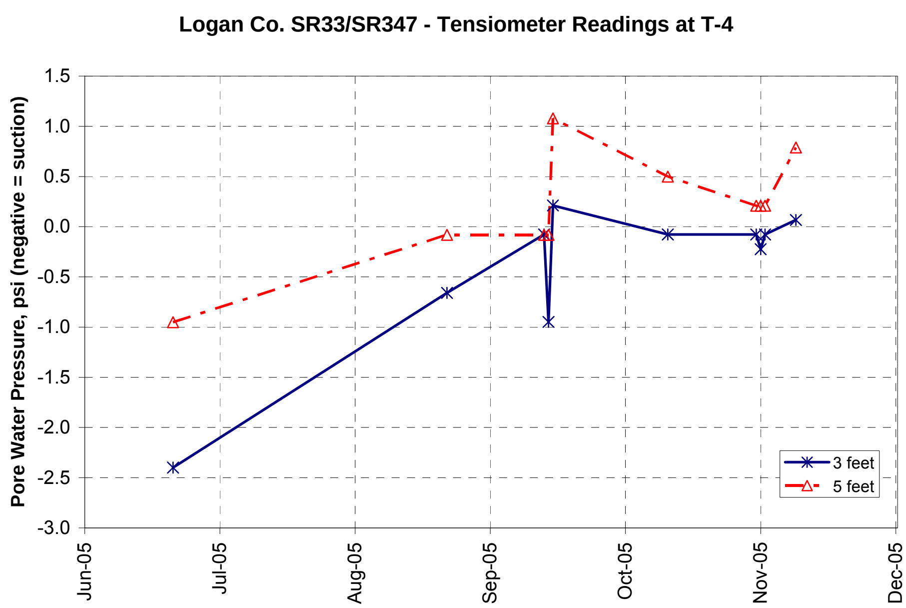


Figure 4.22: Tensiometer data for T-4.

4.4.3 Inclinometers

Four 30 ft. deep inclinometer casings designated I-1, I-2, I-3, and I-4 were installed in boreholes B-16, B-14, B-10, and B-12, respectively. The casings have been surveyed using a vertical probe to measure the extent and depth of the slope movements. Cumulative and incremental time history plots of the down slope movements are presented in Figures 4.23 through 4.26. Only two data sets were measured for the I-1 inclinometer prior to slope movements that distorted the casing sufficiently that the survey probe would no longer fit down the casing.

Slope indicator surveys show that the failure surface extends to a depth of approximately 5 to 10 ft. below the ground surface at mid-slope. The most significant slope movements were recorded during the winter and spring of December 2004 through April 2005 and were likely triggered by the winter and spring precipitation being well above normal. The largest cumulative movement of approximately 5½ in. was recorded at I-2 (northwest slope at mid-slope) for the period between December 2004 and January 2006.

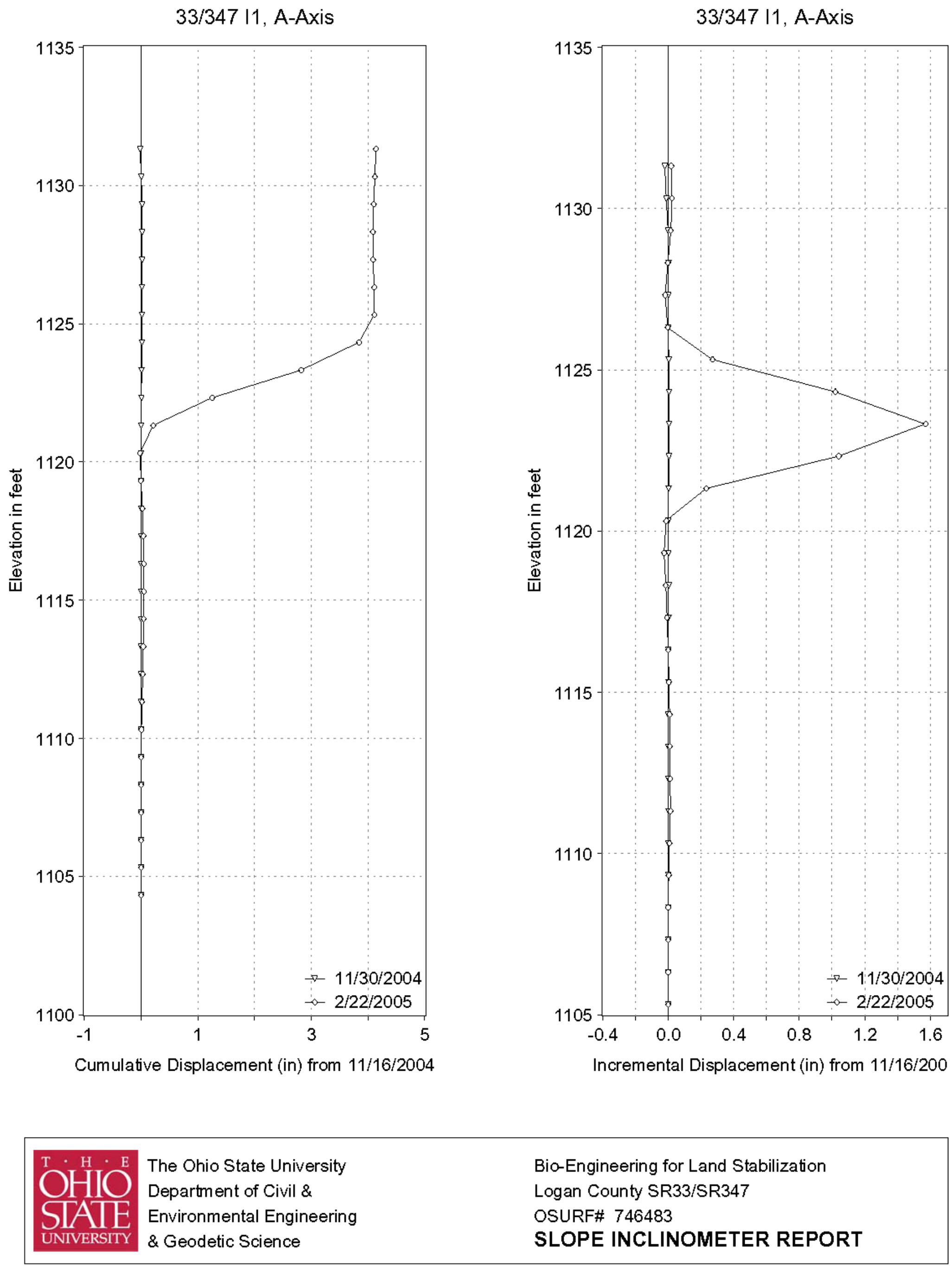


Figure 4.23: Cumulative and incremental down slope displacements at I-1.

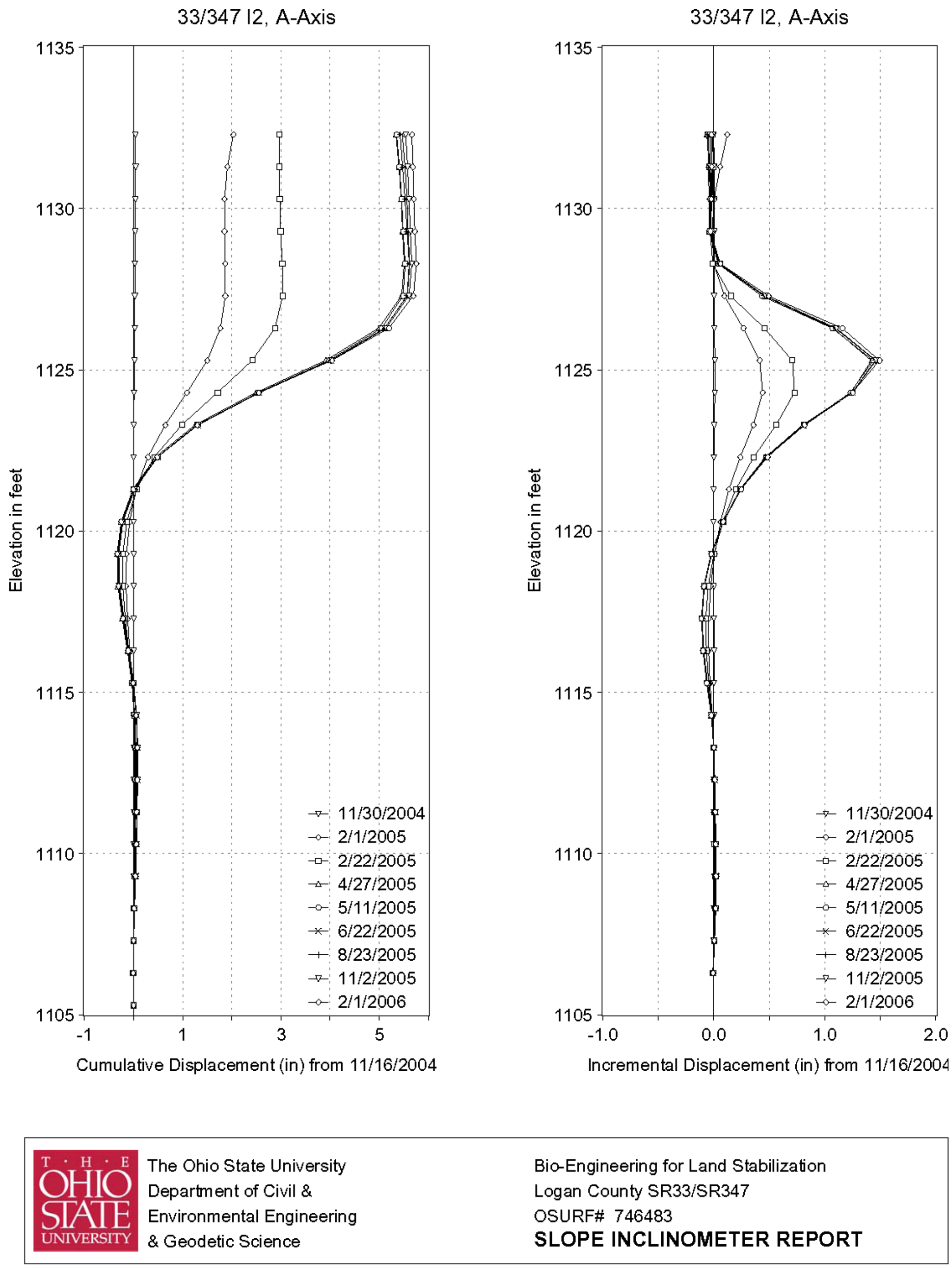


Figure 4.24: Cumulative and incremental down slope displacements at I-2.

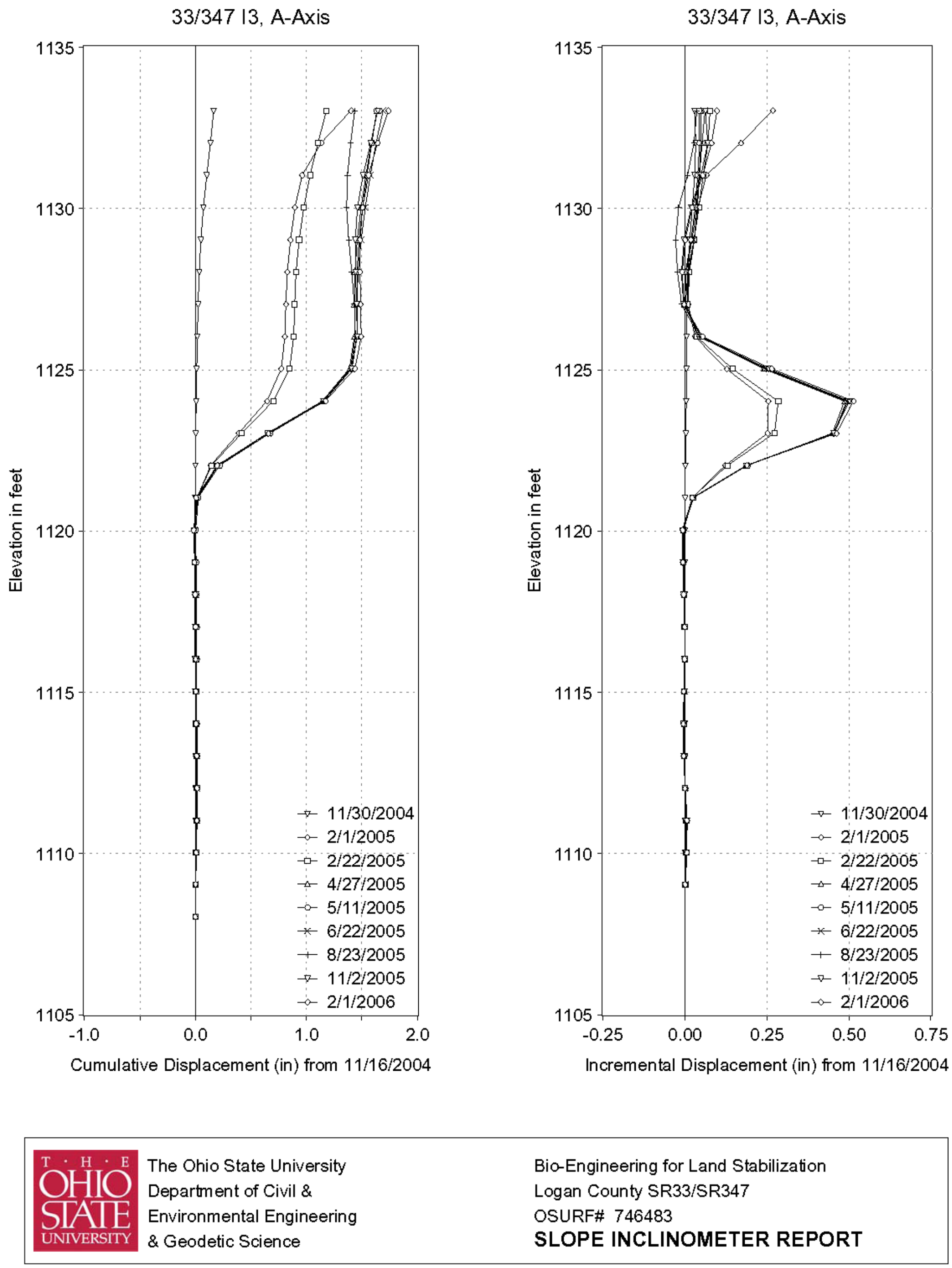


Figure 4.25: Cumulative and incremental down slope displacements at I-3.

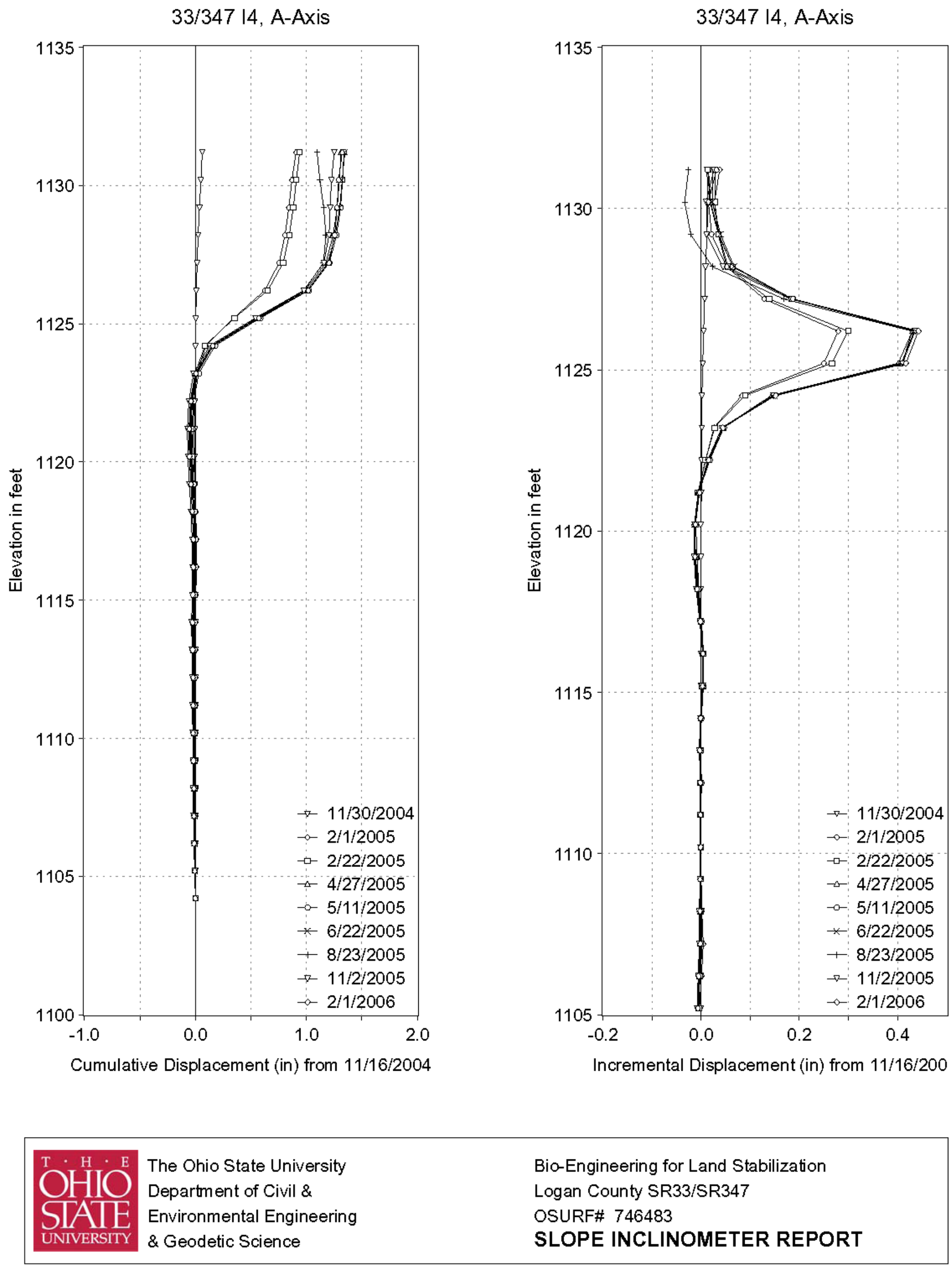


Figure 4.26: Cumulative and incremental down slope displacements at I-4.

4.5 SOIL BIOENGINEERING DESIGN

Various approaches were considered for stabilizing the slopes at the Logan County US-33/SR-347 site. Regardless of the chosen method, the shallow and deep slides would have to be mitigated and surface erosion would have to be addressed to prevent the formation of gullies and cracks which would likely increase infiltration.

Bioengineering methods (e.g., brushlayers and live poles) can be used to stabilize the surface layers and control erosion. Trees with deeper roots can further reduce moisture and increase strength, which would help stabilize the deeper failures. However, the deep failure surface was judged too deep to fix using biostabilization methods alone. To stabilize the deep failure surfaces, other options such as slope flattening, toe berm construction, drains, and lime injection were explored. The lime injection approach was dismissed because it was unfamiliar to the research team and it was thought that the determination of the strength gain would be difficult and perhaps expensive. Although slope dewatering is often an effective and economical approach, drain installation was dismissed as a potential remediation measure as the potential to lower the groundwater level is limited because the groundwater table at the site is only slightly above the level of the ditch.

Among the bioengineering options, willow poles were selected to address the shallow landslides. Slope flattening along the top of the west slope and the construction of a berm at the toe of the east slope were chosen to arrest the deep-seated failures, and hydro seeding in conjunction with erosion mats was chosen to address erosion. In order to evaluate the effectiveness of bioengineering installations, several soil-bioengineering configurations, as well as a control section were included in the construction plan.

4.5.1 Slope Stability

4.5.1.1 Assessment

The most significant slope movements at the Logan County US-33/SR-347 site were observed during the winter and spring months of December 2004 through April 2005. During this period, typical mid-slope movements of 3 to 5 in. were measured along the deep failure surfaces. Deep failure surfaces on both slopes were identified at approximately 5 to 10 ft. deep at mid-slope by slope inclinometer surveys. It is speculated that during this period the slopes experienced the most severe loading because winter and spring precipitation was well above normal. This suggests that the slopes would be subjected to the maximum pore pressure and weight at that time. The isolated large movements indicate that the factor of safety against sliding along the critical surface was approximately equal to unity, or $F_s \approx 1$, during this "critical" period. ODOT observed that no large movements had occurred since the initial failure in August 1997 when eight in. of rain fell. Hence, the factor of safety against sliding for the slopes at this site is typically slightly greater than 1.0 except when landslide events are triggered by excessively heavy precipitation.

In addition to deep-seated rotational landslides, numerous shallow translational slips, marked by numerous scarps and bulges, have been identified. Movement of one such shallow slide located near the near the bottom third of the NE slope was directly measured using a shallow slope inclinometer. The exhumed slope inclinometer recorded approximately 3 in. of movement at the surface and the depth of the sliding mass extending about 2 ft. below the ground surface (personal communication with Brian Trenner).

4.5.1.2 Slope Stability analysis using SLOPE/W

The slope stability analysis software, SLOPE/W, produced by Geo-Slope International (Calgary, Alberta, Canada), was used for slope stability analysis of the current condition for the Logan County site. Using the Janbu (Janbu et al. 1956; Janbu 1973) and Morgenstern and Price (Morgenstern and Price 1965) circular slip methods, baseline soil strength parameters were determined by back-calculating for when failure is imminent (factor of safety, $F_s \approx 1$). For this analysis; the slope geometry was determined from the topographic survey performed by ODOT, the failure surfaces was deduced from field observations of scarp and bulge locations and inclinometer data, and the groundwater conditions estimated from piezometer data. Using the site information and data mentioned above, the model for the site prior to repair was developed to back-calculate soil shear strength parameters to be used for the design of the bioengineered remediation for the unstable slopes at this site. Table 4.5 presents the safety factors that were calculated using the back-calculated shear strength parameters. Printouts of the slope stability analyses are included in Appendix A.

The back-calculated shear strength parameters for global failure surfaces similar in geometry to the observed failures under drained loading are:

Till: c' =50 psf; ϕ' =30° (γ =120 pcf)

Clay: c' =50 psf; ϕ' =30° (γ =120 pcf)

The back-calculated shear strength parameters for global failure surfaces similar in geometry to the observed failures under undrained loading are:

Till: s_u= 500 psf (ϕ_u = 0; γ = 120 pcf)

Clay: s_u = 200 psf (ϕ_u = 0; γ = 120 pcf)

Based on site observations, scarp faces are near vertical or θ = 90°. The F_s calculated based on cohesionless soil strength is much less than unity for these local steep zones; thus, it can be concluded that the soil is in fact cohesive to some degree for our analyses based on our field observations; however, this cohesion may actually be apparent cohesion from suctions in soil. The laboratory data (see Table 4.2) further supports this, being that all of the soil samples contained a significant amount of fine-grained particles. However, near surface deterioration from weathering in stiff or recompacted clay soils in Ohio have been shown to approach the strength state of c' = 0 and ϕ' > 0 (Wu et al. 1993).

Table 4.5: Logan Co. slope stability results using back-calculated shear strength parameters.

Slope	Analysis	Factor of Safety	
		Undrained	Drained
North-East	Morgenstern-Price	1.3	**1.0**
	Janbu	1.3	**0.9**
North-East With Berm	Morgenstern-Price	1.4	1.2
	Janbu	1.3	1.0
South-West	Morgenstern-Price	**0.9**	1.3
	Janbu	**1.0**	1.4

Baseline F_s used to determine shear strength parameters shown boldface.

Subsequent to the back-calculated stability analyses, the soil strength parameters were used to evaluate the slope stability for remediation involving toe-berm construction and slope flatting near the top. By excavating approximately 2 ft. of material near the top of the southwest slope and using this material to construct a three ft. toe berm on the northeast slope the driving moment would be reduced for southwest slope and the resisting moment would be increased for the northeast slope as shown in Figure 4.27. This translates to a factor of safety against sliding increase from $F_s \approx 1$, imminent failure, to $F_s \approx 1.2$ for the worst case loading (i.e., drained conditions for the northeast slope).

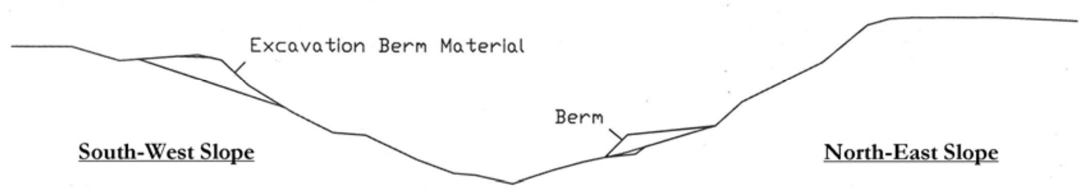


Figure 4.27: Cross-section schematic of slope excavation/berm construction at Logan County site.

4.5.1.3 Slope Stability Analysis – Infinite Slope

The infinite slope stability analysis approach was used to analyze the shallow slips at the site. The analysis followed the infinite slope procedure and formulas presented in Taylor (1948) and Gray and Sotir (1996) which were discussed in section 2.4.1.1 and Figure 2.6. For this analysis, the slope and failure surface geometries were determined from observations made during field reconnaissance of the site. In addition, site observations and piezometer data was used to identify the groundwater conditions at this site that indicated seepage may act parallel or vertical to the slope. Average soil strength parameters from the laboratory CU triaxial tests (see Table 4.2) and back-calculated values from the global slope stability analysis were used for the analysis (see section 4.5.1.2). Using a root cohesion, s_r = 81 psf, (see section 3.8.1) to estimate the increase in shear resistance along the failure plane by introducing vegetation (i.e., live poles installed on a 3 ft. by 3 ft. grid) the factor of safety increases for the worst loading (horizontal seepage) from failure condition to a factor of safety equal to about 1.8 (see table 4.7). The results of the infinite slope stability analysis are summarized in Tables 4.6 and 4.7. The following parameters were used for the analysis:

Slope inclination = 26.6° (2H:1V).

Soil shear strength parameters (CU tests): ϕ' = 32°, c' = 0.

Soil shear strength parameters (back-calculated): ϕ' = 30°, c' = 50 psf.

Groundwater table: Assume saturated conditions.

Groundwater flow/seepage direction: Parallel, horizontal, and vertical to the slope.

Soil density = 120 pcf (back-calculated) and 130 pcf (laboratory).

Depth to failure surface: H = 2 ft. (based on field observations).

Root cohesion, s_r = 81 psf (live pole grid of 3 ft by 3 ft, see section 3.8.1).

Table 4.6: Summary of infinite slope stability analysis for the Logan County demonstration site using shear strength parameters from CU tests.

Seepage Direction	Saturated Soil Density γ (pcf)	Depth to failure surface H (ft.)	Root Cohesion s_r (psf)	Factor of Safety F_s
Horizontal	130	2	0	0.50
Parallel to slope	130	2	0	0.65
Vertical	130	2	0	1.25
Horizontal	130	2	81†	1.28
Parallel to slope	130	2	81†	1.43
Vertical	130	2	81†	2.03

Note: Slope inclination = 26.6° (2H:1V); Soil shear strength parameters: $\phi' = 32°$, $c' = 0$

† Root cohesion, s_r = 81 psf, for 3 ft. by 3 ft. grid spacing (see section 3.8.1).

Table 4.7: Summary of infinite slope stability analysis for the Logan County demonstration site using back calculation shear strength parameters.

Seepage Direction	Saturated Soil Density γ (pcf)	Depth to failure surface H (ft.)	Root Cohesion s_r (psf)	Factor of Safety F_s
Horizontal	120	2	0	0.92
Parallel to slope	120	2	0	1.07
Vertical	120	2	0	1.67
Horizontal	120	2	81†	1.77
Parallel to slope	120	2	81†	1.92
Vertical	120	2	81†	2.52

Note: Slope inclination = 26.6° (2H:1V); Soil shear strength parameters: $\phi' = 30°$, $c' = 50$ psf

† Root cohesion, s_r = 81 psf, for 3 ft. by 3 ft. grid spacing (see section 3.8.1).

4.5.2 Logan County Demonstration Site Design

Following the stability analyses of the drainage channel slopes, a plan for the bioengineering demonstration plots was produced by the research team. Figure 4.28 shows the site plan layout developed during the preliminary drafting of the final plans to be put out for bid by ODOT.

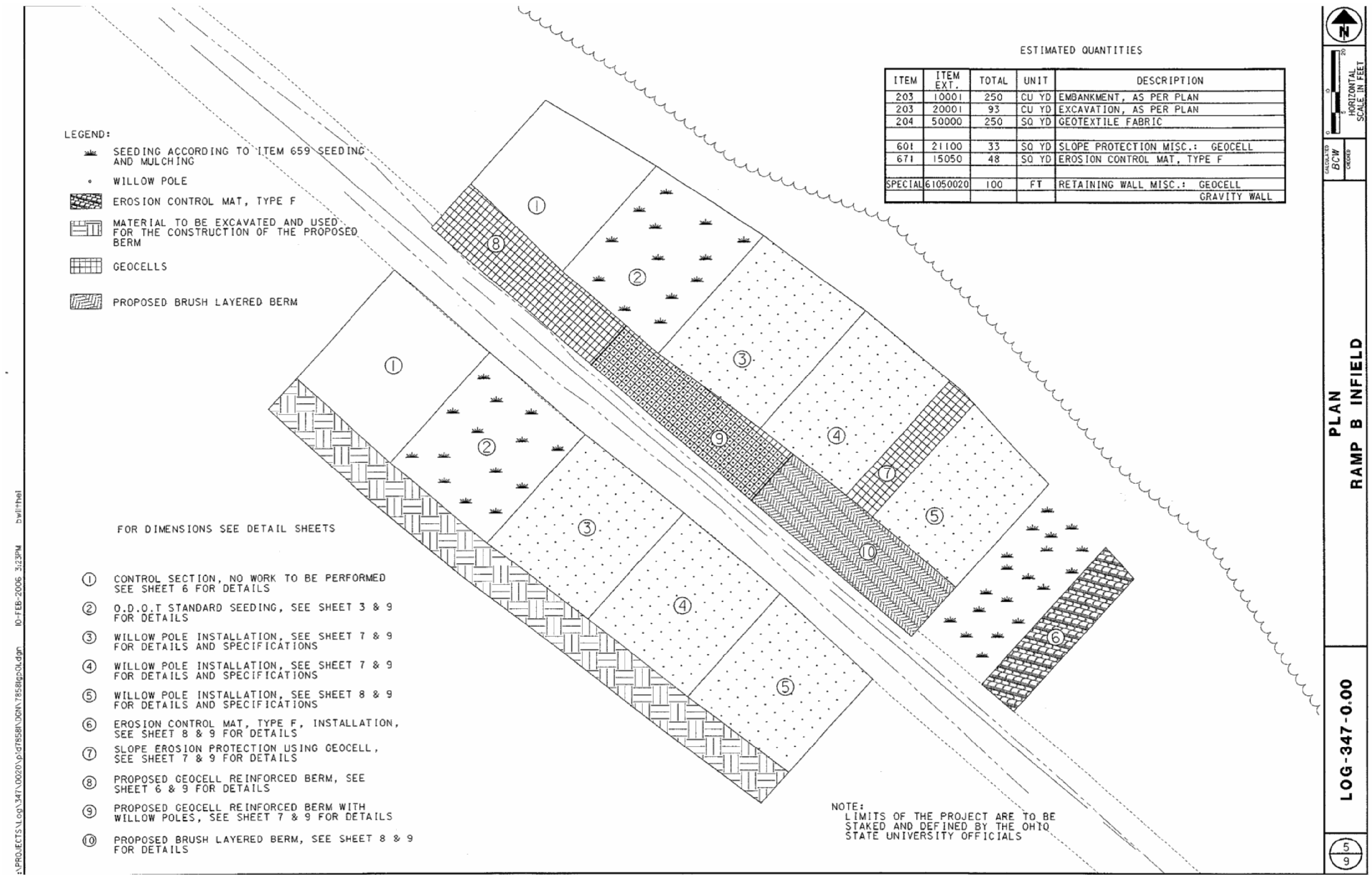


Figure 4.28: Logan County demonstration site design layout.

4.5.3 Installations

The different installation methods for the Logan County bioengineering demonstration site are summarized below. The complete design document by ODOT "LOG-347-0.00" presents all of the installation details for this site.

4.5.3.1 Demonstration Plots

Seven demonstration plot types, numbered 1 through 7 as shown on Figure 4.28, were constructed for the site. The plot types are as follows: (1) Control section, (2) ODOT standard seeding, (3) Best Method willow pole installation with graded/scraped slope face, (4) Best Method willow pole installation with no slope grading, (5) Minimal Method willow pole installation, (6) Erosion control mat, and (7) Geocell slope protection.

The two live pole installations methods, Best Method and Minimal Method, are outlined in sections 4.5.3.1.1 and 4.5.3.1.2, respectively. The purpose of having two installation methods is to evaluate if the degree of effort required for the Best Method is warranted. In other words, does the added labor of the Best Method in comparison to the Minimal Method lead to a greater survivability of the live poles and is the survival difference between the two methods substantial enough to warrant the additional time and labor of the Best Method? Figure 4.29 shows the details of the Best Method installation technique.

A control section where no live poles are to be installed was also included to compare the live pole installations to the "do nothing" approach. Further, the usefulness of regrading a slope face prior to live willow pole installation will also be evaluated. The usefulness of geocells, seeding, and an erosion control mat will be evaluated as plots for each of these installations are included in the design.

4.5.3.1.1 Live Pole Willow Installation: Best Method

1. Install between Nov 1 and April 1
2. Poles submerged/soaked in water several days prior to installation
3. V-cut bottom end of pole, cut off notches on sides
4. Wire top of pole to prevent splitting during installation
5. Drive pole to refusal into pre-augured hole which has been drilled 6 in. shorter than desired installation depth
6. Backfill hole with appropriate material (e.g., loam, sand) and add deer/animal repellant fertilizer tablet
7. Top 6 in. of backfill: native soil cuttings or bentonite with a 1 ft. PVC breather tube
8. Damaged top of pole cut off at a slight angle to leaving 1½ ft. protruding above grade
9. Top of pole rewired as in Step 4 to prevent splitting from desiccation
10. Square of biodegradable landscape fabric pinned around the base of the pole to prevent adjacent growth of competitive vegetation
11. Erosion control matting/seeding

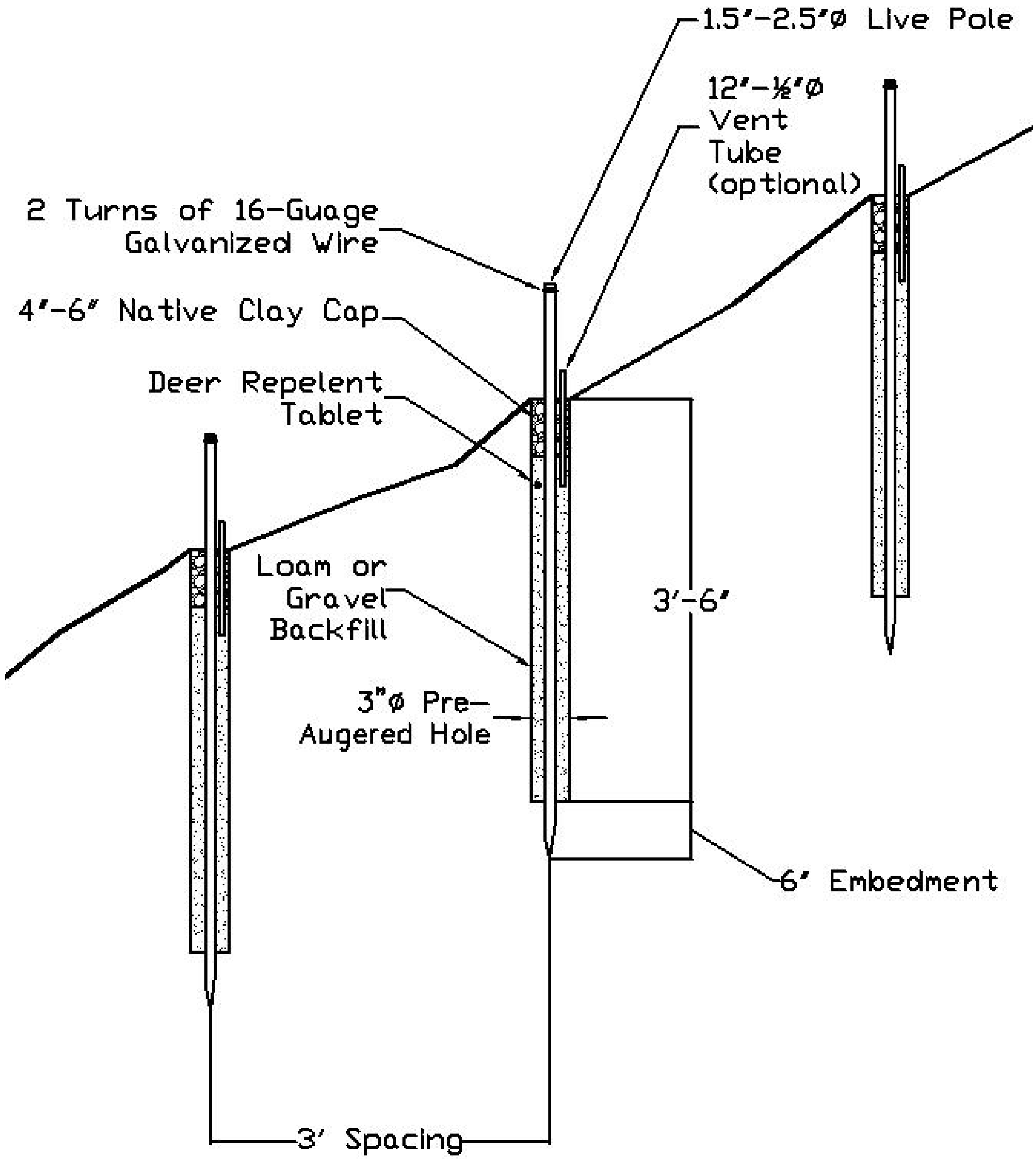


Figure 4.29: Typical Best Method live pole installation detail.

4.5.3.1.2 Live Willow Pole Installation: Minimal Method

1. Install between November 1 and April 1
2. Poles submerged/soaked in water several days prior to installation
3. Cut off knobs on pole
4. Pole inserted into pre-augured hole and tamped
5. Backfill hole with appropriate material + deer/animal repellant fertilizer tablet
6. Top 6 in. backfill with native soil cuttings or bentonite with 1 ft. PVC breather tube
7. Square of biodegradable landscape fabric
8. Erosion control matting/seeding

4.5.3.2 Berms

Three berm types numbered 8 through 10 as indicated on Figure 4.28 are planned for the site. The berm types are as follows: (8) Geocell berm, (9) Geocell berm with willow poles, and (10) Brushlayer berm. The three berms will enable a side-by-side comparison of different approaches to slope stabilization using berm construction.

4.6 SUMMARY

In closing, this chapter presented the research efforts completed for the Logan County field demonstration site prior to construction. Throughout this chapter, data and analyses are presented and discussed. Specifically, attention is given to the site conditions including the geology, climate, and subsurface and groundwater conditions. Sections outlining field exploration, laboratory testing, and site instrumentation are also presented.

Lastly, the stability analyses showed that adequate factors of safety could be achieved and the subsequent bioengineering design that was selected is presented.

CHAPTER 5

DEMONSTRAION SITE: MUSKINGUM COUNTY I-70/SR-83

5.1 PROJECT DESCRIPTION

The Muskingum County I-70/SR-83 demonstration site is located near the village of New Concord in Muskingum County, Ohio, approximately 80 miles east of Columbus, Ohio. The site location is shown in Figures 5.1 and 5.2. Two embankments, one supporting the onramp from Ohio State Route 83 (SR-83) to westbound Interstate 70 (I-70) and the other one supporting westbound I-70, have experienced shallow landslides and erosion. The two areas are shown relative to the surrounding site features in Figure 5.2. The upper, southernmost, slope area has shown signs erosion whereas the lower, northernmost, slope area has evidence of both erosion and shallow mass movement. Because landslide activity is only on the lower slope area, the lower embankment is the focus of this demonstration site and no remedial measures were used on the upper embankment.

Figure 5.1: Vicinity Map: Muskingum County bioengineering demonstration site.

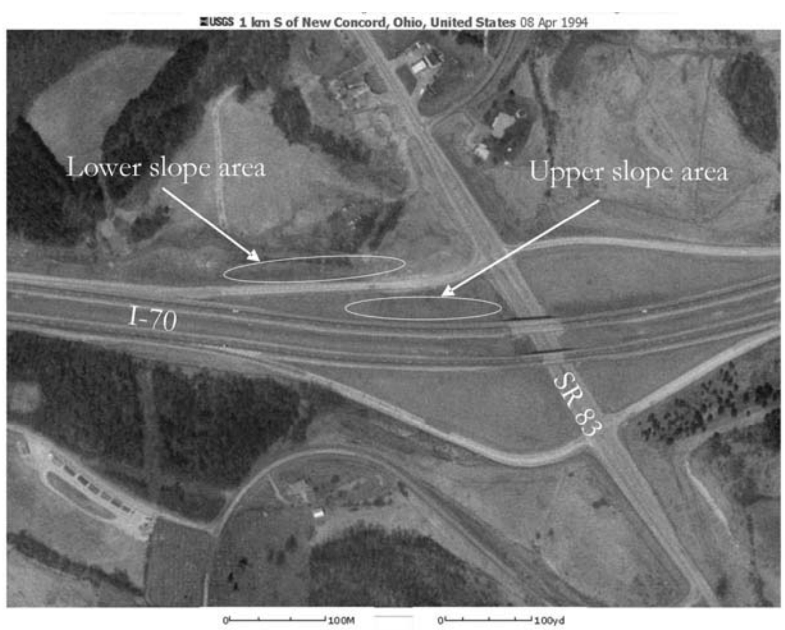


Figure 5.2: Site Map: Ariel view of project location. (From USGS)

5.2 SITE CONDITIONS

5.2.1 General conditions

The lower embankment was originally constructed at a 2H:1V slope using compacted native fill material comprised mostly of red clay (A-6 and A-7-6 – AASHTO Classification). Following a period of active landslides in the 1980s, the embankment was repaired under the supervision of the ODOT. The 1991 reconstruction included a 10H:1V bench and back drain. A typical cross section of the reconstruction in 1991 is shown in Figure 5.3.

Shallow slides and erosion gullies were observed by ODOT personal and OSU researchers in spring 2004. The section of the lower embankment, which has been affected by slope instability (i.e., erosion and landsliding), is approximately ¼ mile in length with slope heights ranging from 30 to 50 ft. The most pronounced section of instability is marked by shallow scarps that extend approximately 150 ft. across the slope. The scarps visually define areas of shallow mass movement where blocks or slabs of soil, approximately 2 ft. thick, can be clearly identified. Seepage has also been observed at several areas on the slope where saturated zones exist. These seepage areas are marked by the telltale growth of cattails and other hydrophilic vegetation. Additionally, the seepage and landslide areas appear to coincide. Figure 5.4 shows a photograph of the lower embankment taken in May 2005 where the landslide scarps are clearly visible. It is suspected that storm water runoff is a prime cause of slope instability at this site and observations made by OSU and ODOT personal in October 2005 identified several locations where storm water runoff appeared to be channelized along the ramp above the lower embankment near the areas of slope instability.

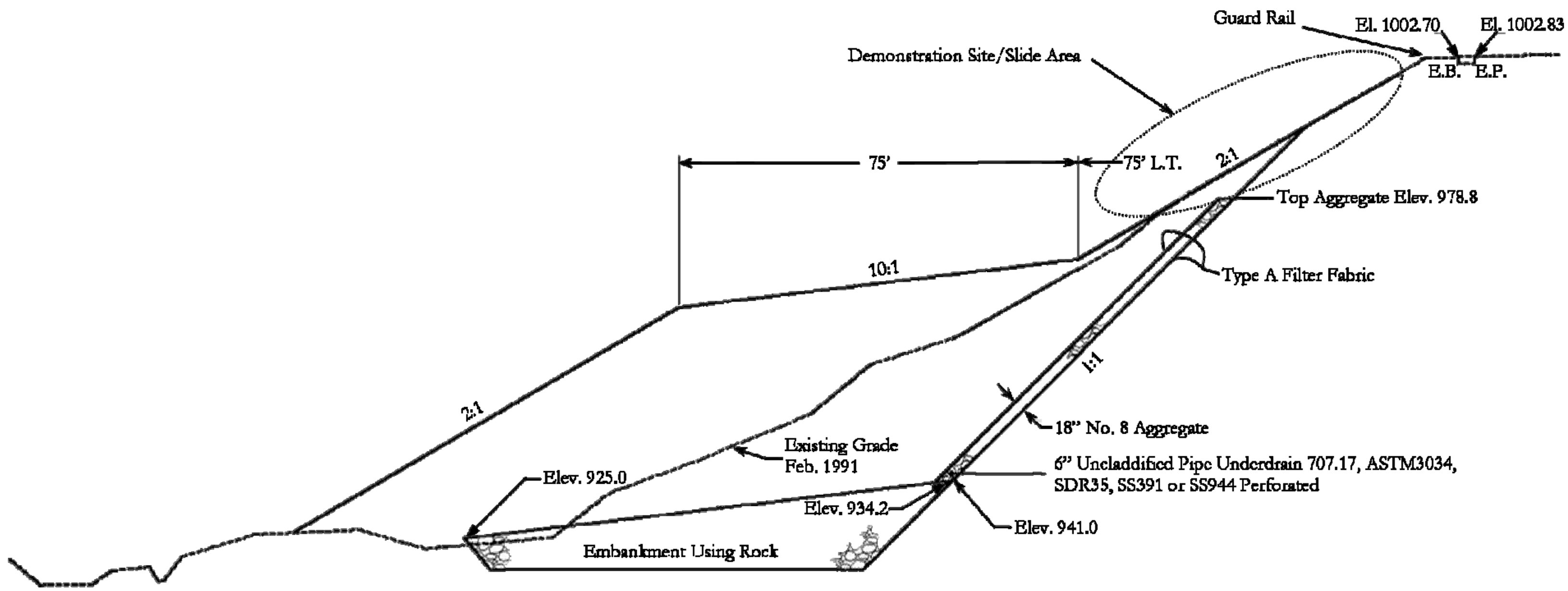


Figure 5.3: Cross-section of repaired lower embankment,ODOT STA 731+50 from ODOT 1991 plans (Mus-70-26.61).

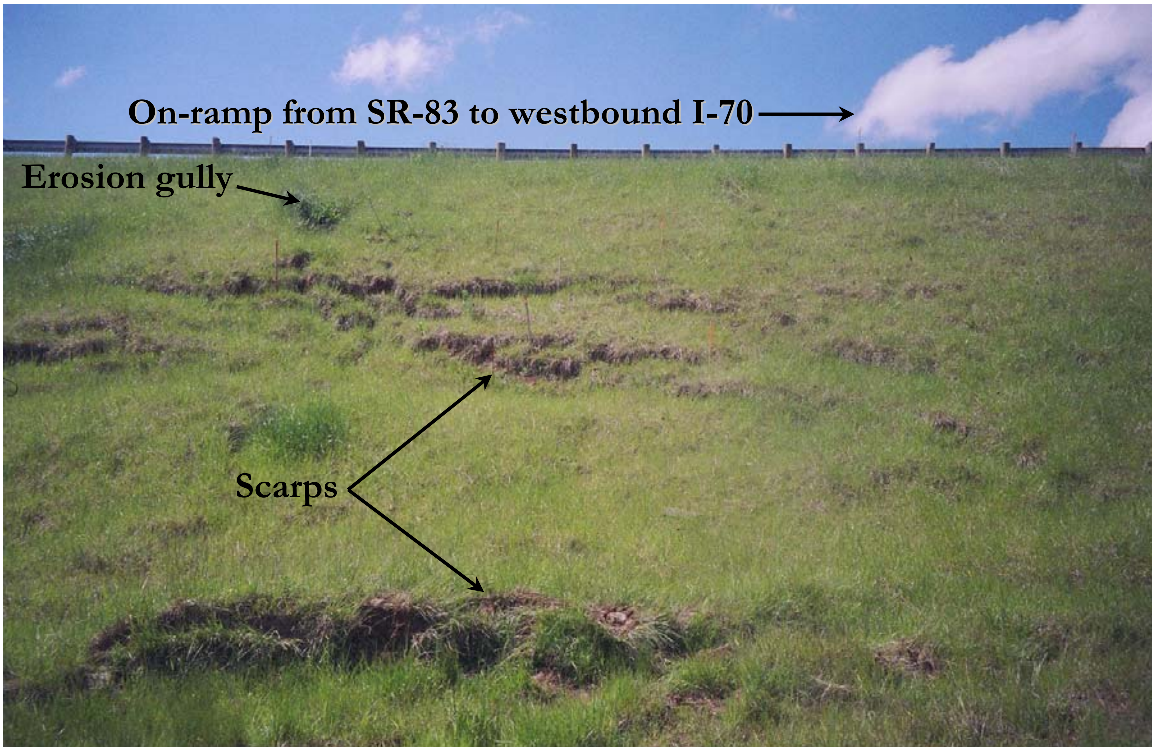


Figure 5.4: Landslide scarps at lower embankment (May 2005). Photo taken from 10H:1V bench near STA 0+75. (see Figure 5.7 for STA reference)

A bioengineering remediation plan was designed for the unstable section of the lower embankment in consultation with David H. Barker and Donald H. Gray and construction of this plan occurred during May and June 2005.

5.2.2 Regional Geology

Muskingum County lies on the unglaciated, dissected Allegheny Plateau. The underlying bedrock in the county is mainly sandstone, siltstone, clay shale, and limestone; all of which were derived from sediments laid down during the Late Mississippian, Pennsylvanian, and Early Permian periods (Stout 1918; Steiger 1996).

The local geology and more importantly the propensity for slope instability in this region has been well documented where a cycle of weathering, landslide, and repair following construction is commonplace (Wu et al. 1987; Wu et al. 1993; DeLong 1996). Geologically, the Conemaugh formation is predominant in this area and has been mapped by the Ohio Department of Natural Resources (ODNR) as being subject to severe slope failure (Hansen 1995). The geographic extent of the Conemaugh formation is shown in Figure 5.5. Embankments constructed of and cut slopes in the Conemaugh formation have historically been notorious for slope failures. Since their construction, ODOT has been burdened with embankment slope failures along I-77 and I-70 in Muskingum and surrounding counties where red clays derived from the Conemaugh formation have been used for construction. One such event, pictured in Figure 5.6, occurred just east of the Muskingum County demonstration site in 1986. This deep-seated rotational landslide destroyed the westbound lanes of I-70 and took 30 days to repair with a cost of over $600,000 (DeLong 1996).

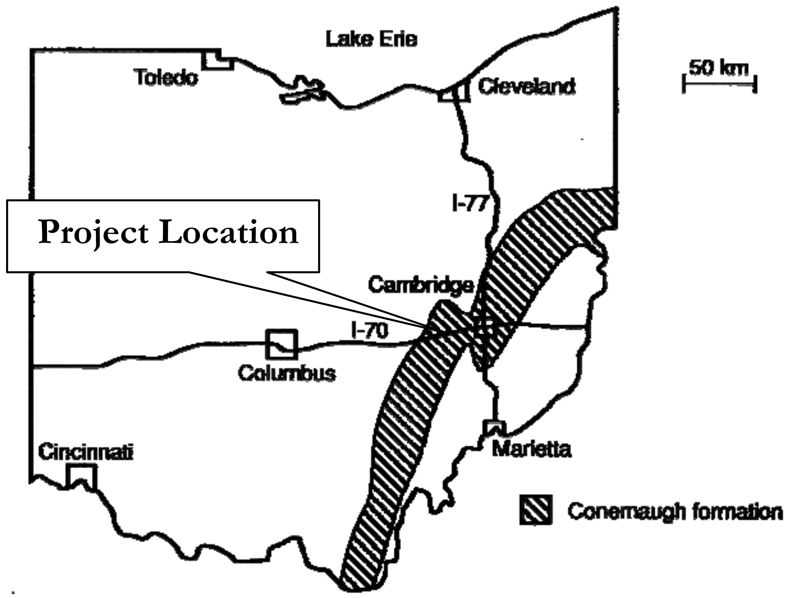


Figure 5.5: Ohio map showing the location of the Conemaugh formation. (After Wu et al. 1987)

Figure 5.6: Landslide in Conemaugh formation in 1986 destroyed the westbound lanes of I-70 near New Concord, Guernsey County. (Photo from DeLong (1996))

5.2.3 Climate

Historical climate data has been collected for Cambridge, Ohio, approximately 10 miles east of the Muskingum County demonstration site, by the Midwestern Regional Climate Center (MRCC) for the period between 1971 and 2000. The average annual rainfall over this 30-year period is 39.16 in. February is typically the driest month with an average monthly rainfall of 2.30 in. and July is wettest producing 4.25 in. of rain. The average maximum, minimum, and mean annual temperatures are 63.5°F, 41.5°F, and 52.5°F, respectively, with January being the coldest (29.1°F mean avg.) and July being the warmest (73.6°F mean avg.) months on average. Temperature, rainfall, and growing season tables are included in Appendix B.

It is also noteworthy that both of the embankments studied at this site are north facing. This typically influences a site's microclimate because north-facing hillsides/slopes receive less direct sunlight than their south-facing counterparts do. For this reason, one can typically expect north facing slopes to be less susceptible to drought; however, this is not the case at this site. It has been observed that periods of little to no rainfall and excessively high temperatures are commonplace during the summer months despite the fact that July is historically recorded as the wettest month of the year. Field observations indicate that there is virtually no shade at this site and that little if none of the rainfall is retained in the slope's surface "crust" during the summer months. Therefore, drought conditions should be considered during the plant selection and maintenance scheduled for bioengineered projects in this area.

5.2.4 Subsurface Conditions

The soils encountered during the boring and test pit explorations generally consist of compacted fill material comprised of red lean to fat clay with varying amounts of gravel derived from the Conemaugh formation.

5.2.5 Groundwater Conditions

Groundwater levels at the site were directly observed in the borings while drilling and on regular basis thereafter from installed piezometers, monitoring wells, and tensiometers. Perched groundwater has been observed near the surface of the lower slope. Areas of seepage from the slope face have also been observed during the site visits. Data collected from the instrumentation indicates that in general soil suction in the upper crust of the slide prone slope is less than 5 psi.

5.3 FIELD EXPLORATION AND LABORATORY TESTING

5.3.1 Field Reconnaissance

Numerous field visits have been made to evaluate the site conditions. Notes, measurements, and photographs were taken to identify the landslide and erosion features at the site and to observe groundwater, soil, climate, and vegetative conditions. During the initial site visits, features like landslide scarps, cracks, bulges, erosion gullies, and bare ground were mapped. Figure 5.7, shows the initial landslide and erosion features.

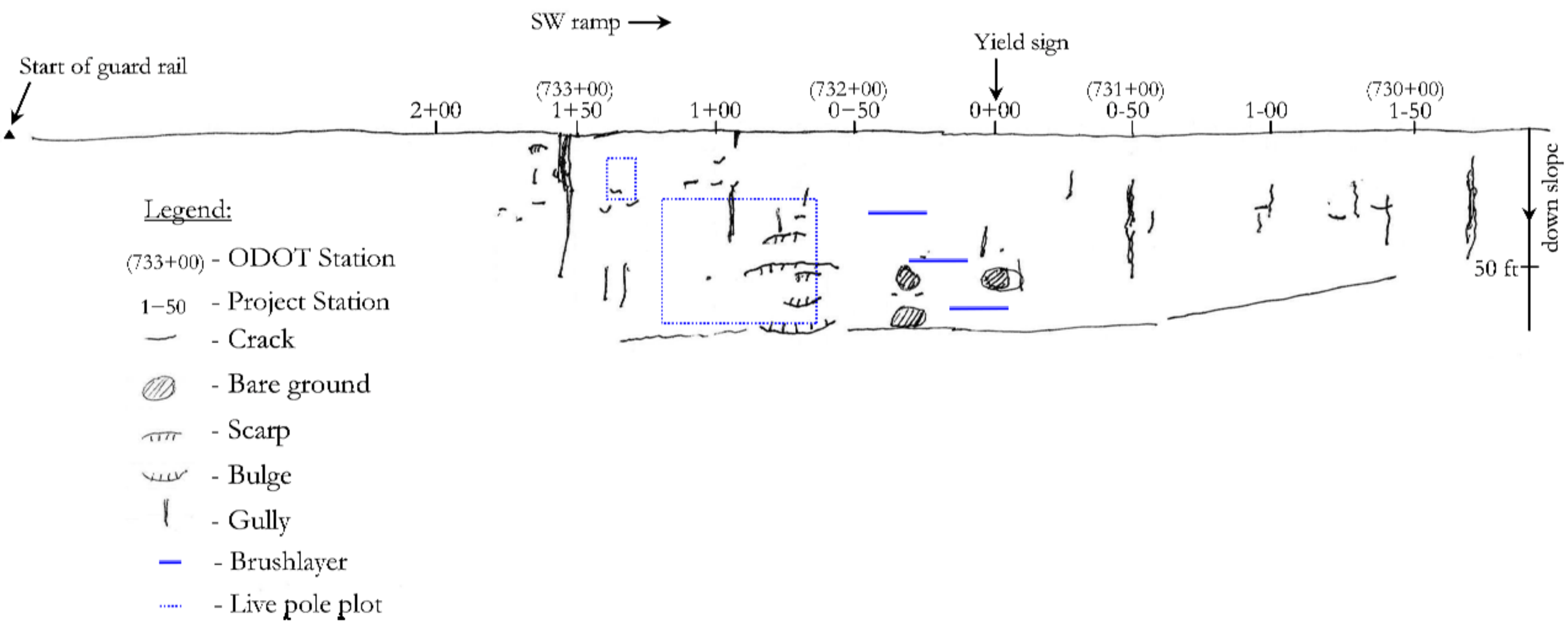


Figure 5.7: Muskingum County demonstration site – Landslide and erosion feature survey of lower embankment slope by Wu and Kokesh 9/12/04.

5.3.2 Field Exploration

The subsurface conditions at the site were evaluated by drilling and sampling from three borings ranging in depth from 10 to 35 ft. and excavating three test pits to depths of approximately 3 ft. Borings, designated as B-1 through B-3, were drilled under the supervision of ODOT at the locations shown on Figure 5.8. Boring B-1 was drilled 32 ft. deep on the 10H:1V bench of the lower embankment (ODOT STA 732+49, OFFSET 271' LT.); boring B-2 was drilled 32 ft. deep near the lower embankment guardrail (ODOT STA 732+38, OFFSET 154' LT.); and boring B-3 was drilled 10 ft. deep near the upper embankment guardrail (ODOT STA 741+42, OFFSET 112' LT.). The borings were drilled with a truck mounted drill rig using hollow stem augers on March 8 and 9, 2005. Standard penetration tests (SPT) were conducted using a 140 pound hammer falling 30 in. to drive a 2 in. O.D. split barrel sampler for 18 in. Soil samples obtained from the SPT sampling were preserved in plastic tubs, visually identified in the field, and classified at the ODOT laboratory. Relatively undisturbed soil samples were obtained using Shelby tubes and preserved with paraffin/petroleum jelly seals. Boring logs from this subsurface investigation program were prepared by ODOT and are included in Appendix B.

Shallow test pits were excavated using hand tools by OSU researchers to depths of approximately 3 ft. below the ground surface. The approximate location of the test pits are shown on Figure 5.9. Relatively undisturbed samples were obtained by pressing thin-walled samplers into the soil at the bottom of the test pits.

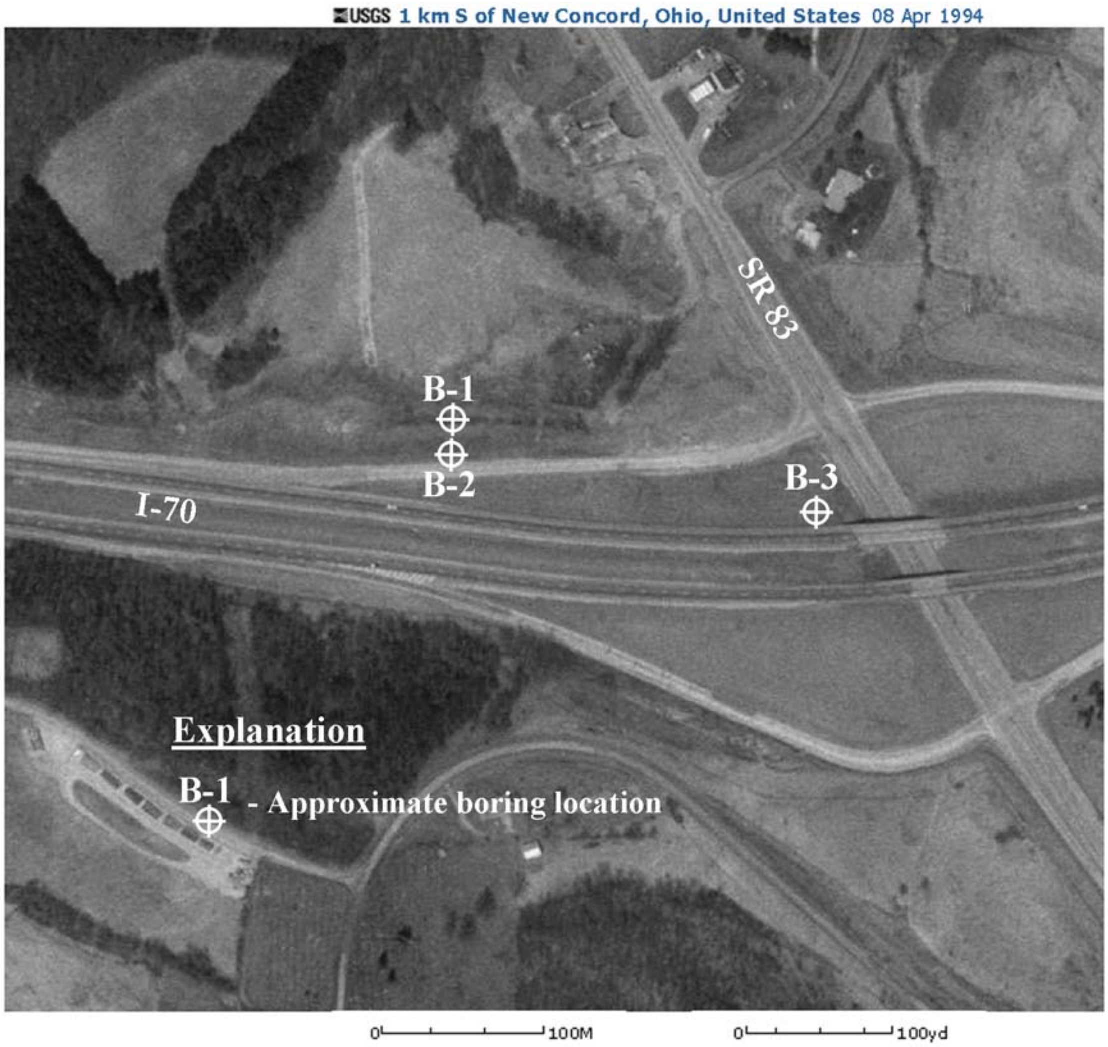


Figure 5.8: Approximate boring locations on aerial photo. (Photo from USGS)

5.3.3 Laboratory Testing

5.3.3.1 Physical properties

Relatively undisturbed specimens were recovered from shallow test pits near the unstable areas of the lower embankment. Two unconfined compression (UCS) tests were performed at the OSU Soil Mechanics Laboratory. The unconfined compressive strengths, q_u, of the samples were 998 and 1,626 psf. Unconfined compression test reports are included in Appendix B.

Classification tests were performed at the OSU and ODOT geotechnical laboratories. The material in the upper several ft. of the unstable areas has been classified as A-6a, A-6b (ODOT classification). The results for all of the classification tests are included on the boring logs in Appendix B.

5.3.3.2 Chemical properties

Soil nutrient testing was performed by Calmar, Inc. of Westerville, Ohio, on a representative soil sample recovered from the upper two ft. of the lower slope area. The sample was obtained by mixing soil collected from several locations across the area experiencing instability and that is to be repaired using bioengineering methods. The soil nutrient report for this site is included in Appendix B. Based on the test results provided by Calmar, it has been determined that the pH at this site is higher than and the nitrogen, phosphorus, potassium, and organic matter levels are lower than optimum for the growth of hardwood trees and shrubs and bushes. As per Calmar's recommendation, 2.8, 3.4, and 3.1 pounds per 1,000 ft^2 of nitrogen, phosphorus, and potassium, respectively, should be added to the soil per season. Additionally, 5, 11, 16, and 21 pounds per 1,000 ft^2 of sulfur should be added seasonally for soil depths of 3, 6, 9, and 12 in., respectively.

5.4 SITE INSTRUMENTATION

Groundwater and stability conditions have been regularly monitored as part of the ongoing site investigation. Tensiometers, piezometers, and gypsum moisture blocks have provided data about the conditions. Shallow and deep slope inclinometers have also been installed to measure slope movements. Figure 5.9 shows the location and type of instruments that have been installed at this site and the proceeding sections provide specific details pertaining to different instrumentation. The identification labels used for the tensiometers, gypsum moisture blocks, and shallow piezometers consist of two digits followed by a letter followed by two digits. The first two digits correspond to the location number shown on Figure 5.9. The letter signifies the type of instrument; for example, "T" for tensiometer, "G" for gypsum moisture block, and "P" for piezometer. The final two digits represent the instrument depth in inches. For example, 02T36 is the identification label for the 36 in. deep tensiometer installed at the 02 location shown on Figure 5.9. Similarly, 07P54 identifies the 54 in. deep piezometer installed at the 07 location.

5.4.1 Tensiometers

Jet fill tensiometers were installed to measure the soil suction of the upper three ft. of the unstable slope. In total, 11 tensiometers, seven 24 in. deep and four 36 in. deep, were installed. Data collected between August and November 2005 indicates that the soil suction in the upper 3 ft. is generally less than 5 psi. The locations are shown in Figure 5.9 and the data is presented in Table 5.1 and Figures 5.10 through 5.14. Positive pressure is reported when the correction for the standing column of water in the tensiometer is greater than the in situ suction.

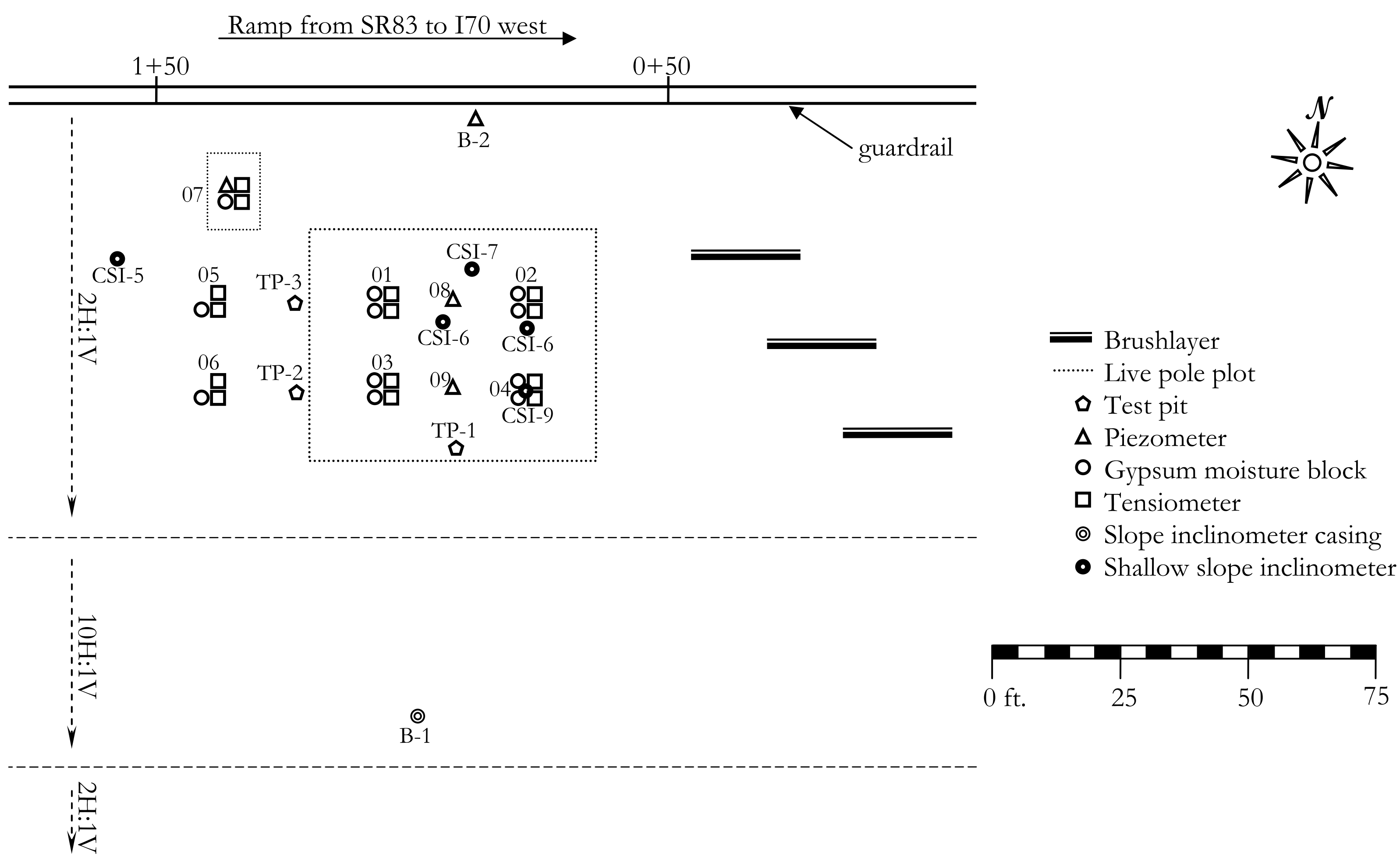


Figure 5.9: Muskingum County site plan and instrumentation layout. (see Figure 5.7 for STA reference)

Table 5.1: Tensiometer data for Muskingum County site.

#	ID	Depth (ft.)	Pore Water Pressure in Soil* (psi) (negative = suction)				
			8/3/05	8/4/05	9/28/05	11/3/05	11/10/05
01T	01T24	2	-0.5	-0.5	--	0.6	0.6
	01T36	3	-0.5	-0.2	0.8	0.5	0.8
02T	02T24	2	-1.2	-0.2	--	--	--
	02T36	3	-0.8	-0.7	0.6	0.5	0.6
03T	03T24	2	-0.4	-0.4	--	0.6	0.8
	03T36	3	-0.2	-0.2	1.2	1.1	--
04T	04T24	2	-0.5	-0.5	0.6	--	--
	04T36	3	-0.9	-0.9	0.8	--	1.1
05T	05T24	2	-1.2	-1.1	--	0.6	0.6
06T	06T24	2	-3.6	-5.3	--	0.5	0.6
07T	07T24	2	-2.1	-2.1	--	0.4	0.4

*values have been corrected for elevation corresponding to the water column in the tensiometer

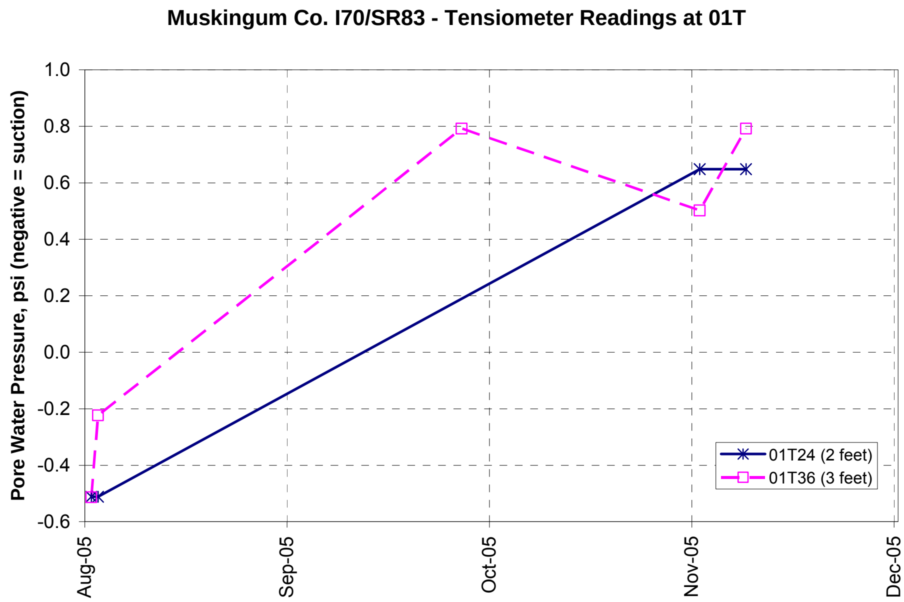


Figure 5.10: Tensiometer data for 01T.

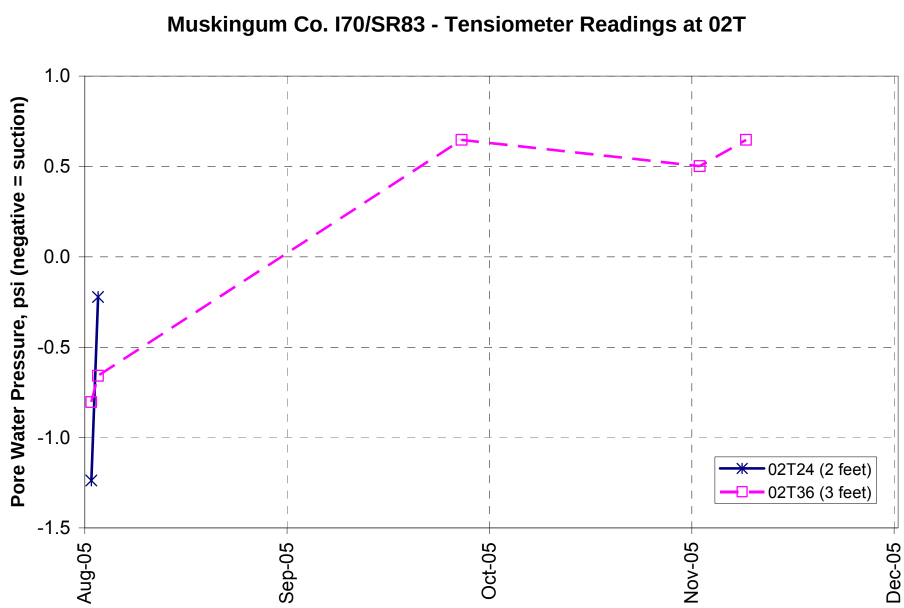


Figure 5.11: Tensiometer data for 02T.

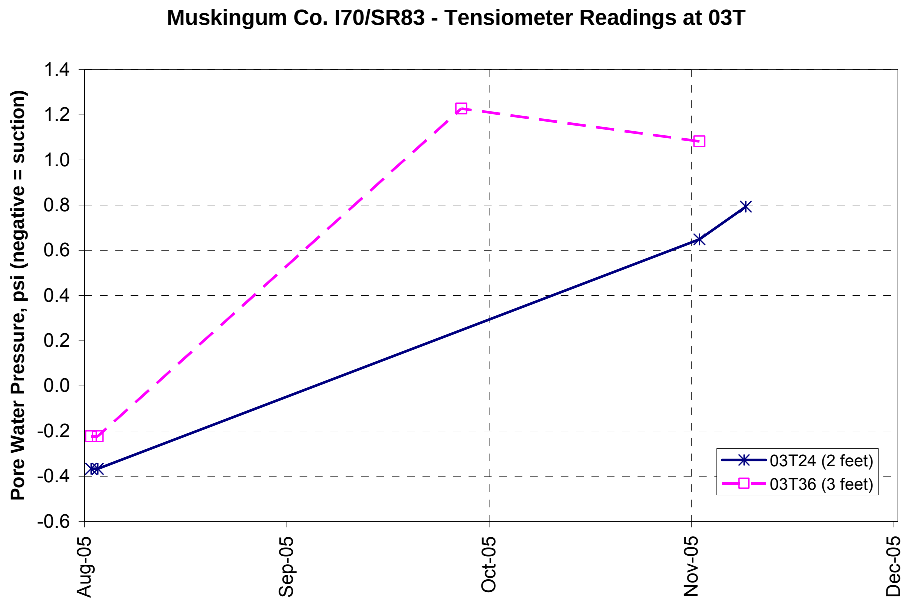


Figure 5.12: Tensiometer data for 03T.

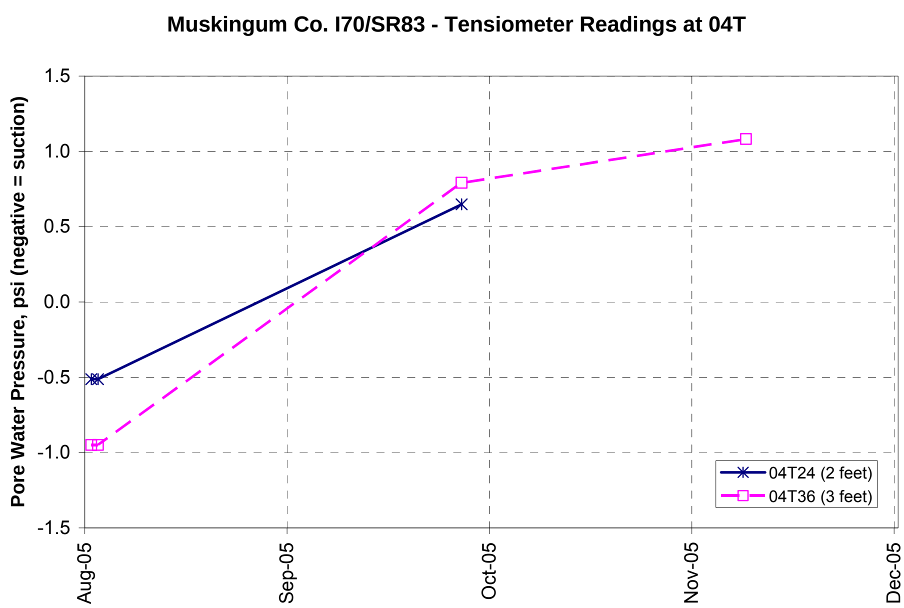


Figure 5.13: Tensiometer data for 04T.

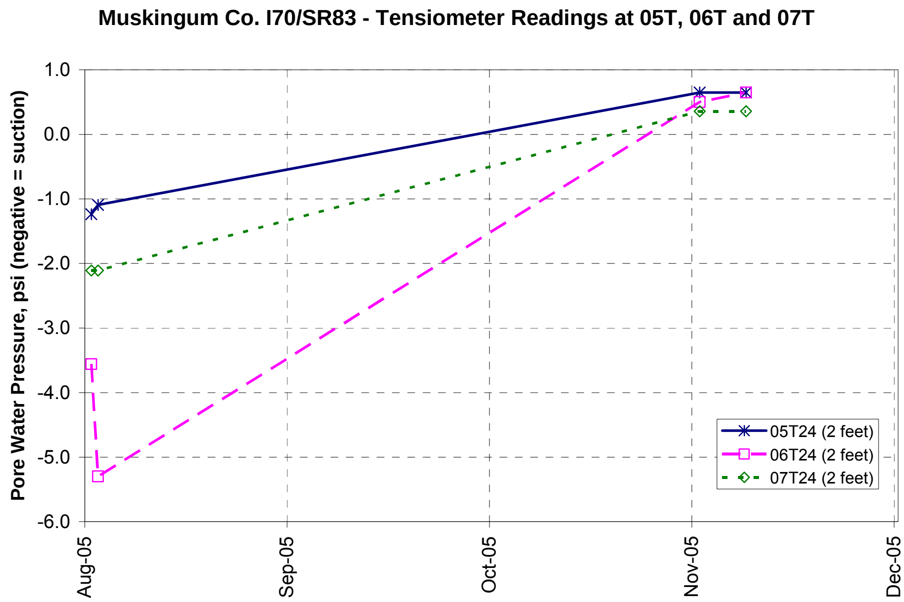


Figure 5.14: Tensiometer data for 05T, 06T and 07T.

5.4.2 Gypsum Moisture Blocks

Gypsum moisture blocks (G-Blocks) manufactured by the Soilmoisture Equipment Corp. have also been used to monitor the subsurface moisture regime. Fourteen G-Blocks, seven 24 in. deep and seven 36 in. deep, were installed in August 2005. Soil suction values have been indirectly measured through correlations to the electronic resistance measured across each G-Block as given in the manufacture's G- Block relationships (see Figure 3.1). The collected G-Block data is presented in Table 5.2 and Figures 5.15 through 5.21. Large data scatter, particularly at 2 ft., may reflect irregular pattern of moisture content near the surface. In an attempt to correlate the moisture block readings to the in situ moisture content, moisture content of soil samples were obtained at the G-Block locations. Figure 5.22 presents the initial G-Block reading taken on 8/3/05 plotted against the in situ moisture content. Due to the high degree of scatter, it is concluded that no correlation exists between moisture content and G-Block readings at this site. Figure 5.23 presents the G-Block readings plotted against the tensiometer readings taken at the same depth. Due to the high degree of scatter, it is concluded that no correlation exists between the soil suction measured by the tensiometers and G-Block readings at this site.

Table 5.2: Gypsum-block data for Muskingum County site.

			Pore Water Pressure in Soil* (psi) (negative = suction)					
#	ID	Depth (ft.)	8/3/05	8/4/05	9/28/05	11/3/05	11/10/05	2/15/06
01G	01G24	2	-4.8	-5.3	-6.9	-7.3	-7.1	-4.6
	01G36	3	-5.3	-5.1	-5.1	-5.6	-4.6	-5.6
02G	02G24	2	--	--	--	--	--	--
	02G36	3	-3.5	-3.2	-1.5	-3.5	-3.5	-4.3
03G	03G24	2	-5.9	-5.8	-4.7	-3.8	-3.5	-4.7
	03G36	3	-4.9	-4.9	-4.8	-4.9	-4.9	-4.8
04G	04G24	2	-5.1	-5.6	-5.8	-5.1	-4.8	-2.8
	04G36	3	-4.1	-4.1	-3.2	-3.2	-2.8	-4.1
05G	05G24	2	> -1.5	> -1.5	> -1.5	> -1.5	> -1.5	-3.5
	05G36	3	-2.2	-3.2	-2.8	-2.8	> -1.5	-3.5
06G	06G24	2	-2.8	-5.1	-1.5	-1.5	> -1.5	-3.2
	06G36	3	> -1.5	> -1.5	> -1.5	> -1.5	-3.2	-3.5
07G	07G24	2	-4.9	-5.6	-10.3	-9.6	-9.8	-8.4
	07G36	3	-4.3	-4.8	-5.3	-2.2	-2.8	-3.2

*suction values determined from empirical data provided by Soilmoisture Equipment Corp. (2000)

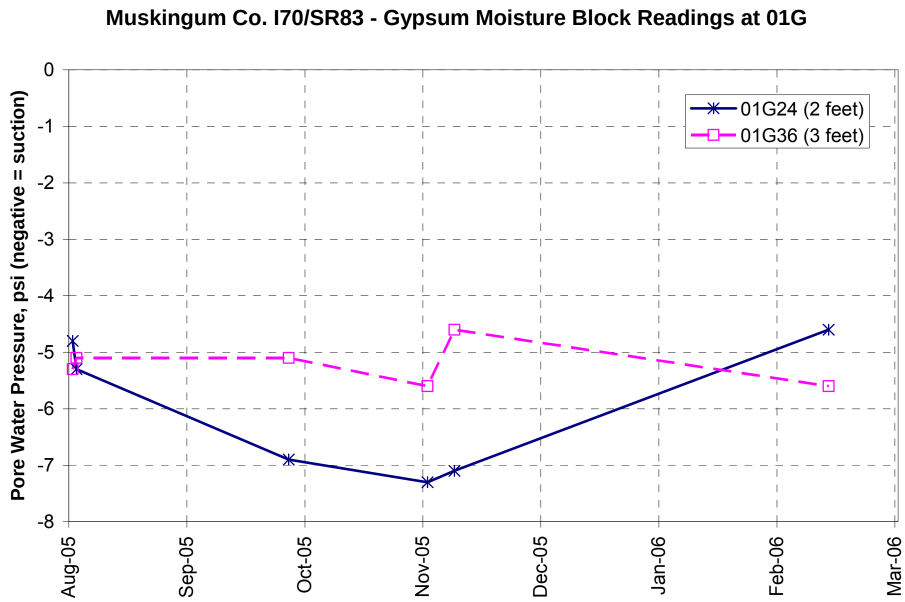


Figure 5.15: G-Block data for 01G.

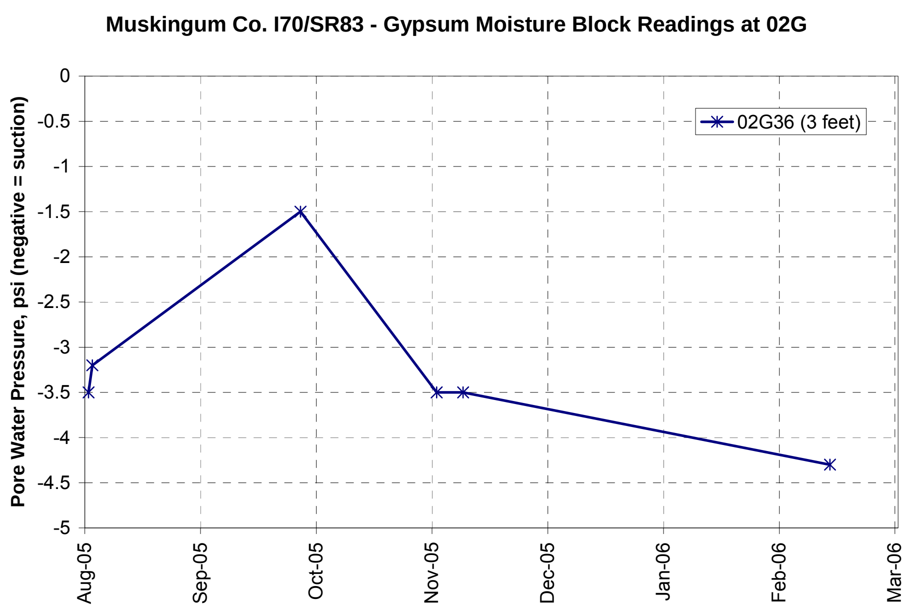


Figure 5.16: G-Block data for 02G.

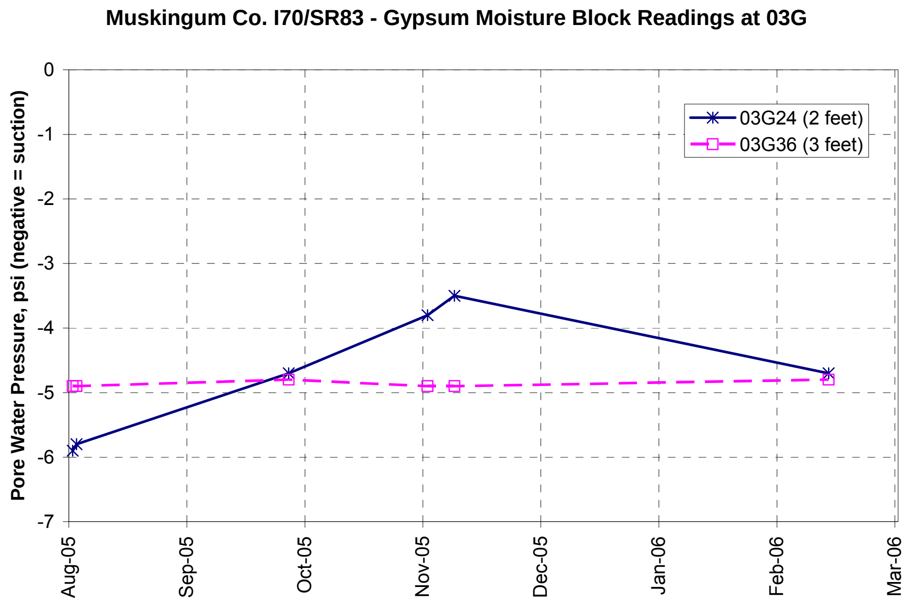


Figure 5.17: G-Block data for 03G.

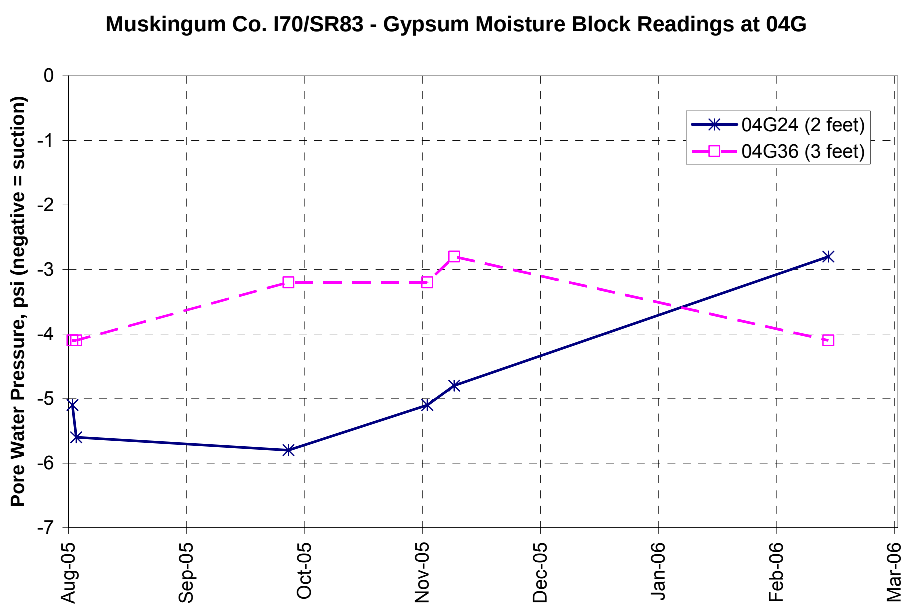


Figure 5.18: G-Block data for 04G.

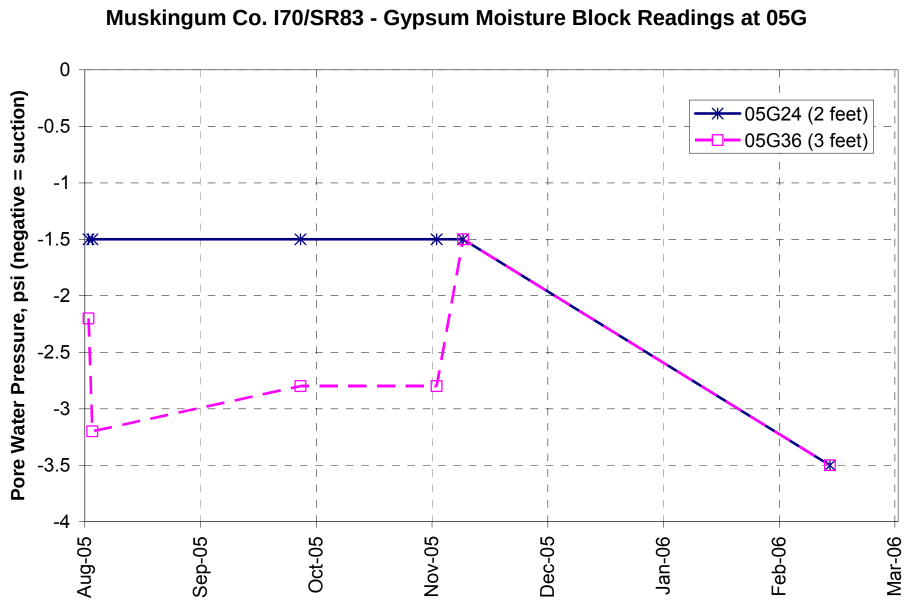


Figure 5.19: G-Block data for 05G.

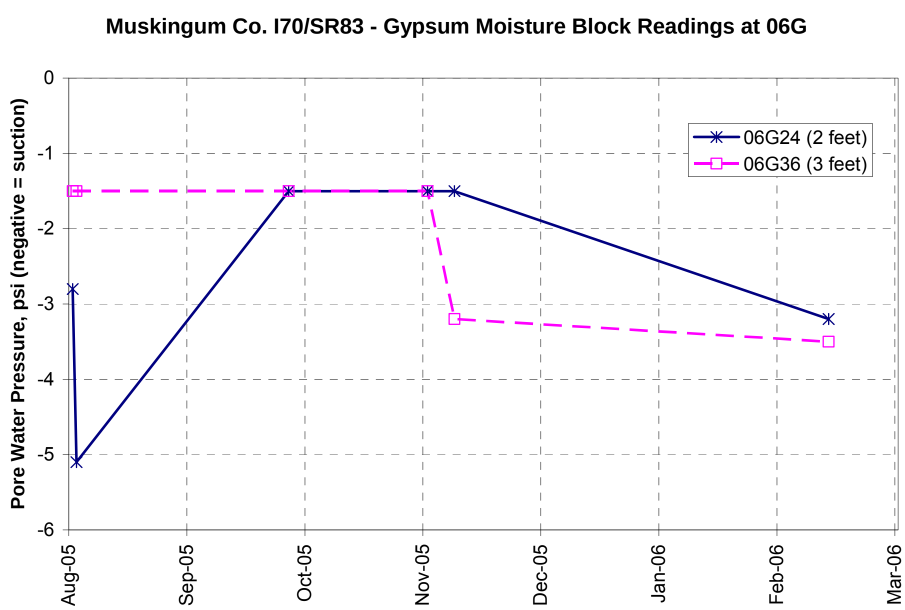


Figure 5.20: G-Block data for 06G.

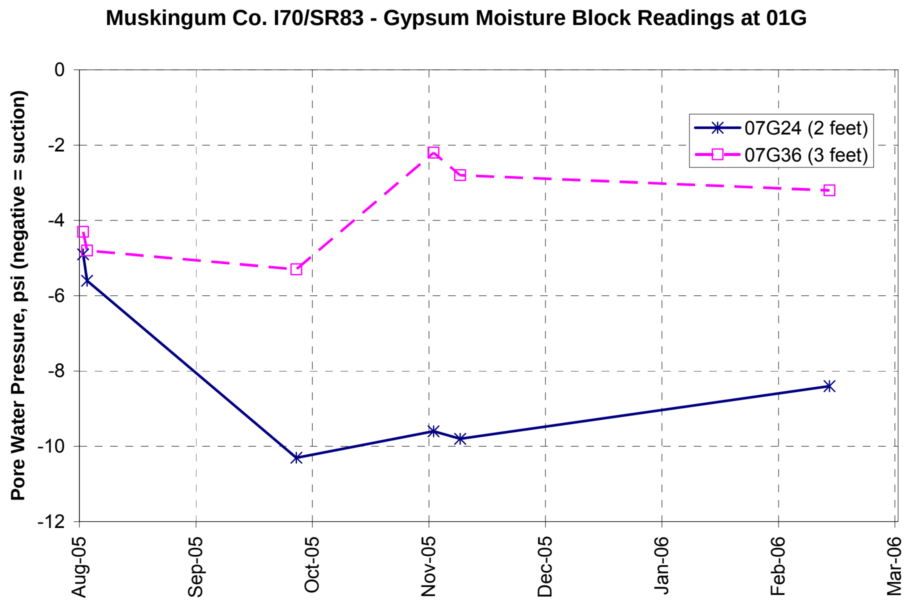


Figure 5.21: G-Block data for 07G.

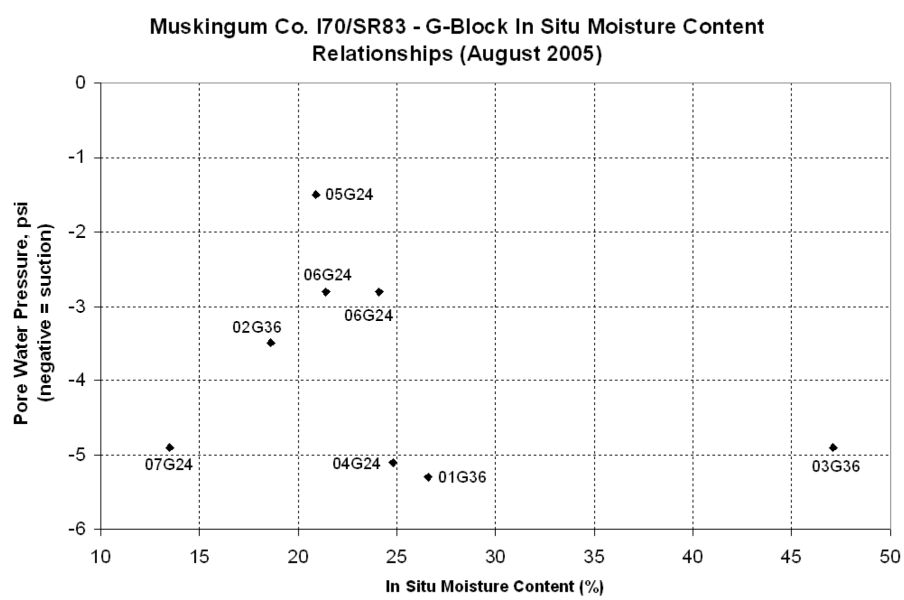


Figure 5.22: G-Block data versus in situ moisture content.

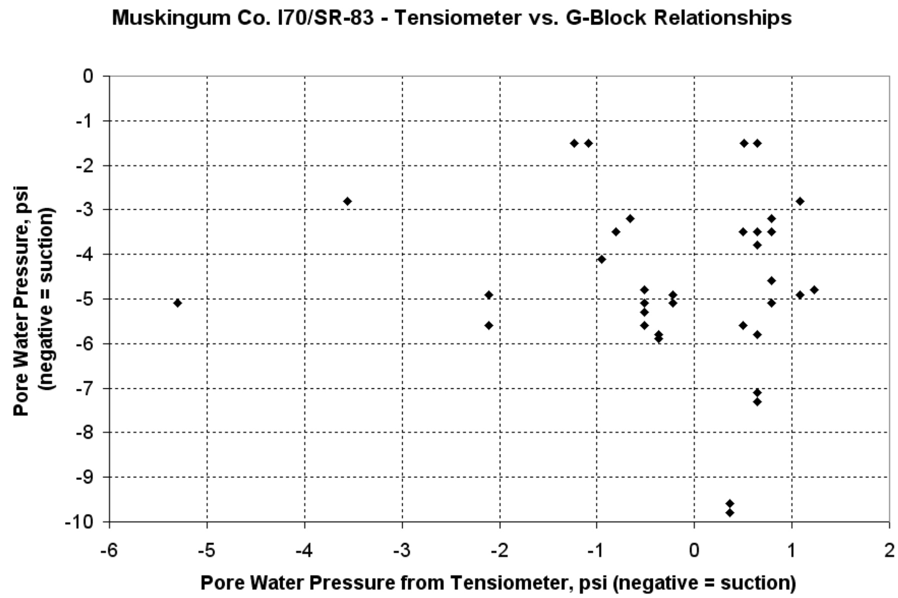


Figure 5.23: G-Block data versus tensiometer data.

5.4.3 Piezometers

In addition to tensiometers and G-Blocks, piezometers were installed across the Muskingum County demonstration site to investigate the groundwater conditions further. Piezometers at 32 and 10 ft. deep were installed in boreholes B-2 and B-3, respectively. Three shallow, approximately 4½ ft. deep, piezometers were installed; two in the main live pole plot and one in the upper live pole plot. The approximate locations of the piezometers are shown on Figures 5.8 and 5.9.

Piezometric levels of 10.2 and 7.1 ft. have remained essentially unchanged throughout the measurement period in the B-2 and B-3 piezometers, respectively. Piezometric heads ranging from 1.1 to 3.5 ft. have been observed in the shallow piezometers. Piezometer data is presented in Figures 5.24 and 5.25.

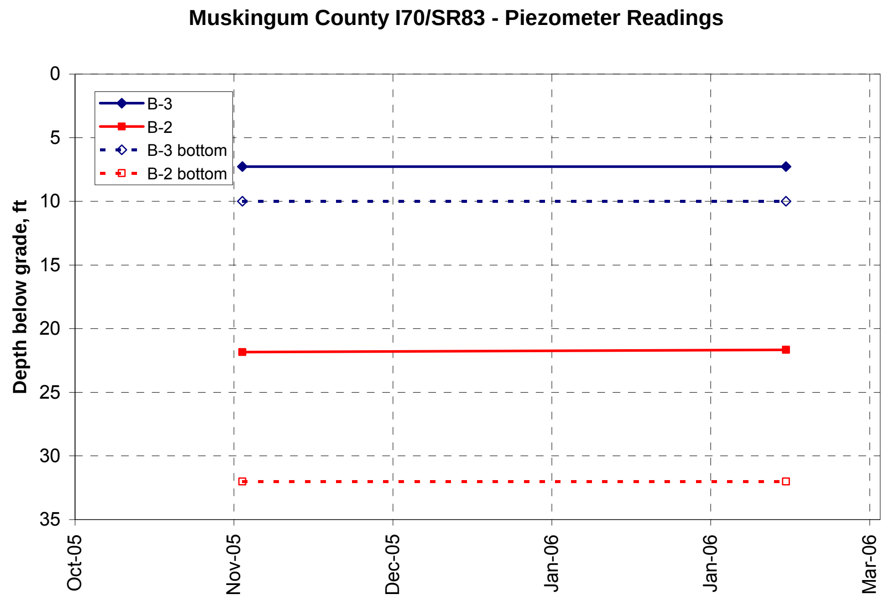


Figure 5.24: Piezometer data for B-1 and B-2.

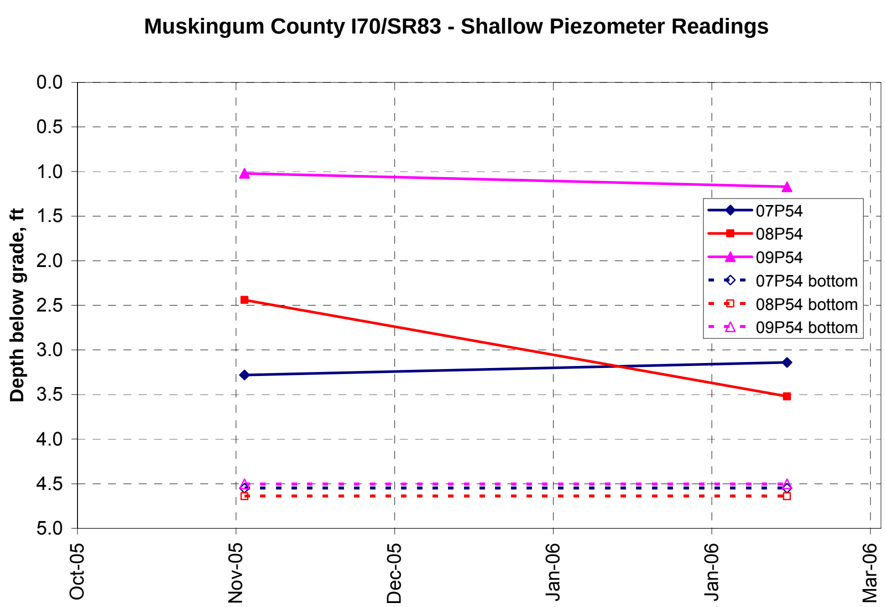


Figure 5.25: Piezometer data for 07P54, 08P54, and 09P54.

5.4.4 Inclinometers

5.4.4.1 Slope indicator

One 30 ft. inclinometer casing was installed in borehole B-1. The upper section of casing became disconnected from the lower sections shortly after the installation, rendering the installation unreadable and, for this reason; data was not collected or reported.

5.4.4.2 Shallow slope inclinometers

Shallow copper slope inclinometers (CSIs) were installed between December 2004 and January 2005 to measure near-surface slope movements. In order to measure movement at shallow depths flexible inclinometer tubes are needed. The CSI installations consist of a five ft. long ⅜ in. diameter copper pipe placed in a 2 in. diameter pre-augered hole backfilled with clean sand. In total, nine such installations were installed at the Muskingum County field demonstrations site; CSI-1 through CSI-4 on the upper slope and CSI-5 through CSI-9 on the lower slope placed in soil blocks that appeared to be moving as shown in Figure 5.26.

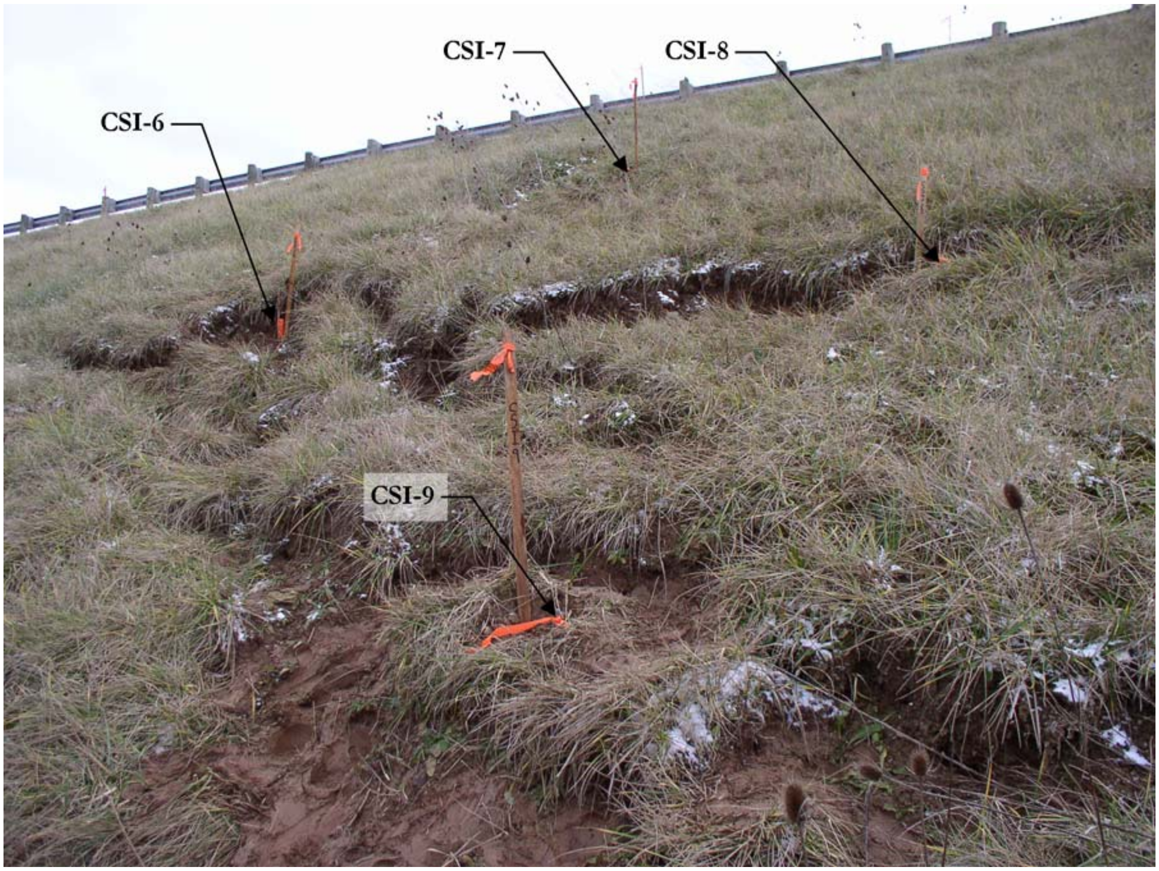


Figure 5.26: Muskingum County lower slope CSIs near STA 0+75 (January 2005). (see Figure 5.7 for STA reference)

Prior to slope regrading and installation of live poles, in May 2005, CSI-5 through CSI-9 were exhumed from the lower slope. Of the five exhumed CSIs, CSI-8 and CSI-9 were the only ones to be recovered successfully without damage. Slope movements of approximately 2 in. were measured at the ground surface by CSI-8 and CSI-9 for the five-month period between January 2005 and May 2005. The shapes of the exhumed CSIs indicate that the slope movement (displacement) is gradually increasing from about 2 ft. below the ground surface towards the top (ground surface), as shown on Figure 5.27.

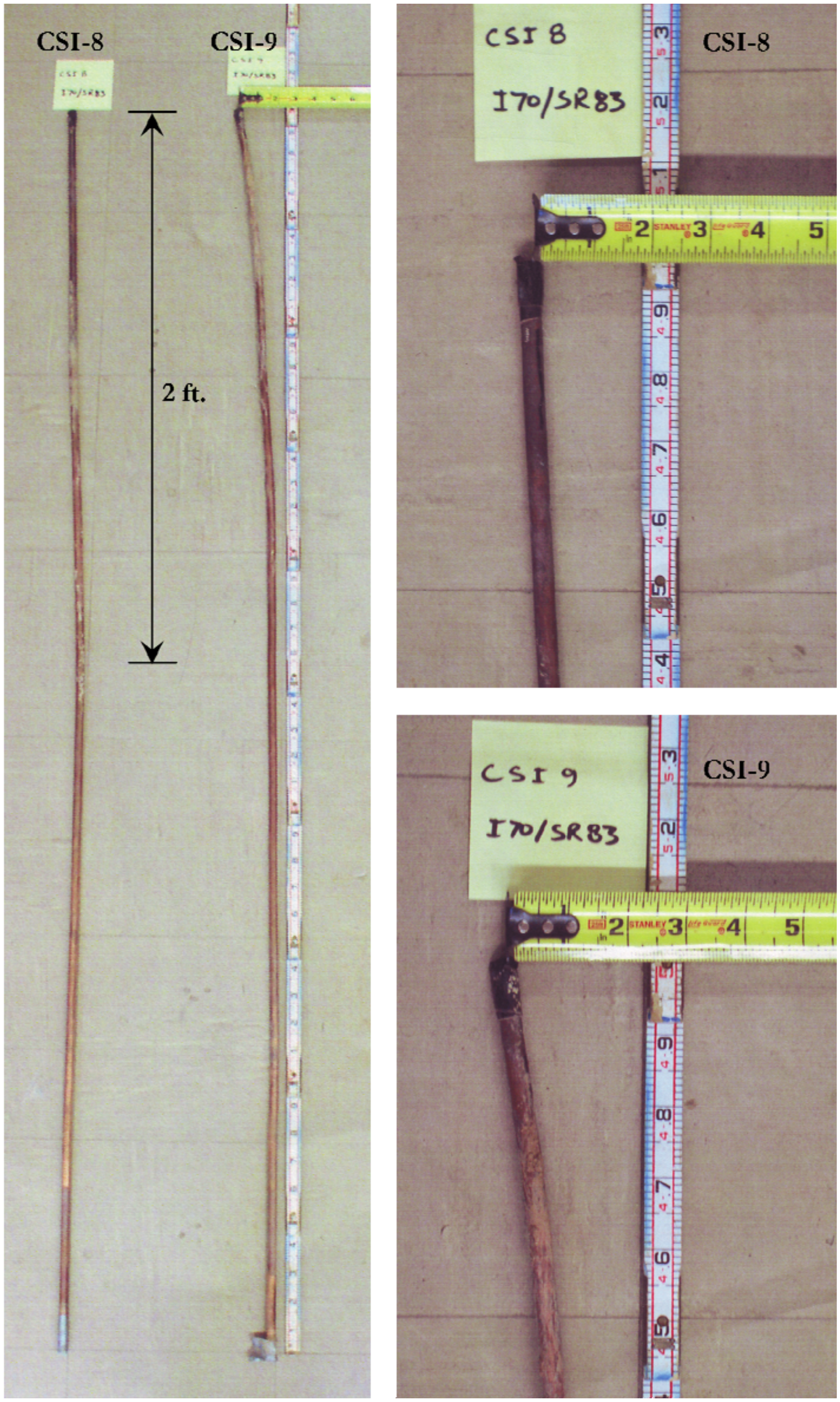


Figure 5.27: Exhumed CSI-8 and CSI-9.

5.5 SOIL BIOENGINEERING DESIGN

5.5.1 Introduction

The research team chose live poles and brushlayer berms as the soil bioengineering methods to mitigate shallow sliding and erosion on the lower embankment. The live poles were chosen for slope stabilization and the brushlayers for erosion control. Two live pole plots and a series of three brushlayer berms were constructed during May and June 2005. Figure 5.9 provides a site plan showing the locations of the live pole and brushlayer bioengineering demonstration sections. The stations listed along the guardrail in Figure 5.9 are referenced to a yield sign as shown on the landslide and erosion feature survey, Figure 5.7.

5.5.2 Slope Stability

5.5.2.1 Assessment

During the period between January and May 2005, mid-slope movements of the lower embankment slope, on the order of 2 in., were directly measured at the ground surface using copper slope indicators (CSIs). Numerous shallow (approximately 2 ft. deep) block-type failure surfaces on the lower embankment slope were observed during the site visits. It is expected that during the winter and spring months of 2005 the slope experienced the most severe loading because precipitation was well above normal and the slope would experience the maximum pore pressure and weight. Further, the upper several ft. of the embankment prone to slope movement appears to be highly weathered due to wetting and drying. During the winter months, the entire slope was observed to be saturated or near saturated. During the summer months a dry and desiccated surface crust several in. thick forms with saturated to near saturated soils not encountered until approximately 2½ to 4 ft. below the ground surface. The persistent surficial movements further suggest that the factor of safety against sliding along the failure plane approximately two ft. below the ground surface is just slightly greater than unity and falls to approximately equal to unity ($F_s \approx 1$) during critical periods when episodic movements are triggered, most likely by heavy precipitation.

5.5.2.2 Slope Stability Analysis

The infinite slope stability analysis approach was used to determine baseline soil strength parameters for the embankment material by back calculating for the soil shear strength parameters where failure is imminent (factor of safety, $F_s \approx 1$) for the Muskingum County demonstration site. The slope analysis followed the infinite slope procedure and formulas presented in Taylor (1948) and Gray and Sotir (1996) which were discussed in section 2.4.1.1 and Figure 2.6. For this analysis, the slope and failure surface geometries were determined from observations made during field reconnaissance of the site. In addition, piezometer data was gathered to identify the groundwater conditions at this site, which indicate that near saturated to saturated conditions are expected near surface. The soil strength parameters from the back-calculated stability analysis for the site prior to repair were used for the design of the bioengineered remediation for the unstable slope at this site.

5.5.3 Live Pole Design

Preliminary calculations suggest that that a live pole grid spacing of 3 ft. by 3 ft. would increase the factor of safety from 1 (failure) to 1.8. The infinite slope stability analysis approach was used to analyze the shallow slips at the site (see section 2.4.1.1 and Figure 2.6). The analysis was performed where the undrained shear strength, s_u, and shear strength parameters c', and ϕ' of the soil were back calculated for failure surfaces 2, 2½, and 3 ft. below the ground surface. Using a root cohesion, s_r = 81 psf and 182 psf for pole grid spacing equal to 3 ft. by 3 ft. and 2 ft. by 2 ft., respectively, (see section 3.8.1), the factor of safety for these three depths was calculated. The results of the infinite slope stability analysis are presented in Tables 5.3, 5.4, and 5.5. The following parameters were used for the analysis:

Slope inclination = 26.6° (2H:1V).

Soil $\phi_u = \phi = 0°$ (undrained strength conditions).

Soil $c' = 0$ (drained strength conditions).

Groundwater table: Assumed at the ground surface.

Groundwater flow/seepage direction: Parallel, horizontal, and vertical to the slope.

Soil density = 125 pcf (laboratory testing).

Root cohesion, s_r = 81 psf (live pole grid of 3 ft by 3 ft, see section 3.8.1).

Root cohesion, s_r = 182 psf (live pole grid of 2 ft by 2 ft, see section 3.8.1).

Table 5.3: Summary of undrained infinite slope stability analyses for the Muskingum County demonstration site.

Depth to Failure Surface H (ft.)	Saturated Soil Density γ (pcf)	Soil Undrained Shear Strength* s_u (psf)	Pole Grid Spacing (ft. x ft.)	Root Cohesion† s_r (psf)	Factor of Safety F_s
2	125	100	3 x 3	81	1.81
2½	125	125	3 x 3	81	1.65
3	125	150	3 x 3	81	1.54
2	125	100	2 x 2	182	2.82
2½	125	125	2 x 2	182	2.45
3	125	150	2 x 2	182	2.21

* Soil shear strength, s_u, back calculated for when failure is imminent ($F_s \approx 1$) and no root cohesion ($s_r = 0$).

† Root cohesion, s_r = 81 psf, for 3 ft. by 3 ft. and 182 psf for 2 ft. by 2 ft. grid spacing (see section 3.8.1).

Table 5.4: Summary of drained infinite slope stability analyses for the Muskingum County demonstration site for pole grid spacing of 3 ft. by 3 ft.

Seepage Direction	Saturated Soil Density γ (pcf)	Drained Shear Strength Parameters* ϕ'	c' (psf)	Depth to failure surface H (ft.)	Root Cohesion† s_r (psf)	Factor of Safety F_s
Horizontal		33	51			
Parallel	125	33	35	2	81	1.81
Vertical		26.6	0			
Horizontal		33	64			
Parallel	125	33	44	2½	81	1.65
Vertical		26.6	0			
Horizontal		33	77			
Parallel	125	33	52	3	81	1.54
Vertical		26.6	0			

Note: Slope inclination = 26.6° (2H:1V);

* Shear strength parameters back calculated for when failure is imminent ($F_s \approx 1$) and no root cohesion ($s_r = 0$).

† Root cohesion, s_r = 81 psf for 3 ft. by 3 ft. grid spacing (see section 3.8.1).

Table 5.5: Summary of drained infinite slope stability analyses for the Muskingum County demonstration site for pole grid spacing of 2 ft. by 2 ft.

Seepage Direction	Saturated Soil Density γ (pcf)	Drained Shear Strength Parameters* ϕ'	c' (psf)	Depth to failure surface H (ft.)	Root Cohesion† s_r (psf)	Factor of Safety F_s
Horizontal		33	51			
Parallel	125	33	35	2	182	2.81
Vertical		26.6	0			
Horizontal		33	64			
Parallel	125	33	44	2½	182	2.45
Vertical		26.6	0			
Horizontal		33	77			
Parallel	125	33	52	3	182	2.21
Vertical		26.6	0			

Note: Slope inclination = 26.6° (2H:1V);

* Shear strength parameters back-calculated for when failure is imminent ($F_s \approx 1$) and no root cohesion ($s_r = 0$).

† Root cohesion, s_r = 182 psf for 2 ft. by 2 ft. grid spacing (see section 3.8.1).

5.6 INSTALLATIONS

This section describes the bioengineering installations during May and June 2005. All vegetative stock, consisting primarily of willow and some poplar was harvested from a drainage swale located on the property of Honda of America in Marysville, Ohio. The dormant live pole and brushlayer vegetative material was harvested during April 2005 and stored in a refrigerator (temp ≈ 35°) until installation. The willow species was identified, based on examination of leaf specimen, by Dr. Mac Alford of OSU Herbarium on June 7, 2005, to be *Salix exigua*, common name "sandbar willow" but possibly *Salix longifolia*.

5.6.1 Main Live Pole Plot

The main live pole plot was constructed during the last week of May and the first two weeks of June 2005. In total, 256 live poles, dormant hardwood cuttings of willow and poplar measuring approximately five ft. in length and 1 to 2½ in. in diameter, were installed on a 3 ft. grid pattern to stabilize this roughly 65 ft. by 45 ft. section of unstable slope. Additionally, nine live poles were installed in an erosion gully just above the main live pole plot to reduce erosion by lessening the velocity of highway runoff. Prior to the live pole installation, the slope face was graded (i.e., "scraped"). The gullies, rills, scarps, and bulges resulting from the previous erosion and landslide activity were smoothed over and filled with a dozer to achieve positive drainage.

The general installation method was similar for all of the live poles in the main plot with the only significant variable being the backfilling procedure. Three methods were employed to backfill the annular space between the live poles and the pre-augured installation holes: (1) loam backfill and a 4 to 6 in. clay cap, (2) gravel backfill and a 4 to 6 in.

clay cap, and (3) gravel backfill with a ½ in. diameter vent tube and a 4 to 6 in. clay cap. Vent tubes were installed for the third method with the intent of providing ventilation for the future live pole roots. Following the live pole installation, geo-jute mat was installed to reduce erosion on the graded slope face. The subsequent figures show the main plot after installation (Figure 5.28), the installation layout for the main live pole plot with the backfill method used for each live pole and the pole type (i.e., willow or poplar) (Figure 5.29), and a general detail of the live pole installation (Figure 5.30).

Figure 5.28: Muskingum County main live pole plot after installation near STA 0+75 (June 2005). (see Figure 5.7 for STA reference)

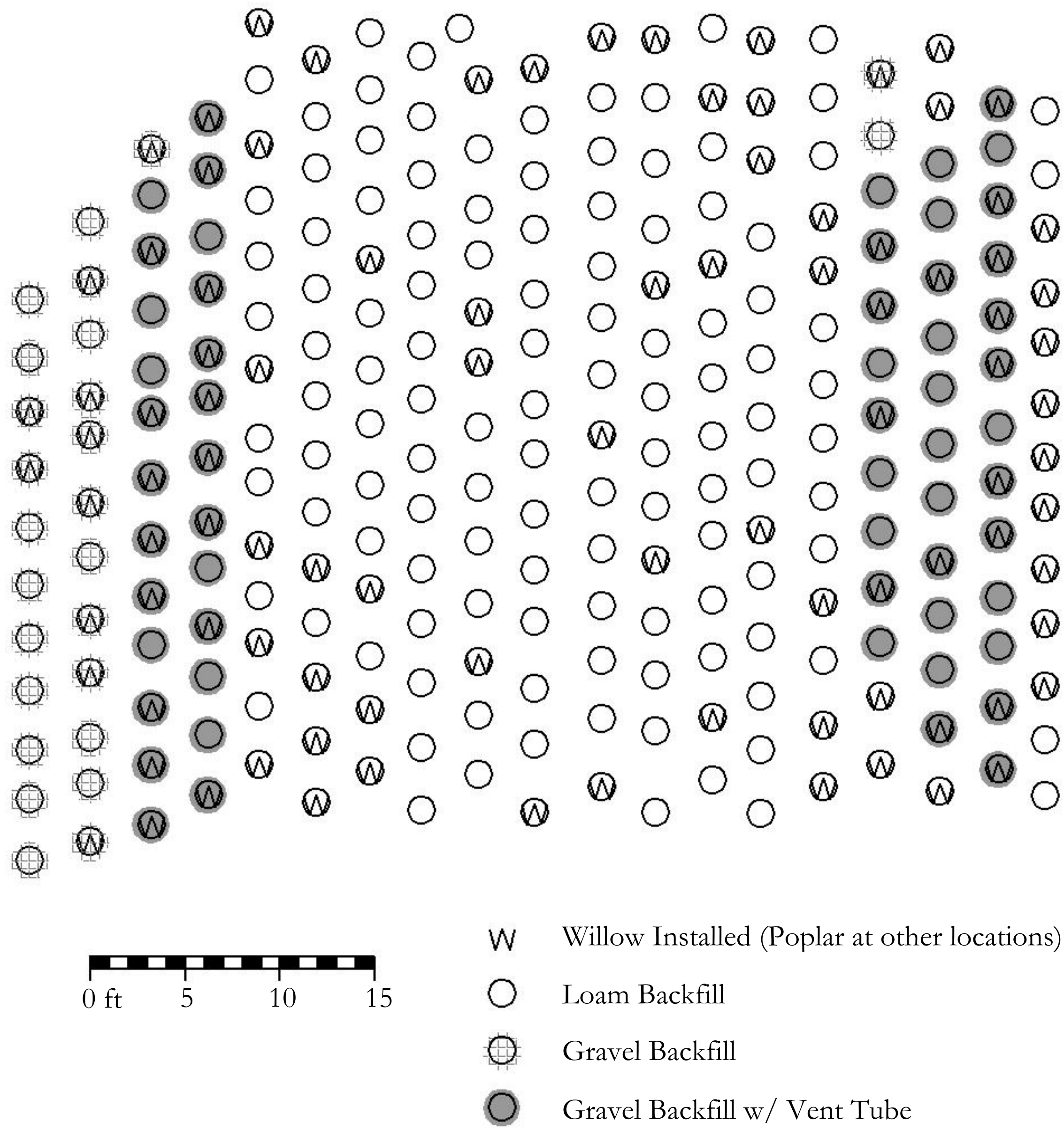


Figure 5.29: Muskingum County main live pole backfill/installation layout.

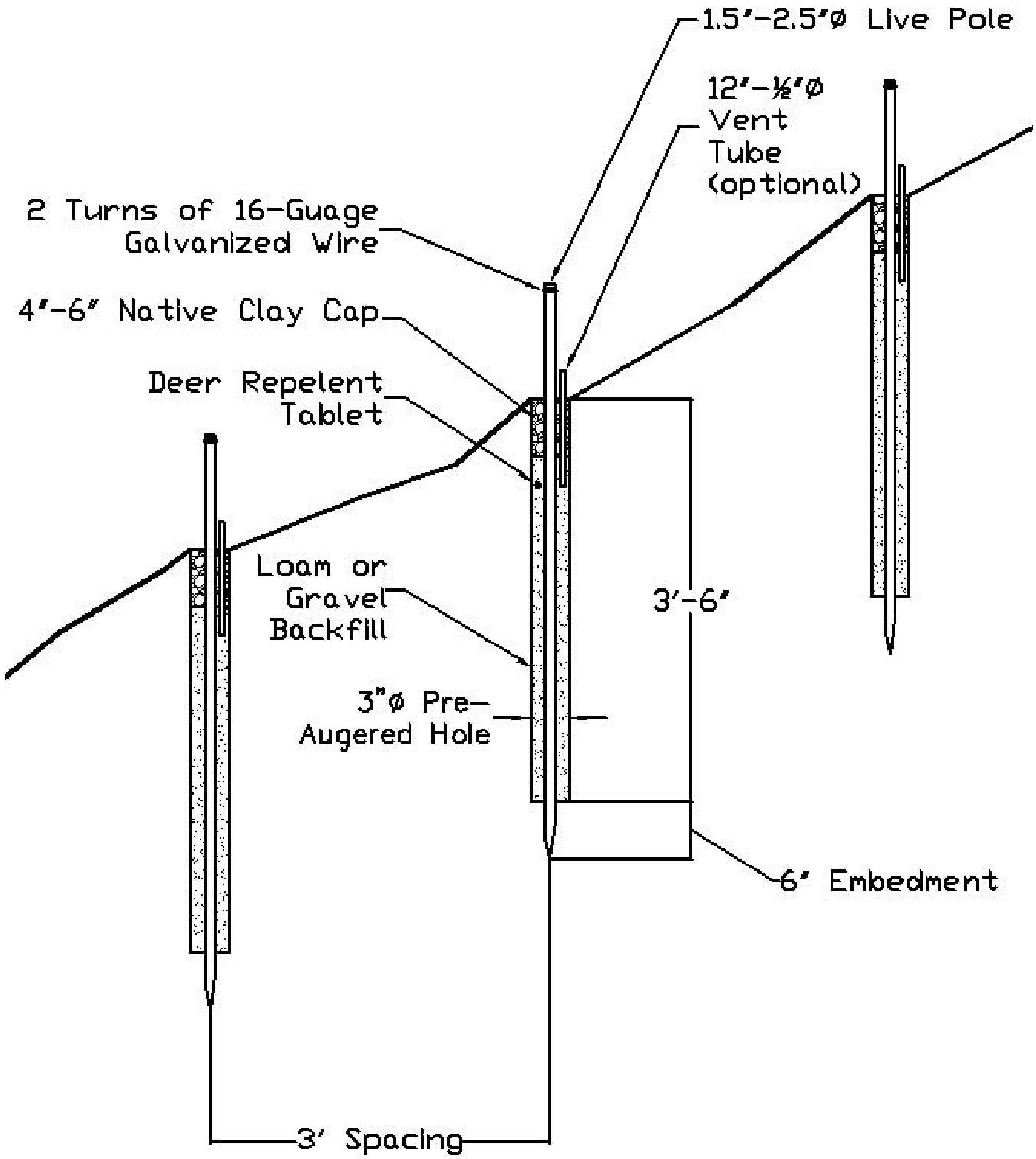


Figure 5.30: Typical live pole installation detail.

5.6.2 Upper Live Pole Plot

The upper live pole plot was completed during the last week of May and the first week of June 2005. In total 63 live poles, dormant hardwood cuttings of willow and poplar measuring approximately five ft. in length and 1 to 2½ in. in diameter were installed on a two ft. grid pattern to stabilize this roughly ten ft. by fifteen ft. section of unstable slope.

Unlike the main live pole plot where the slope was regraded with a dozer to eliminate erosion and landslide features, the poles in the upper plot were installed with the slope "as is" because there were no scarps and drainage was positive. Thus, no special site preparation was used to mitigate the landslide scarps and erosion rills prior to live pole installation. The live poles in this trial section were installed into 3 in. diameter pre-augured holes, which were backfilled with loam and a four to six in. clay cap. A photograph of the installation is shown in Figure 5.31 and the layout of the plot and pole type (i.e., willow or poplar) is shown in Figure 5.32.

Figure 5.31: Muskingum County upper live pole plot during installation near STA 1+00 (June 2005). (see Figure 5.7 for STA reference)

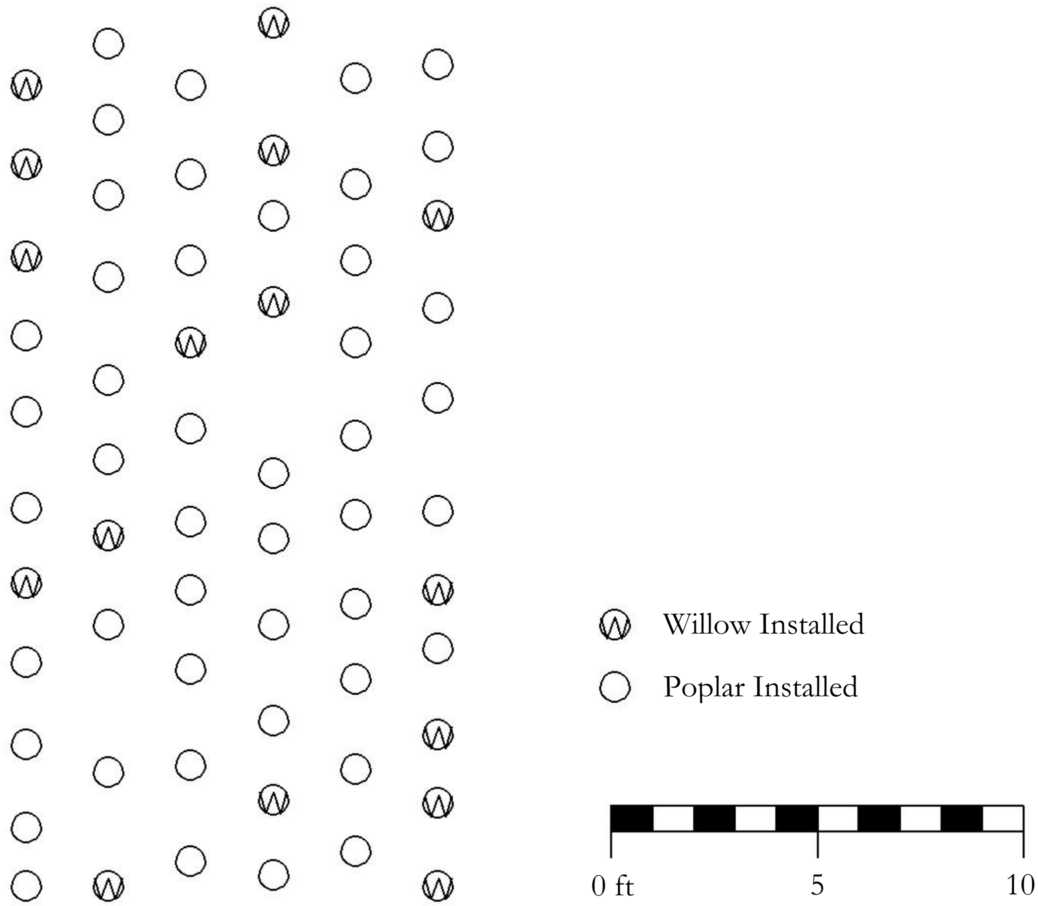


Figure 5.32: Muskingum County upper live pole installation layout.

5.6.3 Brushlayers

Three brushlayer berms were installed during the last week of May 2005 and the first week of June 2005. The brushlayers consist of vegetation placed into the slope face at a slight angle to horizontal (i.e., slightly dipping into the slope). Because the chief intention of a brushlayer berm is provide erosion control rather than improving slope stability, no calculations have been preformed to predict the performance of these installations. Figure 5.33 shows two photographs of the brushlayer installation, one during construction and the other 4½ months later after the brushlayer had taken root.

Figure 5.33: Muskingum County brushlayer installation: Left: Brushlayer construction (6/15/05); Right: Established brushlayer (8/2/05).

5.6.4 Bioengineering Installations Procedure – Harvest to Establishment

The following list outlines the systematic chronology, from harvest to establishment, of the bioengineering installations that were constructed during late spring of 2005 at the Muskingum County demonstration site. The live pole installation was similar to the Best Method (see section 4.5.3.1.1).

1. Harvest (April 2005) – Live pole and brushlayer vegetative material was harvested using chainsaws, lopers, and similar pruning and cutting tools. All of the live pole and brushlayer material used at the Muskingum Co. site was harvested from a drainage ditch located on the property of Honda of America in Marysville, Ohio. 1 to 2½ in. diameter dormant hardwood (willow and poplar) poles were cut to 5 to 6 ft. length. Live pole cuttings were selected to be relatively straight and all of the branches and twigs were removed. Brush layer cuttings were taken from long, branch material. All of the cuttings were bundled into manageable loads using jute twine, submerged in the flowing water of the ditch until transport, and finally, transferred via box truck to storage.
2. Storage (April to May 2005) – Refrigerated storage was used to keep the cuttings in their dormant state prior to installation. The temperature was maintained slightly warmer than freezing during the storage process. In addition to temperature control, the cuttings were kept moist to prevent drying by covering with burlap and routinely spraying with water. Two refrigerated walk-in storage coolers were used during this process, one at Acorn Farms in Zanesville, Ohio, and the other at The Ohio State University Howlett Hall Greenhouse, Columbus, Ohio.

3. Soaking (May 2005) – Prior to installation, the live pole and brushlayer cuttings were soaked in water for a minimum of three days. This was achieved by rotating the bundled vegetation from cold storage to a pond at Acorn Farms as needed.
4. Transportation (May to June 2005) – Using either a box truck or pickup truck, the cuttings were transported from the Acorn Farms' pond to the site. During the transfer, the cuttings were covered with a tarpaulin to prevent drying.
5. Onsite storage (May to June 2005) – Vegetative material was held in a water filled galvanized stock tank prior to installation. The water in the trough was replenished every couple of days because it would become spoiled due the heat at the site.
6. Site preparation (May 2005) – The main live pole plot was graded and the brushlayer sites were excavated with a dozer prior to installation. The location for all of the bioengineering activities were staked and marked. For live pole installation, 3 in. diameter 3½ ft. deep vertical holes were pre-drilled with a gas powered auger. The excavations for the brushlayers were approximately 20 ft. long and 4 to 5 ft. back into the slope at an angle between 10º to 20º.
7. Vegetation preparation (May to June 2005) –
    a. Live pole – Prior to installation, each live pole's dimensions were recorded and the poles were prepared so that the butt ends were shaped to a V-point. Any protruding branches and knobs were trimmed flush, and several turns of 16-gauge wire were secured to the top ends to prevent splitting during installation.

b. Brushlayer – Although some of the smaller live pole stock was used, "as is" branch cuttings served as the primary medium for construction and no special preparatory measures were used.

8. Installation (May to June 2005)

a. Live pole – One live pole was placed into each pre-augered hole.

i. The pole was then hammered firmly about 6 in. into the base of the hole using a sledgehammer.

ii. The annular space between each pole and its respective hole was backfilled with either pea gravel or loam 4 to 6 in. to grade in 6 in. lifts which where tamped with a rod to ensure that all of the void space was filled.

iii. A deer/animal repellant fertilizer tablet was placed near the top of the backfill.

iv. The top 4 to 6 in. of each hole was then capped with the clay cuttings that were recovered during the augering process. A ½ in. diameter 1 ft. length tube was placed in some of the gravel backfilled holes protruding 4 in. above the surface and extending into the gravel fill to permit the circulation of air to the rooting zone.

v. Each pole top was trimmed at a slight angle leaving 1 to 1½ ft. of live pole remaining above the slope face.

vi. The pole tops were rewired to prevent splitting due to desiccation.

vii. Finally, an 18 in. diameter piece of roofing felt was pinned around the base of each pole to prevent competitive plant growth.

b. Brushlayer –

    i. The brush material was placed at 2 to 4 in. spacing in a crisscross overlapping fashion to cover the bottom of the excavated bench.

    ii. Loam was used to cover the brush material and native clay soil was used to fill the remaining space to return the excavation to the pre-existing slope grade.

    iii. The brushlayer branches were trimmed to protrude only 1 to 2 ft. from the slope contour.

9. Maintenance (June 2005) – Examples of the maintenance duties which have been performed at this site are weeding of competitive flora using a commercial herbicide, application of animal repellent sprays, watering, and the placement of geo-jute erosion mat on the surface.

10. Establishment (Summer to Fall 2005) – The bioengineering vegetation began to take root and new growth was observed.

5.7 PLANT GROWTH AND SURVIVABILITY

The survival statistics for the bioengineering vegetation installed during spring 2005 are presented in this section. An inventory of live pole survival was conducted on September 28, 2005, and the results are summarized in Table 5.6. The overall live pole survival rate for the first growing season was 53% (54% for the main plot and 48% for the upper plot). Additionally, a graphical representation of the first growing season pole survival status for the main live pole plot is presented in Figure 5.34 and in Figure 5.35 for the upper live pole plot. Figures 5.34 and 5.35 also show the location and species of poles that were replaced during April 2006 at the main and upper live pole plots, respectively. It is also noted that all three brushlayers were growing and well established during this inventory.

It is likely that the major contributing factors for the low live pole survival rate was the late season (i.e., early June) planting/installation because the early spring window for establishing vegetation had passed and the vegetation did not have the chance to establish the necessary root base to endure the summer months' heat and dryness and drying out and dying during refrigerated storage. The statistics for the first growing season indicated that willow species have a higher survivability than poplar species, 67 versus 44 percent and for the climatic region of central and eastern Ohio. Hence, willow species may be more appropriate than poplar for bioengineering vegetation.

Table 5.6: First growing season live pole survival statistics for the Muskingum County demonstration site (based on survey on September 28, 2005).

	Main Live Pole Plot	Upper Live Pole Plot	Total
Total poles installed	261	62	323
Number of willow poles installed	96	16	112
Number of willow poles alive	63	15	78
Number of willow poles that sprouted and died	8	0	8
Number of willow poles that did not sprout	25	1	26
Number of poplar poles installed	165	46	211
Number of poplar poles alive	78	15	93
Number of poplar poles that sprouted and died	46	15	61
Number of poplar poles that did not sprout	41	16	57
Percent of all poles alive	54.0%	48.4%	52.9%
Percent of all poles that sprouted	74.7%	72.6%	74.3%
Percent of willow poles alive	65.6%	93.8%	69.6%
Percent of willow poles that sprouted	74.0%	93.8%	76.8%
Percent of poplar poles alive	47.3%	32.6%	44.1%
Percent of poplar poles that sprouted	75.2%	65.2%	73.0%

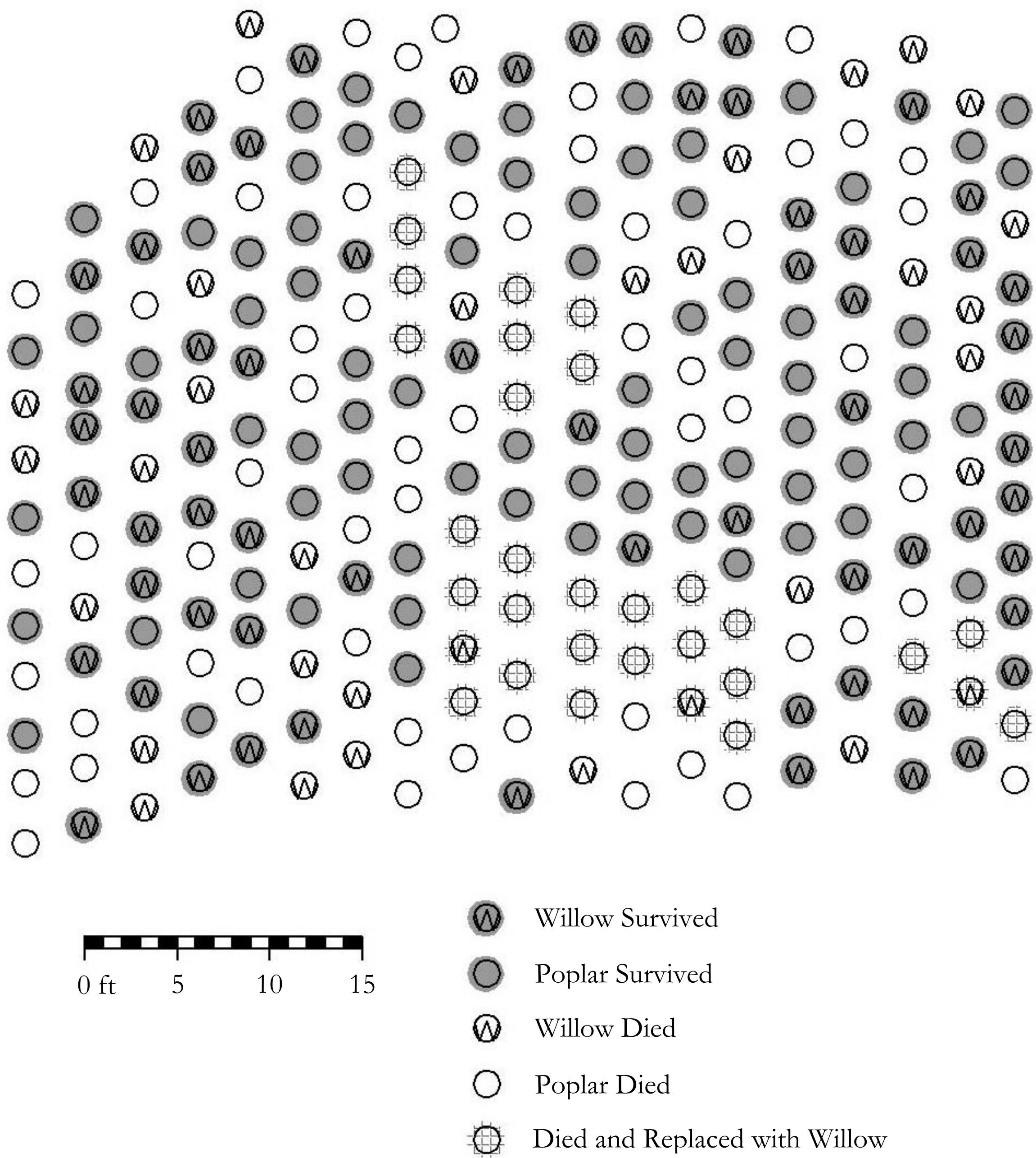


Figure 5.34: Muskingum County main live pole plot showing first season survival data.

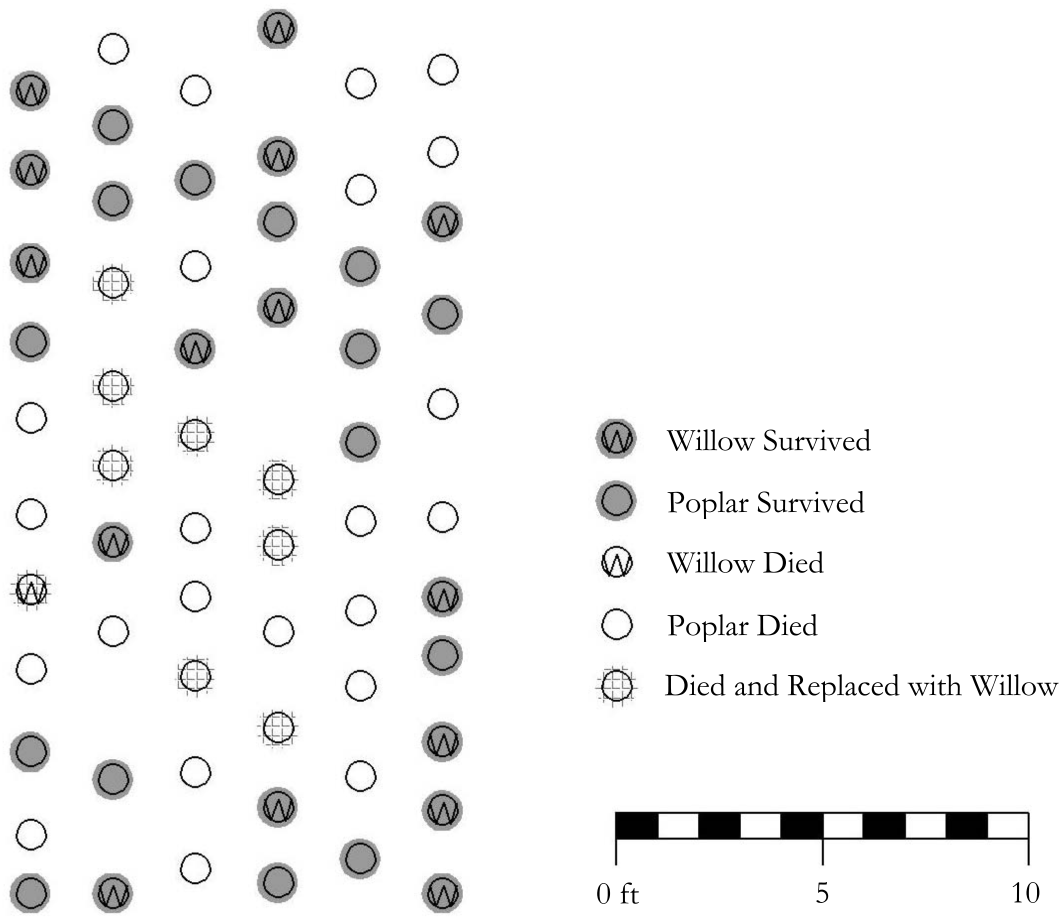


Figure 5.35: Muskingum County upper live pole plot showing first season survival data.

5.8 LIVE POLE VERTICAL PULLOUT TESTS

A device was designed to measure pullout resistance of live poles consisting of a lever with a dynamometer (load cell) connected to the pole as shown in Figure 3.2. Vertical pullout load tests were performed on 30 the dead poles, which were replaced at the Muskingum County site on April 8 and 9, 2006. Figure 5.36 shows the pullout apparatus being used for a live pole pullout test at the site.

The load cell permitted the measurement of the force required to pullout or mobilize the pole in the upward or vertical direction. The force in lb was recorded for each 0.1 ft. of pullout while the live pole was mobilized in the upward direction with the lever. The recorded data including the vertical pullout readings and the pole dimensions (top diameter, bottom diameter, length, and embedment). The pullout test data is tabulated and presented in Appendix B.

Measured vertical resistance ranged from 66 to 700 lb, which corresponds to unit side-friction resistance range from 810 to 8,930 psf with an average unit side-friction resistance equal to 3,100 for all pull-pout tests completed. The measured values for pullout resistance represent the lower bound on shear strength because the tested poles were dead and thus did not have established roots to contribute to the pullout resistance. Using equations 3.1 and 3.2 presented in section 3.7, the estimated undrained shear strength, s_u, for the minimum and maximum pullout tests corresponds to 1,620 psf and 17,860 psf, respectively with the average for all pullout tests completed equal to 6,210 psf.

Figure 5.36: Live pole vertical pullout test (April 2006).

5.9 SUMMARY

In closing, this chapter presented the research efforts completed for the Muskingum County field demonstration site. Throughout this chapter, data and analyses are presented and discussed. Specifically discussed are the site conditions including the geology, climate, and subsurface and groundwater conditions. Additionally, sections outlining field exploration, laboratory testing, and site instrumentation are also presented. The stability analysis and subsequent bioengineering designs that were selected for construction at the Muskingum County I-70 and SR-83 site are presented. Lastly, the results from the first growing season of the installed bioengineering methods is presented and discussed. It is concluded that live willow poles can be effective for stabilization of shallow slides if the vegetation can be established.

CHAPTER 6

CONCLUSIONS

6.1 INTRODUCTION

Presented are conclusions and recommendations pertaining to the objectives of this research: (1) to identify important factors that control success or failure of bioengineering methods, (2) to develop installation techniques and designs for successful application of bioengineering methods, and (3) to provide thorough documentation to develop design guides for future work in bioengineering for ODOT. Also presented are findings relating to site instrumentation, success, failure, and limitations and recommendations for future study.

6.2 CONCLUSIONS

6.2.1 Factors Which Control the Success or Failure of Bioengineering Methods

Although this thesis covers the project in the early stages and only one growing season has been observed/documented at only one of the three demonstration sites, several key factors that control the success or failure of bioengineering methods can be concluded thus far:

1. The construction of bioengineered projects is labor and detail intensive. Paramount to the success of a bioengineered project is that the work crews have specialized training in the handling, transporting, preparation, and

installation of the chosen vegetation and methods. A major setback during the Muskingum County installation was the inexperience of the project team in installing bioengineering vegetation with the basic project logistics and coordination being perhaps the critical factor.

2. Construction must be completed during the dormant season for the vegetation (winter and early spring). Survival of the installed live poles was low at the Muskingum County site, at 53%, and it is likely that the major contributing factors were the negative impact of cold storage in addition to the late season planting/installation. Based on the survivability data at the Muskingum County site and further supported by the literature, it can be concluded that in order to give the vegetation the best chance for survival, taking advantage of early spring window for establishing vegetation before the hot summer months is necessary unless additional water is supplied to the site. At the Muskingum County field site, by the time the vegetation was planted in early June the critical early spring establishment window had passed. It is suggested that the vegetation did not have the chance to establish the necessary root base to endure the summer months' heat and dryness and as a result, low survivability was observed, again, providing water to site would help.
3. Species selection: Willow species had higher survivability than poplar species as observed during the first growing season at the Muskingum County field demonstration site. It is concluded that for the climatic region of central and

eastern Ohio that willow species are considered heartier and more appropriate than poplar for bioengineering designs.

4. Based on field observations, possible drought conditions should be considered during the plant selection and maintenance scheduled for bioengineered projects in Ohio and plans to provide water on site should be considered.

6.2.2 Designs for Successful Application of Bioengineering Methods

1. Stability analysis for this bioengineering project was carried out using traditional geotechnical limit equilibrium approaches, such as, infinite slope and circular slip, where the vegetation's effect on stability was estimated.

6.2.3 Installation Techniques for Successful Application of Bioengineering Methods

1. Cold storage of dormant willow and poplar cuttings was used to delay the installation following harvest. The method used for this project: humid refrigerated storage at slightly warmer than freezing where the cuttings are bundled, wrapped in burlap, and routinely sprayed with water to keep the cuttings from drying out. It is concluded that this cold storage was generally unsuccessful and contributed to the low first season survival rate at the Muskingum County site.
2. Because live cuttings must be kept moist/wet during all stages of handling (i.e., harvesting, transporting, storage, preparation, and installation), maintaining a source of fresh water throughout the process poses a logistic

challenge. Ponds, where available, and mobile galvanized stock tanks provide such sources. Both were used successfully for pre-planting soaking.

6.2.4 Instrumentation for Successful Application of Bioengineering Methods

1. A live pole vertical pullout test method for determining the shear resistance for poles was devised. The method was generally successful.
2. Gypsum moisture blocks and tensiometers results are inconsistent with one another and correlations for soil moisture and pore water pressure were fraught with inconsistencies and scatter.
3. Shallow slope inclinometers were successfully used to measure near surface slope movements. Installations are limited to about 5 ft. depth due to the use of hand tools for exhumation.

6.3 RECOMMENDATIONS FOR FUTURE STUDY

1. A more thorough record of actual labor-hours used during the construction of the demonstration sites would be beneficial for estimating cost-per installation data.
2. It is recommended that site-specific climatic data (i.e., precipitation, temperature, humidity, and sunlight intensity) be recorded because a site's microclimate has a heavy influence on the survivability of the chosen vegetative species. Moreover, this data would be useful in developing species selection criteria for sites across the state.
3. A more rigorous approach to live pole design may be accomplished by modeling the poles as piles and using approaches for latterly loaded piles

(Broms 1964; Broms 1965; Rao et al. 1996; Poulos 1999), and other methods which model the mechanics of small diameter piles and the flow effects in cohesive, clay, soils.

4. The detrimental effects of late planting and refrigerated storage on live pole cuttings could not be differentiated during the Muskingum County trial. It is suggested to investigate the degree that these factors have on the survivability of live pole installation using control sections in future trial pole plots.
5. Because of the low survivability at the Muskingum County site, the validity of the differing installation methods (i.e., backfill medium and method) could not be fully investigated. Additional research into the advantages and disadvantages of soil backfill type, presence of venting tubes, etc. would be valuable to refine and optimize successful live pole planting procedures.

LIST OF REFERENCES

Abramson, L. W., Lee, T. S., Sharma, S., and Boyce, G. M. (2002). *Slope Stability and Stabilization Methods*, John Wiley & Sons, Inc., New York.

Amlin, N. M., and Rood, S. B. (2002). "Comparative tolerances of riparian willows and cottonwoods to water-table decline." *Wetlands*, 22(2), 338-346.

API. (1984). *API Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms*, API.

Barker, D. H. (1986). "Enhancement of slope stability by vegetation." *Ground Engineering*, 19(3), 11-15.

Barker, D. H. (1997). "Live willow pole for slope stabilization on the A249 at Iwade." Project Report PR/CE/133/97, Crowthorne.

Barker, D. H. (2004). "Live Pole Slope Stabilization in the Tropics." Ground and Water Bioengineering for Erosion Control and Slope Stabilization, D. H. Barker, A. J. Watson, S. Sombatpanit, B. Northcutt, and A. R. Maglinao, eds., Science Publishers, Inc., Enfield (NH), USA, 302-308.

Barker, D. H., Watson, A. J., Sombatpanit, S., Northcutt, B., and Maglinao, A. R. (2004). "Ground and Water Bioengineering for Erosion Control and Slope Stabilization." Science Publishers, Inc., Enfield (NH), USA.

Bishop, A. W. (1955). "The use of the slip circle in the stability analysis of slopes." *Geotechnique*, 5, 7-17.

Broms, B. B. (1964). "Lateral resistance of piles in cohesive soils." *Journal of the Soil Mechanics and Foundations Division, ASCE*, 90(SM2 part1), 27-63.

Broms, B. B. (1965). "Design of laterally loaded piles." *Journal of the Soil Mechanics and Foundations Division, ASCE*, 93(SM3 part1), 79-99.

Casagrande, A. "The determination of the pre-consolidation load and its pratical significance." *Int. Conf. Soil Mechanics*, Harvard University.

Chok, Y. H., Kaggwa, W. S., Jaksa, M. B., and Griffiths, D. V. "Modelling the effects of vegetation on stability of slopes." *Proc. 9th Australia New Zealand Conference on Geomechaics*, Auckland, pp. 391-397.

Coduto, D. P. (1999). *Geotechnical Engineering: Principles and Practices*, Prentice-Hall, Inc., Upper Saddle River, NJ.

Coppin, N. J., Greenwood, J. R., Morgan, R. P. C., and Churcher, D. "CIRCA field evaluation and demonstration trials for bioengineering." *Vegetation and Slopes*, Oxford, 127-135.

Coppin, N. J., and Richards, I. G. (1990). "Use of Vegetation in Civil Engineering." Butterworths, Sevenoaks, Kent (England).

Cram, W. H., and Lindquist, C. H. (1983). "Refrigerated storage of hardwood cuttings of willow and poplar." *Tree Planter's Notes*, 33(4), 3-5.

DeLong, R. M. (1996). "Those d----d redbeds." *Ohio Geology*(Summer), 5-6.

Di Pietro, P., and Brunet, G. (2002). "Design considerations related to the performance of erosion control products combined with soil bioengineering techniques." *Geotechnical Testing Journal*, 25(2), 142-147.

Duncan, J. M. (1996). "Soil Slope Stability Analysis." Landslides Investigation and Mitgation Special Report 247, A. K. Turner and R. L. Schuster, eds., TRB.

Duncan, J. R., Hanahan, R. A., Pleasant, M. E., Gangaware, T. R., and Feldman, D. L. "Restoration techniques for urban streams." *Water Resources Engineering*, Memphis, Tenn., 381-386.

Ekanayake, J. C., and Phillips, C. J. (1999). "A method for stability analysis of vegetated hillslopes: an energy approach." *Canadian Geotechnical Journal*, 36, 1172-1184.

Ekanayake, J. C., and Phillips, C. J. (2002). "Slope stability thresholds for vegetated hillslopes: a composite model." *Canadian Geotechnical Journal*, 39, 849-862.

Ekanayake, J. C., and Phillips, C. J. (2003). "Reply to the discusstion by Tien H. Wu on "Slope stability thresholds for vegetated hillslopes: a composite model"." *Canadian Geotechnical Journal*, 40, 1063-1066.

Ekanayake, J. C., Phillips, C. J., and Marden, M. "A comparison of methods for stability analysis of vegetated slopes." *Ground and Water Bioengineering for Erosion Control and Slope Stabilization*, Enfield (NH), USA, 171-181.

Eubanks, C. E., and Meadows, D. (2002). "A Soil Bioengineering Guide for Streambank and Lakeshore Stabilization." USDA.

Finney, K. (1993). "History of soil bioengineering," Masters Thesis, University of Oregon.

Fotherby, L. M., Hoitsma, T. R., and Miller, D. E. "Bioengineered bank stabilizatoin on the Little Miami River." *Water Resources Engineering*, Memphis, Tenn., 340-345.

Fredlund, D. G., and Krahn, J. (1977). "Comparison of slope stability methods of analysis." *Canadian Geotechnical Journal*, 14, 429-439.

Fredlund, D. G., and Rahardjo, H. (1993). *Soil Mechanics for Unsaturated Soils*, Wiley-Interscience.

Goldsmith, W. (2006). "Soil strength reinforcement by plants." IECA.

Gray, D. H. (1994). "Influence of vegetation on the stability of slopes." Vegetation and slopes, D. H. Barker, ed., Thomas Telford, University Museum, Oxford, 2-25.

Gray, D. H., and Leiser, A. T. (1982). *Biotechnical Slope Protection and Erosion Control*, Van Nostrand Reinhold Company, New York.

Gray, D. H., and Sotir, R. B. (1992). "Biotechnical stabilization of highway cut slope." *Journal of Geotechnical Engineering, ASCE*, 118(9), 1395-1409.

Gray, D. H., and Sotir, R. B. (1995). "Biotechnical stabilization of steepened slopes." *Transportation Research Record*, n 1474, 23-29.

Gray, D. H., and Sotir, R. B. (1996). *Biotechnical and Soil Bioengineering Slope Stabilization : A Practical Guide for Erosion Control*, John Wiley & Sons, Inc., New York.

Greenway, D. R. (1987). "Vegetation and Slope Stability." Slope Stability: Geotechnical Engineering and Geomorphology, M. G. Anderson and K. S. Richards, eds., John Wiley & Sons Ltd., Chichester [West Sussex] ; New York, Chapter 6.

Hansen, M. C. (1995). "ODNR geofacts no. 8." *www.dnr.state.oh.us/geosurvey/geo_fact/geo_f08.htm*.

Janbu, N. (1973). "Slope stability computations." Embankment-Dam Engineering: Casagrande Volume, R. C. Hirschfeld and S. J. Pouros, eds., Wiley, Hoboken, NJ, 47-86.

Janbu, N., L., B., and Kjaernsli, B. (1956). "Stabilitetsberegning for fyllinger skjaeringer og naturlige skraninger." Norwegian Geotechnical Publication No. 16, Olso, Norway.

Kirsten, L. H. (2001). "Quantification of the increase in soil strength due to vegetation roots," PhD Thesis, University of the Witwatersrand, Johannesberg.

Krabel, C. J. (1936). "Erosion control on mountain roads." Circular No. 380, US Department of Agriculture, Washington, DC, 43.

Lambe, T. W., and Whitman, R. V. (1969). *Soil Mechanics*, Wiley, New York.

Lewis, L. (2000). "Soil bioengineering: An alternative for roadside management, a practical guide." *Technical Report 007-1801-SDTDC*, United States Department of Agriculture, Forest Service, San Dimas, CA.

Lewis, L., Hagen, S., and Salisbury, S. L. (2001). "Soil bioengineering for upland slope stabilization." *WA-RD 491.1*, Washington State Department of Transportation.

Morgan, R. P. C., and Rickson, R. J. (1995). "Slope Stabilizatoin and Erosion Control: A Bioengineeirng Approach." E & FN Spon, London.

Morgenstern, N. R., and Price, V. E. (1965). "The analysis of the stability of general slip surfaces." *Geotechnique*, 15, 70-93.

Poulos, H. G. (1999). "Design of slope stabilizing piles." Slope Stability Engineering, N. Yagi, T. Yamagami, and J.-C. Jiang, eds., A. A. Balkema, Matsuyama, Shikoku, Japan, 83-100.

Rao, S. N., Rao, K. M., and Veeresh, C. (1996). "A simplified method of calculating the lateral load capacity of rigid piles in clay." *Ground Engineering*, 29(9), 38-40.

Rowe, K. (2005). "Acorn Farms." C. M. Kokesh, ed., Zanesville.

Sadlon, N. P. (1993). "Soil bioengineering: a natural approach to stream bank stabilization." *USGA Green Section Record*, 31(5), 18-19.

Schaff, S. D., Pezeshki, S. R., and F. Douglas Shields, J. (2002). "Effects of pre-planting soaking on growth and survival of black willow cuttings." *Restoration Ecology*, 10(2), 267-274.

Schaff, S. D., Pezeshki, S. R., and F. Douglas Shields, J. (2003). "Effects of soil conditions on survival and growth of black willow cuttings." *Environmental Management*, 31(6), 748-763.

Schiechtl, H. (1980). *Bioengineering for land reclamation and conservation*, The University of Alberta Press, Edmonton, Alberta, Canada.

Schiechtl, H. M., and Stern, R. (1996). *Ground Bioengineering Techniques for Slope Protection and Erosion Control*, Blackwell Science, Oxford.

Schiechtl, H. M., and Stern, R. (1997). *Water Bioengineering Techniques*, L. Jaklitsch, translator, Blackwell Science, Tokyo.

Schmidt, K. M., Roering, J. J., Stock, J. D., Dietrich, W. E., Montgomery, D. R., and Schaub, T. (2001). "The variability of root cohesion as an influence on shallow landslide susceptibility in the Oregon Cost Range." *Canadian Geotechnical Journal*, 38, 995-1024.

Slope Indicator. (2003). "DigiPro for windows." Mukilteo, Washington.

Soilmoisture Equipment Corp. (1997). "2725A operating instructions." Santa Barbara.

Soilmoisture Equipment Corp. (2000). "5201F1 Soilmoisture G-Blocks operation instructions." Santa Barbara.

Sotir, R. B. "Soil bioengieering experiences in North America." *Vegetation and Slopes*, Oxford, 190-201.

Sotir, R. B. "Case histories in soil bioengineering streambank protection." *Water Resources Engineering*, Memphis, Tenn., 334-349.

Sotir, R. B., and Christopher, B. R. (2004). "Sustainable steep vegetated slopes through soil bioengineering." *Geo-Strata - Geo Institute of ASCE*, 5(2), 14, 16-18.

Spencer, E. (1967). "A method of analysis of the stability of embankments assuming parallel inter-slice forces." *Geotechnique*, 17, 11-26.

Steele, D. P., MacNeil, D. J., Barker, D. H., and McMahon, W. (2004). "The use of live willow poles for stabilising highway slopes." *TRL619*, Highways Agency.

Steiger, J. R. (1996). *Soil Survey of Muskingum County, Ohio*.

Stout, W. (1918). *Geology of Muskingum County, Ohio*.

Szymoniak, T., Bell, J. R., thommen, G. R., and Johnsen, E. L. (1984). "A geogrid-reinforced soil wall for landslide correction on the Oregon Coast." *Transportation Research Record*, 965, 47-54.

Taylor, D. W. (1948). *Fundamentals of Soil Mechanics*, John Wiley & Sons, Inc., New York.

Tomlinson, M.J. (1971). "Some Effects of Pile Driving on Skin Friction." *Proc. Conference on Behavior of Piles*, ICE, London, 107-114.

Varnes, D. J. (1958). "Landslide types and processes." Landslides and Engineering Practice, E. B. Eckel, ed., NAS-NRC, Washington, D. C., 20-47.

Volk, T. A., Ballard, B., Robison, D. J., and Abrahamson, L. P. (2004). "Effect of cutting storage conditions during planting operations on the survival and biomass production of four willow (Salix L.) clones." *New Forests*, 28, 63-78.

Waters, D. D. (1979). *Soil Survey of Logan County, Ohio*.

WSDOT. (2003). "Roadside manual." Chapter 740 Soil Bioengineering.

Wu, T. H. (1984). "Effect of vegetation on slope stability." *Transportation Research Record*, 965, 37-46.

Wu, T. H. (1995). "Slope Stabilization." Slope Stabilization and Erosion Control: A Bioengineering Approach, R. P. C. Morgan and R. J. Rickson, eds., E & FN Spon, London, Chaper 7.

Wu, T. H. (2003). "Discusstion of "Slope stability thresholds for vegetated hillslopes: a composite model"." *Canadian Geotechnical Journal*, 40, 1060-1062.

Wu, T. H., III, W. P. M., and Swanston, D. N. (1979). "Strength of tree roots and landslides on Prince of Wales Island, Alaska." *Canadian Geotechnical Journal*, 35(4), 19-33.

Wu, T. H., Tandolph, B. W., and Huang, C.-S. (1993). "Stability of shale embankments." *Journal of Geotechnical Engineering, ASCE*, 119(1), 127-146.

Wu, T. H., Trenner, B. R., Fox, P. J., Kokesh, C. M., Beach, K., and Barker, D. H. "Soil-bioengineering for slope stabilization in Ohio." *GeoCongress 2008*, New Orleans, 883-898.

Wu, T. H., and Watson, A. (1998). "In situ shear tests of soil blocks with roots." *Canadian Geotechnical Journal*, 35(4), 579-590.

Wu, T. H., and Watson, A. (1999). "Soil-root interaction and slope stability." First Asia-Pacific Conference On Ground and Water Bioengineering for Erosion Control and Slope Stability, International Erosion Control Association, Manila, The Philippines.

Wu, T. H., Williams, R. L., Lynch, J. E., and Kulatilake, P. H. S. W. (1987). "Stability of slopes in red conemaugh shale of Ohio." *Journal of Geotechnical Engineering, ASCE*, 113(3), 248-864.

APPENDIX A

DATA AND ANALYSES: LOGAN COUNTY US-33/SR-347

APPENDIX A CONTENTS

Climate Data for Station: 330563 BELLEFONTAINE, OH - Source: MRCC/NOAA

Precipitation Summary
1971-2000 Normals

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
Precip (in)	2.28	2.02	2.70	3.46	4.02	4.11	3.93	3.64	2.78	2.46	3.08	2.94	37.42

Precipitation Summary
1998-2007 Monthly (in inches)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
1998	4.05	2.17	2.73	5.32	2.28	5.42	4.92	3.75	1.03	2.58	1.17	2.23	37.65
1999	3.56	3.52	1.60	4.62	1.26	2.29	6.79	3.37	1.26	1.93	1.85	2.41	34.46
2000	2.85	2.63	1.79	3.84	5.62	6.29	2.17	6.04	5.06	1.60	2.00	2.65	42.54
2001	0.94	1.64	1.25	3.83	6.97	4.26	3.37	4.29	4.18	5.22	2.64	2.88	41.47
2002	1.73	1.91	3.46	8.43	3.92	3.28	1.79	2.05	4.47	2.13	3.58	3.41	40.16
2003	1.75	3.33	2.99	2.56	5.71	3.71	10.13	5.61	6.23	2.76	3.90	3.45	52.13
2004	3.56	0.93	2.57	1.34	5.63	8.54	4.82	3.47	1.52	2.08	4.30	4.57	43.33
2005	10.88	1.56	2.88	5.14	2.26	3.63	4.42	5.02	5.84	3.77	4.26	1.59	51.25
2006	2.92	2.14	2.25	2.46	3.62	7.22	4.47	3.12	2.81	4.90	1.88	3.89	41.68
2007	5.34	3.33	5.90	3.79	1.35	3.10	3.46	6.39	1.42	2.40	2.67	3.86	43.01

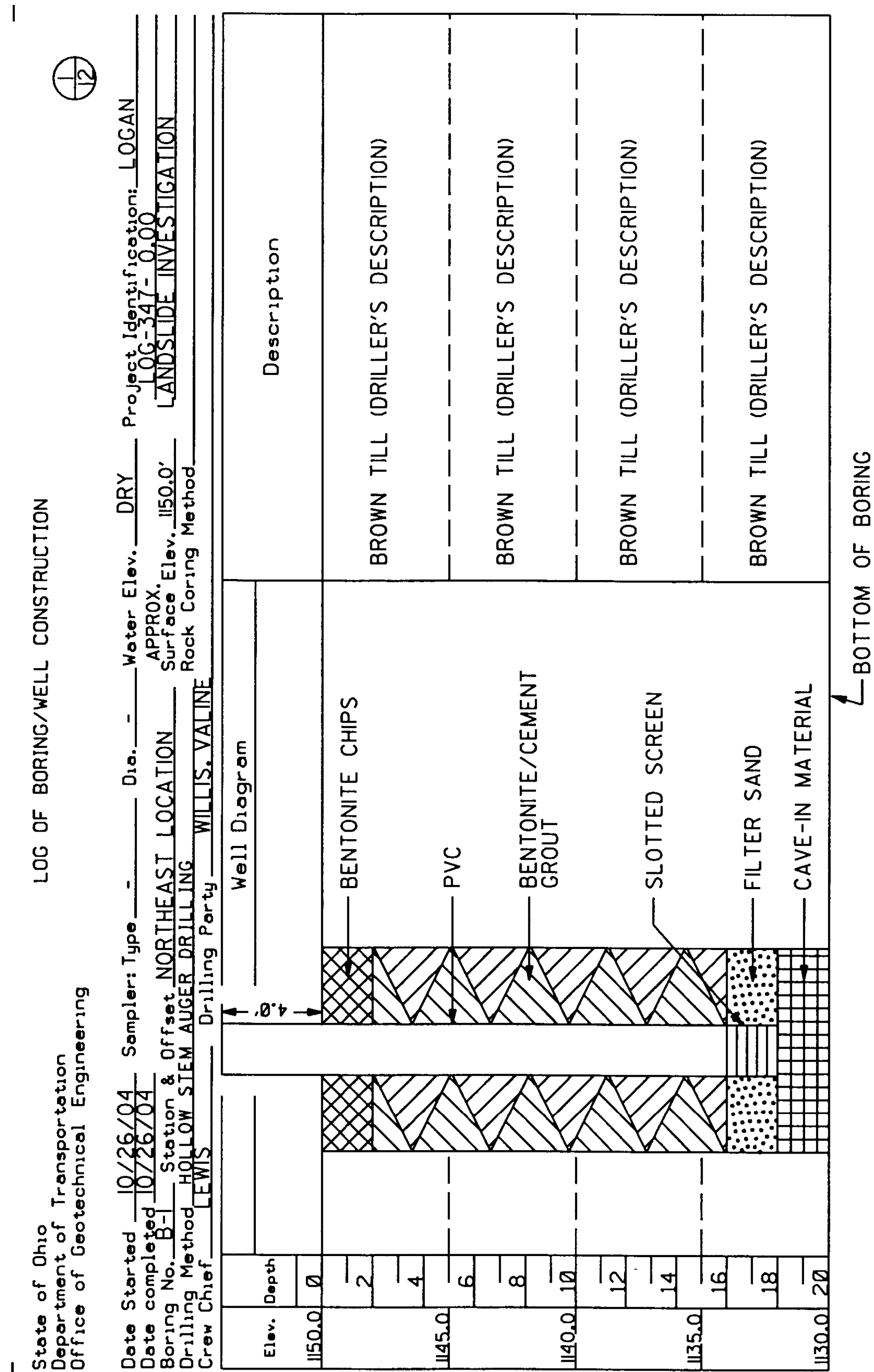
Climate Data for Station: 330563 BELLEFONTAINE, OH - Source: MRCC/NOAA

Temperature Summary
1971-2000 Normals

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
Max °F	32.2	37.0	47.9	59.6	70.8	79.3	83.1	81.0	75.2	63.2	49.4	37.6	59.7
Min °F	15.4	19.1	28.1	38.2	49.6	58.4	62.2	60.5	53.3	41.6	32.0	21.6	40.0
Mean °F	23.8	28.1	38.0	48.9	60.2	68.9	72.7	70.8	64.3	52.4	40.7	29.6	49.9

Growing Season Summary
Derived from 1971-2000 Averages

Base Temp °F	Date of Last Spring Occurrence Median	Early	90%	10%	Late	Date of First Fall Occurrence Median	Early	90%	10%	Late
32	5/1	4/8	4/11	5/15	5/26	10/10	9/23	9/27	10/25	11/4
30	4/23	4/6	4/9	5/8	5/13	10/19	9/22	10/2	11/3	11/6
28	4/11	3/23	4/3	5/1	5/7	10/22	10/2	10/7	11/12	11/22
24	4/7	3/16	3/23	4/16	4/23	11/6	10/14	10/18	11/22	11/29
20	3/25	2/28	3/10	4/11	4/18	11/18	10/24	11/4	12/11	12/18
16	3/17	2/7	2/25	4/2	4/9	11/30	11/4	11/9	12/17	12/22



Log of piezometer P-1 (boring B-1)

State of Ohio
Department of Transportation
Office of Geotechnical Engineering

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LOG OF BORING

Date Started 10/26/04 Sampler: Type SS Dia. ____ Water Elev. 1123.0' Project Identification: LOGAN
Date completed 10/26/04 Approx. LOG-347- 0.00
Boring No. B-2 Station & Offset NORTHEAST LOCATION Surface Elev. 1150.0' LANDSLIDE & MONITORING WELL INSTALL.

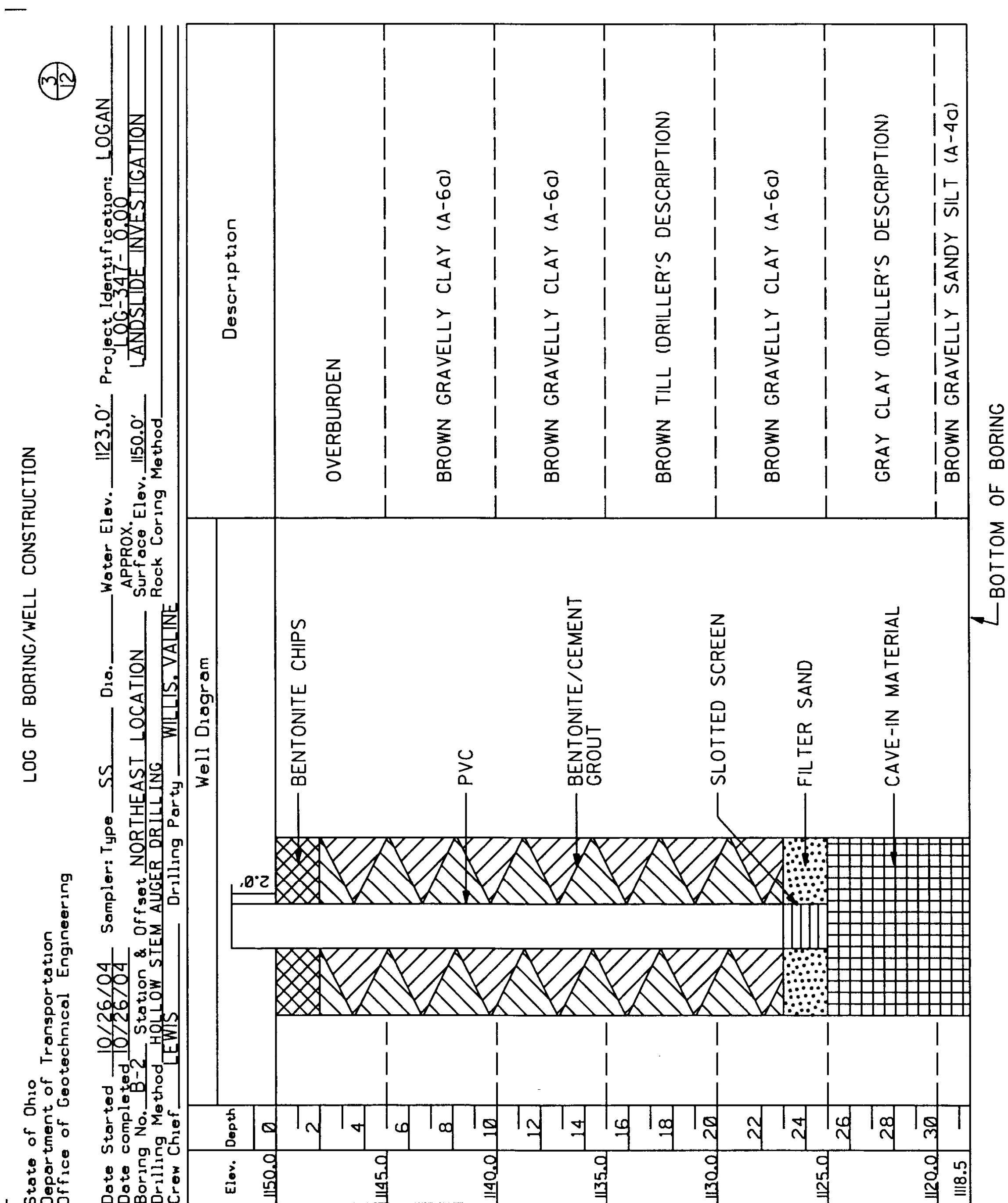
Elev.	Depth	Std. Pen./ R.Q.D.	Rec. ft	Loss ft	Description	Sample No.	Physical Characteristics % Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
1150.0	0														
	2														
	4														
1145.0	6	12/15/17			BROWN GRAVELLY CLAY	16	15	4	7	29	45	34	13	15	A-6a
	8														
1140.0	10														
	12	8/11/17			BROWN GRAVELLY CLAY	17	22	4	6	25	43	32	12	16	A-6a
	14														
1134.9	16	P.S.			BROWN TILL (DRILLER'S DESCRIPTION) NOTE: SAMPLE TESTED BY OSU	18-P	-	-	-	-	-	-	-	-	VISUAL
	18														
1130.0	20														
	22	11/13/19			BROWN GRAVELLY CLAY	19	20	5	7	28	40	32	12	13	A-6a
	24														
1125.0	26	P.S.			GRAY CLAY (DRILLER'S DESCRIPTION) NOTE: SAMPLE TESTED BY OSU	20-P	-	-	-	-	-	-	-	-	VISUAL
	28														
1120.0	30														
1118.5		8/11/8			BROWN GRAVELLY SANDY SILT	21	22	14	11	44	9	18	2	12	A-4a

BOTTOM OF BORING

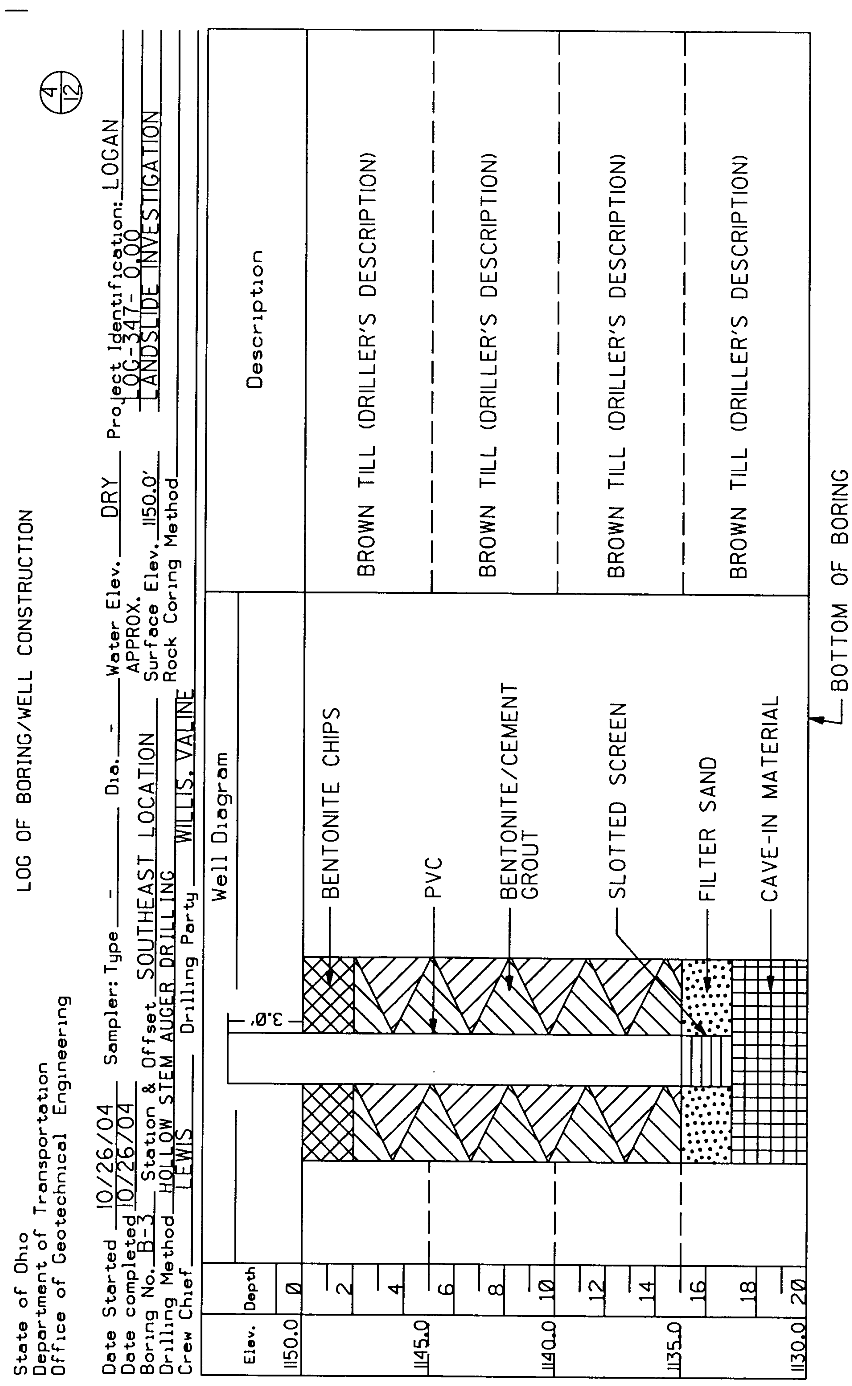
Particle Sizes: Agg= >2.00mm, Coarse Sand= 2.00-0.42mm, Fine Sand= 0.42-0.074mm, Silt= 0.074-0.005mm, Clay= <0.005mm

Form TE-15 Revised 9/95

Log of boring B-2



Log of piezometer P-2 (boring B-2)



Log of piezometer P-3 (boring B-3)

State of Ohio
Department of Transportation
Office of Geotechnical Engineering

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LOG OF BORING

Date Started 10/26/04 Sampler: Type SS Dia. ______ Water Elev. 1124.0' Project Identification: LOGAN
Date completed 10/26/04 Approx. LOG-347- 0.00
Boring No. B-4 Station & Offset SOUTHEAST LOCATION Surface Elev. 1150.0' LANDSLIDE & MONITORING WELL INSTALL.

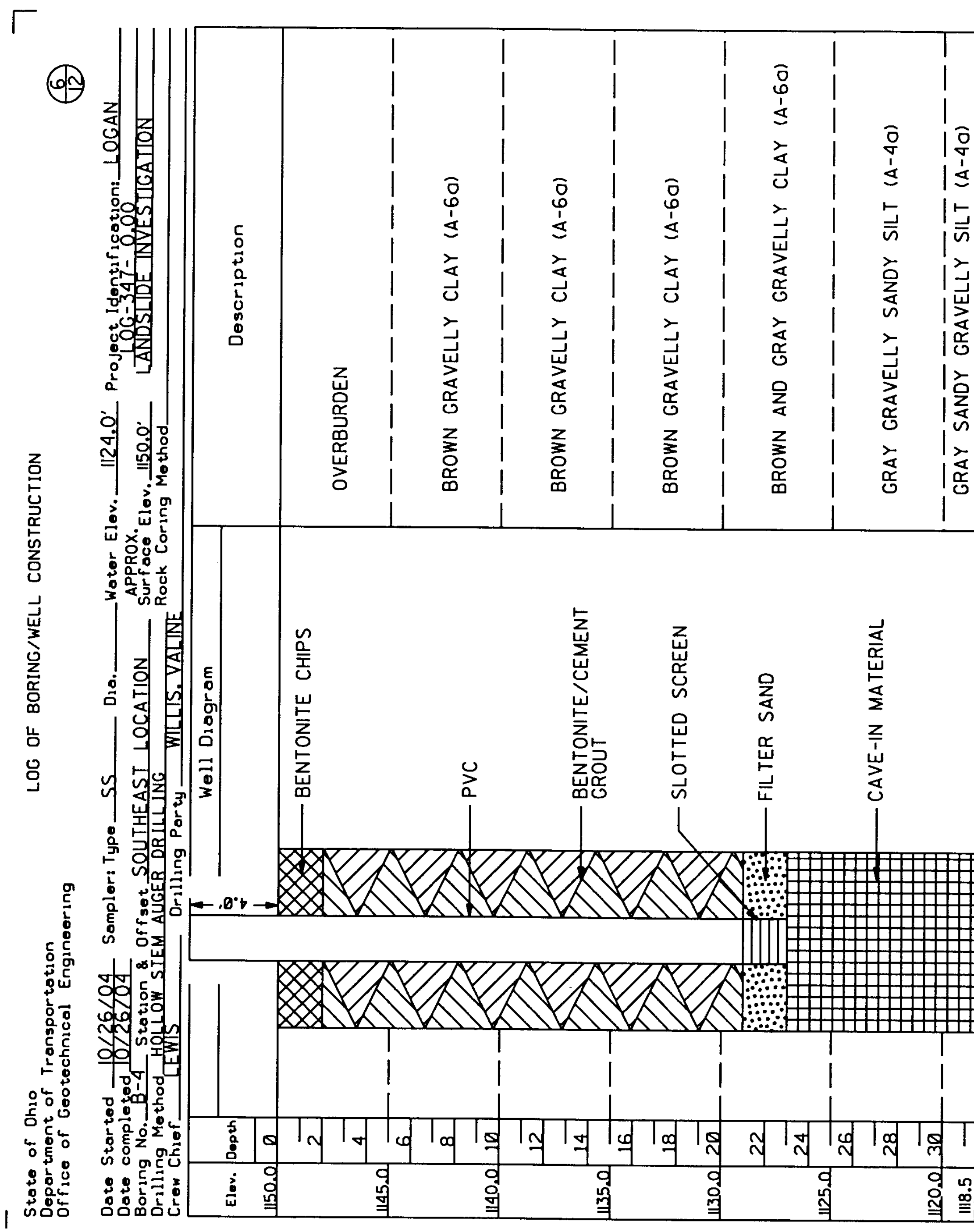
Elev.	Depth	Std. Pen./ R.O.D.	Rec. ft	Loss ft	Description	Sample No.	Physical Characteristics								ODOT Class
							% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	
1150.0	0														
	2														
	4														
1145.0															
	6														
		12/16/21			BROWN GRAVELLY CLAY	10	12	4	7	30	47	32	12	14	A-6a
	8														
1140.0	10														
	12	9/15/17			BROWN GRAVELLY CLAY	11	19	3	6	28	44	35	15	18	A-6a
	14														
1135.0															
	16														
		10/12/13			BROWN GRAVELLY CLAY	12	15	6	8	29	42	33	13	18	A-6a
	18														
1130.0	20														
	22	7/12/14			BROWN AND GRAY GRAVELLY CLAY	13	15	6	8	29	42	30	11	16	A-6a
	24														
1125.0															
	26														
		3/3/3			GRAY GRAVELLY SANDY SILT	14	26	11	16	29	18	21	5	13	A-4a
	28														
1120.0	30														
1118.5		4/8/9			GRAY SANDY GRAVELLY SILT	15	29	14	12	28	17	20	4	10	A-4a

BOTTOM OF BORING

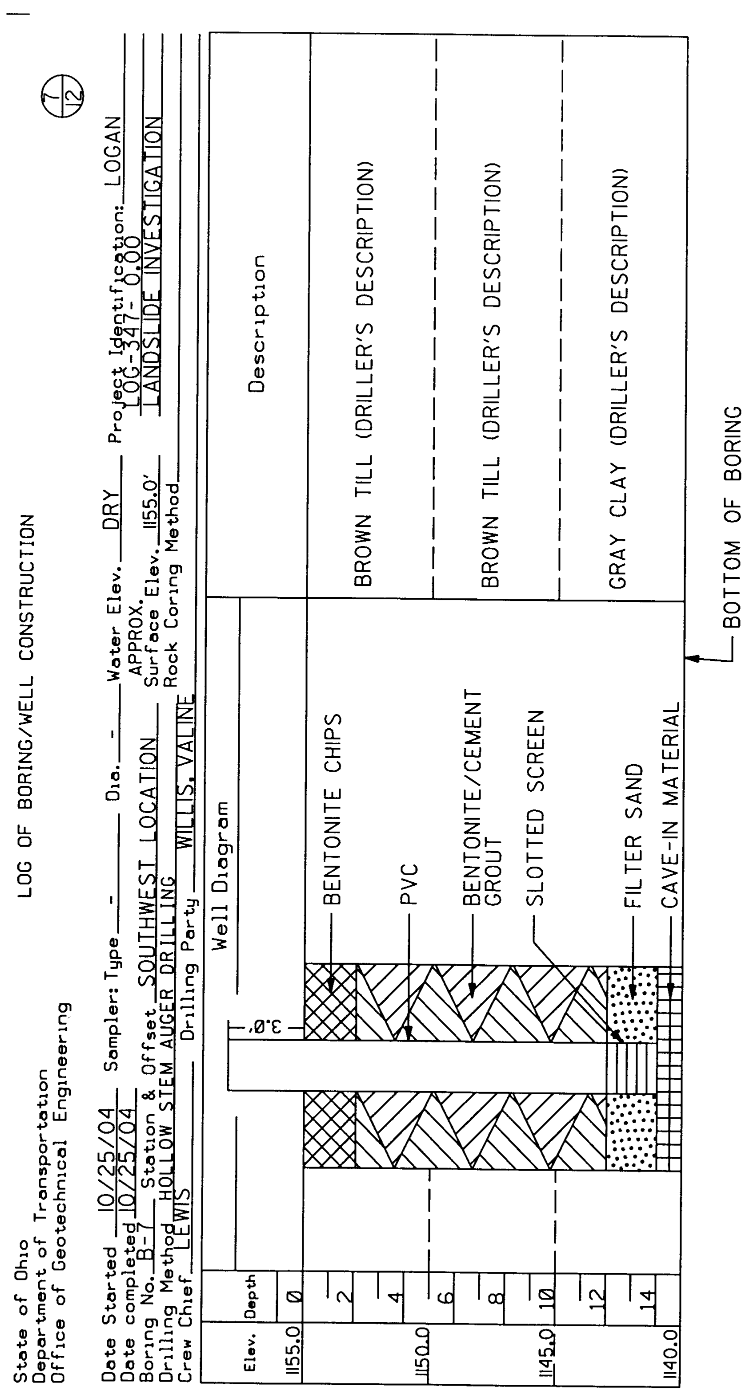
Particle Sizes: Agg= >2.00mm, Coarse Sand= 2.00-0.42mm, Fine Sand= 0.42-0.074mm, Silt= 0.074-0.005mm, Clay= <0.005mm

Form TE-15 Revised 9/95

Log of boring B-4



Log of piezometer P-4 (boring B-4)



Log of piezometer P-7 (boring B-7)

State of Ohio
Department of Transportation
Office of Geotechnical Engineering

8/12

LOG OF BORING

Date Started 10/25/04 Sampler: Type SS Dia. ____ Water Elev. 1135.0' Project Identification: LOGAN
Date completed 10/25/04 Approx. LOG-347- 0.00
Boring No. B-8 Station & Offset SOUTHWEST LOCATION Surface Elev. 1155.0' LANDSLIDE & MONITORING WELL INSTALL.

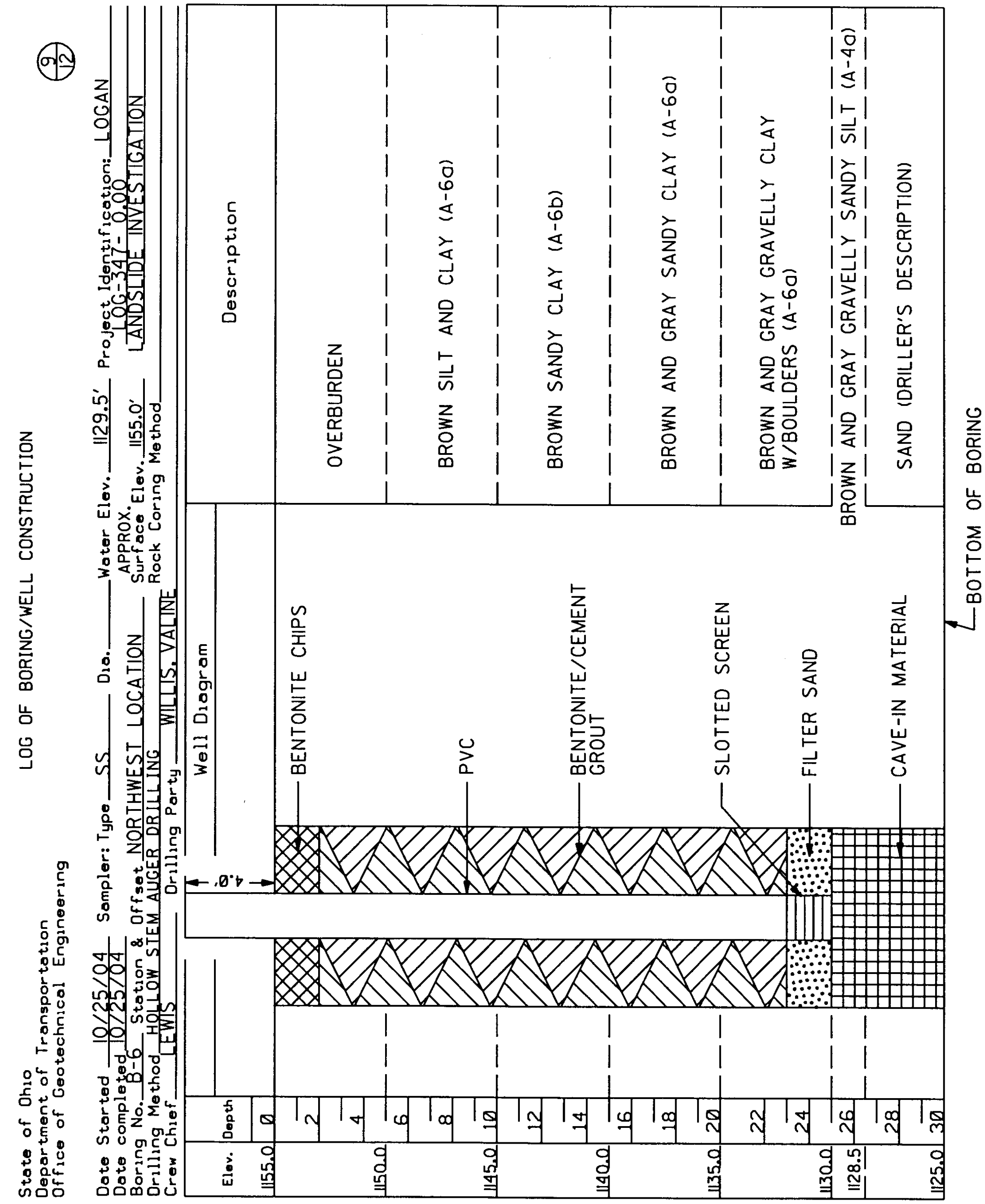
Elev. 1155.0	Depth 0	Std. Pen./ R.Q.D.	Rec. ft	Loss ft	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
	2														
	4														
1150.0	6	P.S.			BROWN CLAY (DRILLER'S DESCRIPTION) NOTE: SAMPLE TESTED BY OSU	6-P	-	-	-	-	-	-	-	-	VISUAL
	8														
1145.0	10														
	12	7/10/13			GRAY GRAVELLY CLAY	7	14	5	7	30	44	31	14	17	A-6a
	14														
1140.0	16	7/8/10			GRAY GRAVELLY CLAY	8	14	5	7	33	41	30	12	16	A-6a
	18														
1135.0	20														
	22	P.S.			BROWN CLAY (DRILLER'S DESCRIPTION) NOTE: SAMPLE TESTED BY OSU	9-P	-	-	-	-	-	-	-	-	VISUAL
	24														
1130.0															

BOTTOM OF BORING

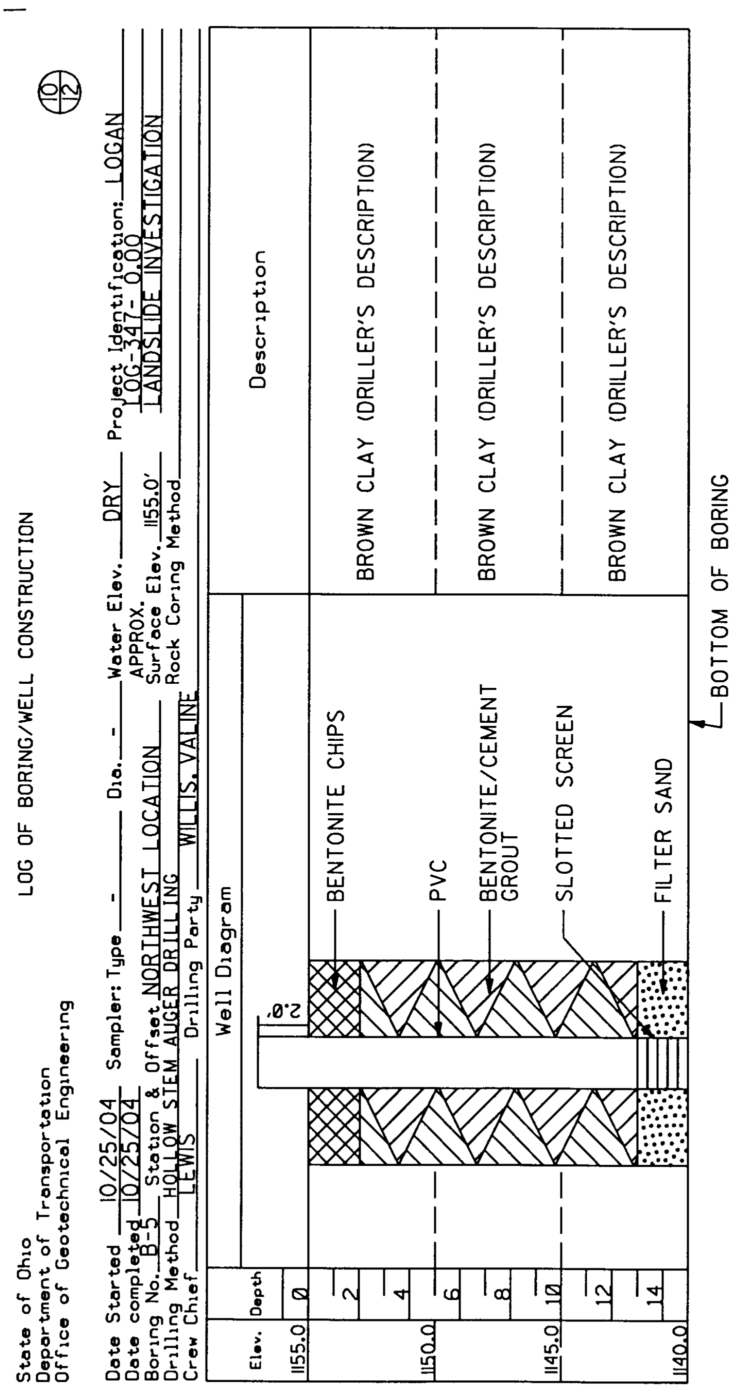
Particle Sizes: Agg= >2.00mm, Coarse Sand= 2.00-0.42mm, Fine Sand= 0.42-0.074mm, Silt= 0.074-0.005mm, Clay= <0.005mm

Form TE-15 Revised 9/95

Log of boring B-8



Log of piezometer P-6 (boring B-6)



Log of piezometer P-5 (boring B-5)

State of Ohio
Department of Transportation
Office of Geotechnical Engineering

LOG OF BORING

11/12

Date Started 10/25/04 Sampler; Type SS Dia. ____ Water Elev. 1129.5' Project Identification: LOGAN
Date completed 10/25/04 Approx. LOG-347- 0.00
Boring No. B-6 Station & Offset NORTHWEST LOCATION Surface Elev. 1155.0' LANDSLIDE & MONITORING WELL INSTALL.

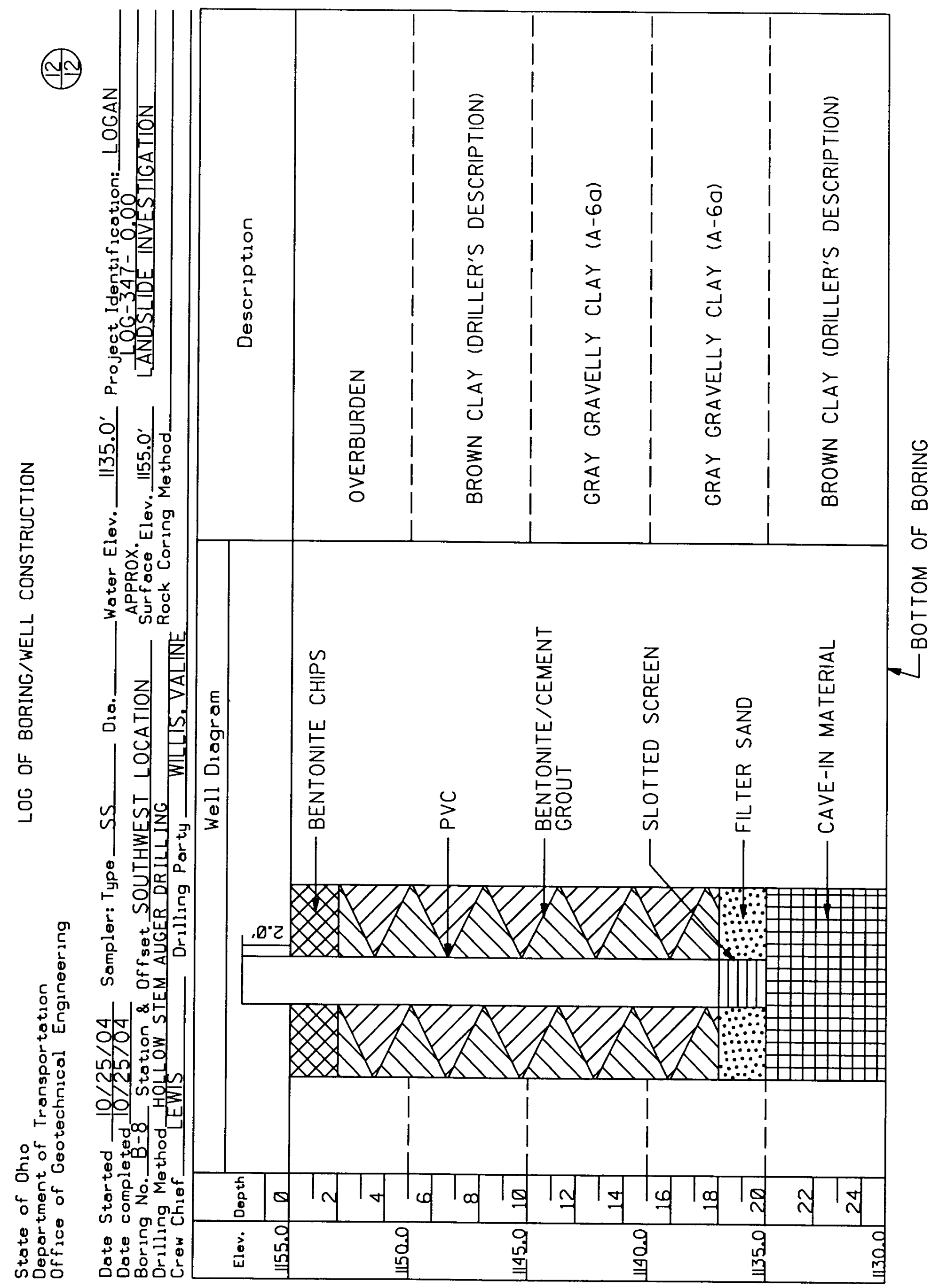
Elev.	Depth	Std. Pen./ R.Q.D.	Rec. ft	Loss ft	Description	Sample No.	Physical Characteristics % Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
1155.0	0														
	2														
	4														
1150.0	6	11/14/16			BROWN SILT AND CLAY	1	8	4	7	30	51	34	14	18	A-6a
	8														
1145.0	10														
	12	7/12/13			BROWN SANDY CLAY	2	11	5	7	31	46	31	12	17	A-6b
	14														
1140.0	16	5/7/11			BROWN AND GRAY SANDY CLAY	3	13	5	8	30	44	30	12	16	A-6a
	18														
1135.0	20														
	22	10/8/8			BROWN AND GRAY GRAVELLY CLAY W/BOULDERS	4	39	4	6	21	30	30	12	17	A-6a
	24														
1130.0	26	9/6/4			BROWN AND GRAY GRAVELLY SANDY SILT	5	16	4	13	41	26	20	4	24	A-4a
	28				SAND (DRILLER'S DESCRIPTION)	-	-	-	-	-	-	-	-	-	VISUAL
1125.0	30														

BOTTOM OF BORING

Particle Sizes: Agg= >2.00mm, Coarse Sand= 2.00-0.42mm, Fine Sand= 0.42-0.074mm, Silt= 0.074-0.005mm, Clay= <0.005mm

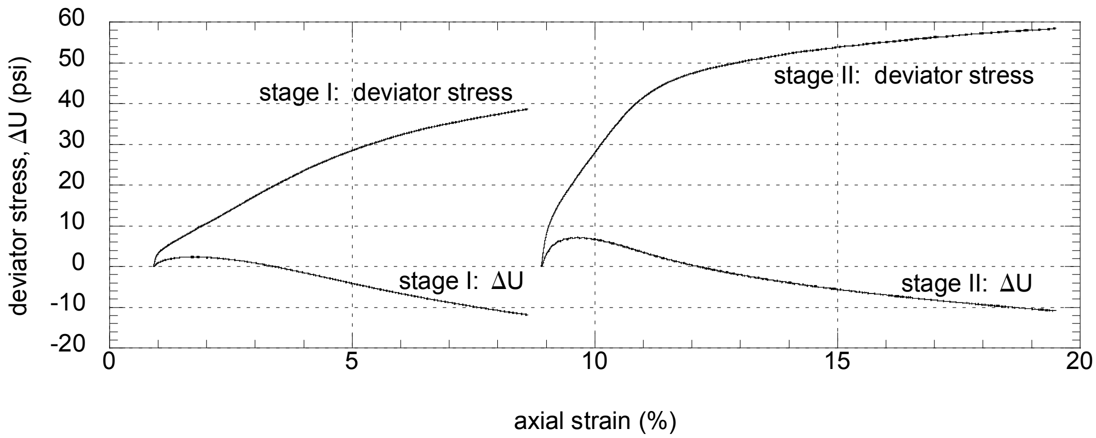
Form TE-15 Revised 9/95

Log of boring B-6



Log of piezometer P-8 (boring B-8)

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY



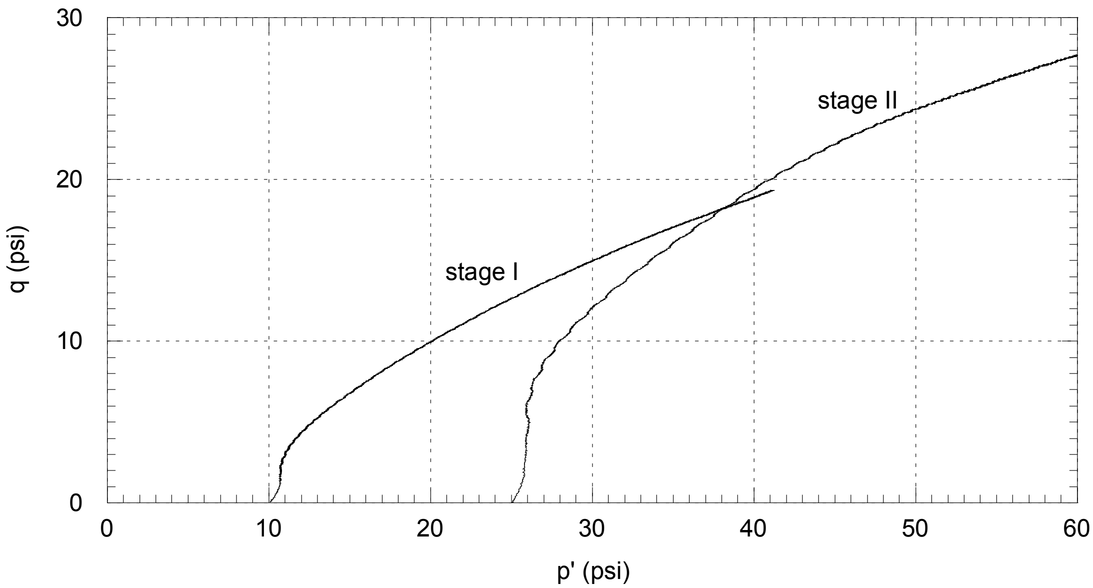
INITIAL	SPECIMEN WEIGHT	W_0 [g]	195.77	MULTI STAGE LOADING		I	II
	SPECIMEN HEIGHT	H_0 [in]	3.25	CELL PRESSURE	σ_{cell} [psi]	25	40
	SPECIMEN DIAMETER	D_0 [in]	1.46	BACK PRESSURE	u_0 [psi]	15	15
	WATER CONTENT	w_0 [%]	15.9	AXIAL STRAIN RATE	[in/min]	0.002	0.002
FINAL	SPECIMEN WEIGHT	W_f [g]	198.23	MAX DEVIATOR STRESS	$(\sigma_1 - \sigma_3)_{max}$ [psi]	38.68	58.51
	SPECIMEN HEIGHT	H_f [in]	2.81	AXIAL STRAIN, $(\sigma_1 - \sigma_3)_{max}$	ε_f [%]	8.62	19.50
	SPECIMEN DIAMETER	D_f [in]	1.54	MAX EFF. STRESS RATIO	$(\sigma_1'/\sigma_3')_{max}$	3.06	2.96
	WATER CONTENT	w_f [%]	17.3	AXIAL STRAIN, $(\sigma_1'/\sigma_3')_{max}$	ε_f [%]	4.10	11.46

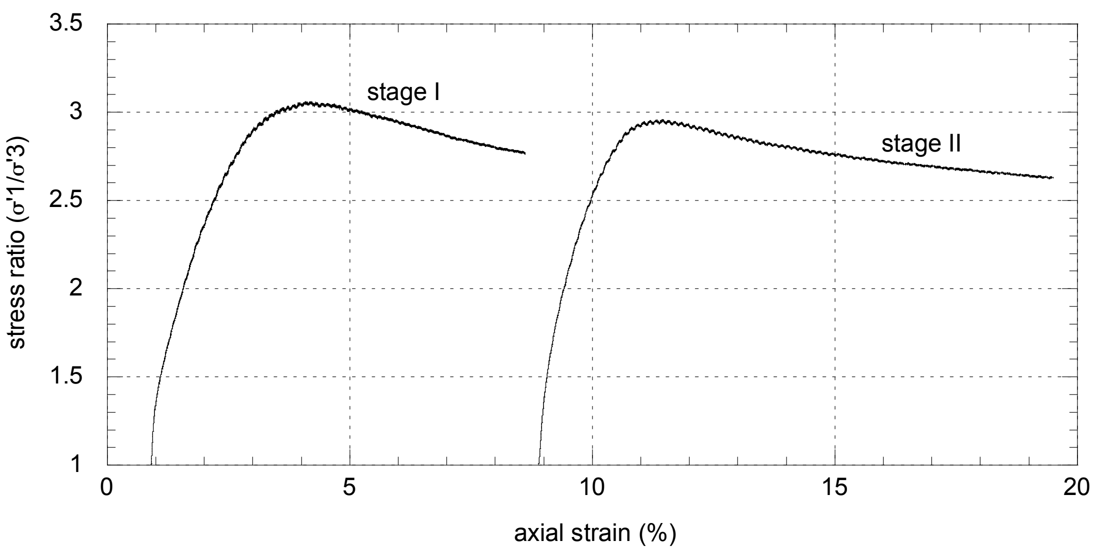
		STAGE I			STAGE II		
		INITIAL	AFTER CONSOLID.	AFTER SHEAR	INITIAL	AFTER CONSOLID.	AFTER SHEAR
DRY UNIT WEIGHT	γ_d [pcf]	118.23	121.50	←	121.50	122.49	←
VOID RATIO	e	0.4303	0.3918	←	0.3918	0.3805	←
WATER CONTENT	w [%]	15.9	14.46	←	14.46	14.04	←
SATURATION	S [%]	100.1	100.0	←	100.0	100.0	←

DESCRIPTION	brown clay with stones (visual)		
CLASSIFICATION	A-6a (ODOT class.)	TYPE OF SPECIMEN	undisturbed
LL 33 / PL 19 / PI 14	G_S 2.71*	TYPE OF TEST	CU

REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
*G_s assumed	AREA	Logan County—SR33/SR347		
B-value = 0.89	BORING ID	BH 2	SAMPLE NO.	18-P 1 bottom
	DEPTH/ELEV.	(15.0-17.0) ft	DATE	May 4, 2005
	TECH	CMK	CHECKED	
1 of 2	**TRIAXIAL COMPRESSION TEST REPORT**			

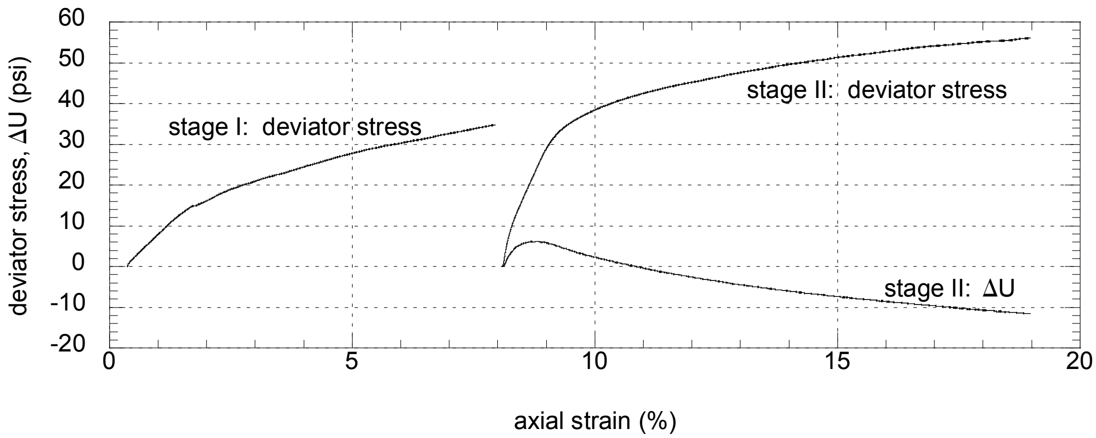
THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY





REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
	AREA	Logan County—SR33/SR347		
	BORING ID	BH 2	SAMPLE NO.	18-P 1 bottom
	DEPTH/ELEV.	(15.0-17.0) ft	DATE	May 4, 2005
	TECH	CMK	CHECKED	
2 of 2	**TRIAXIAL COMPRESSION TEST REPORT**			

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY



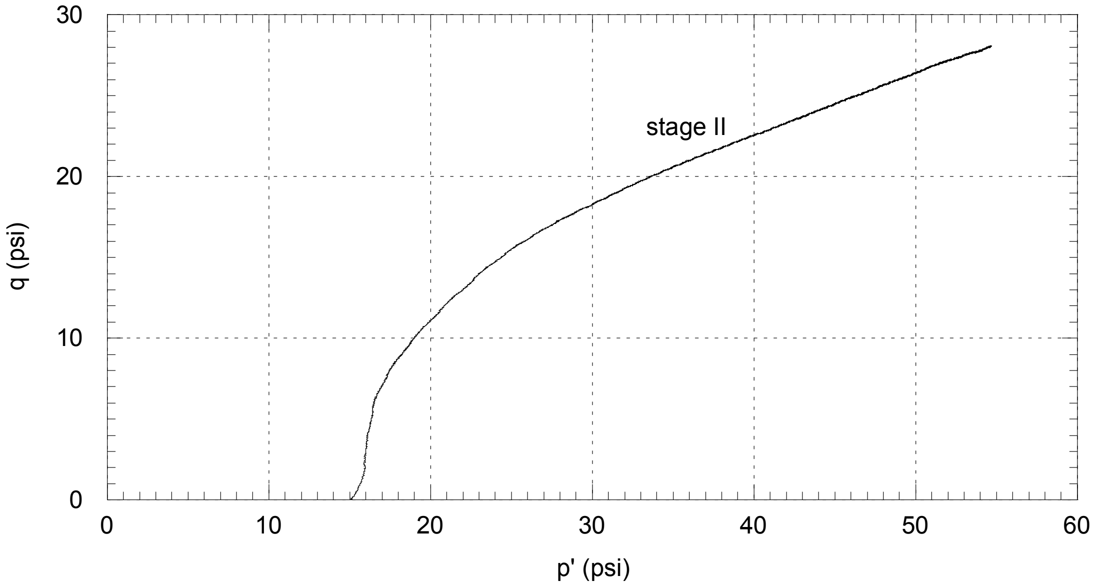
INITIAL	SPECIMEN WEIGHT	W_0 [g]	165.16	MULTI STAGE LOADING		I	II
	SPECIMEN HEIGHT	H_0 [in]	2.75	CELL PRESSURE	σ_{cell} [psi]	25	35
	SPECIMEN DIAMETER	D_0 [in]	1.46	BACK PRESSURE	u_0 [psi]	20	20
	WATER CONTENT	w_0 [%]	16.23	AXIAL STRAIN RATE	[$^{in}/_{min}$]	0.002	0.002
FINAL	SPECIMEN WEIGHT	W_f [g]	165.16	MAX DEVIATOR STRESS	$(\sigma_1 - \sigma_3)_{max}$ [psi]	34.81	56.15
	SPECIMEN HEIGHT	H_f [in]	2.25*	AXIAL STRAIN, $(\sigma_1 - \sigma_3)_{max}$	ε_f [%]	7.93	18.96
	SPECIMEN DIAMETER	D_f [in]	1.60*	MAX EFF. STRESS RATIO	$(\sigma_1'/\sigma_3')_{max}$	**	4.29
	WATER CONTENT	w_f [%]	16.2	AXIAL STRAIN, $(\sigma_1'/\sigma_3')_{max}$	ε_f [%]	**	9.24

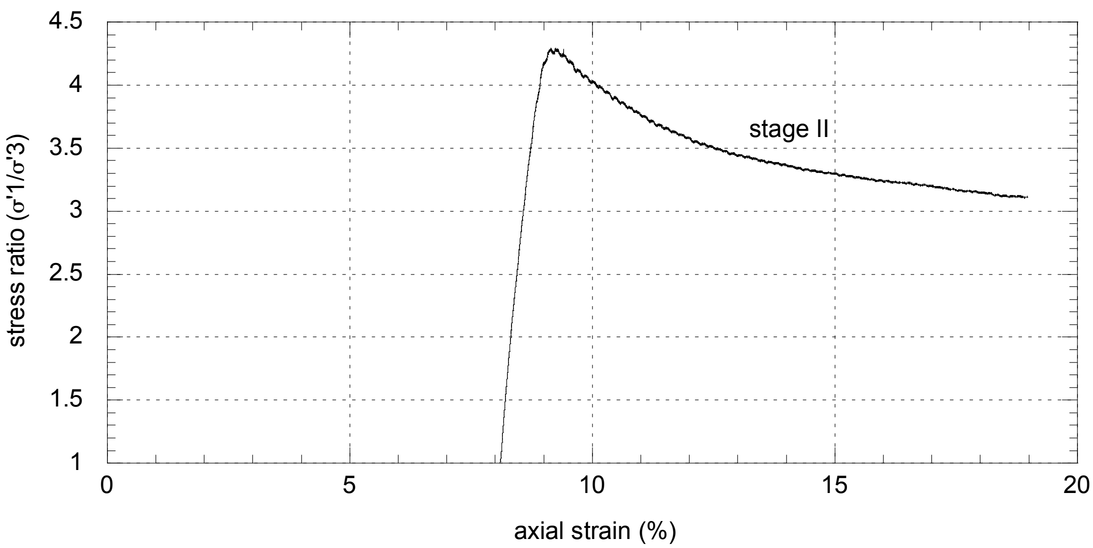
		STAGE I			STAGE II		
		INITIAL	AFTER CONSOLID.	AFTER SHEAR	INITIAL	AFTER CONSOLID.	AFTER SHEAR
DRY UNIT WEIGHT	γ_d [pcf]	117.55	118.81	←	118.81	119.62	←
VOID RATIO	e	0.4385	0.4233	←	0.4233	0.4175	←
WATER CONTENT	w [%]	16.23	15.62	←	15.62	15.41	←
SATURATION	S [%]	100.3	100.0	←	100.0	100.0	←

DESCRIPTION	brown clay with some stones—till (visual)		
CLASSIFICATION	A-6b (ODOT class.)	TYPE OF SPECIMEN	undisturbed
LL 36, PL 19, PI 19, G_S 2.71†		TYPE OF TEST	CU

REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
*value calculated not measured	AREA	Logan County—SR33/SR347		
**pore pressure data lost	BORING ID	BH 8	SAMPLE NO.	2
†G_s assumed	DEPTH/ELEV.	(5.0-7.0) ft	DATE	March 22, 2005
B-value = 1	TECH	CMK	CHECKED	
1 of 2	**TRIAXIAL COMPRESSION TEST REPORT**			

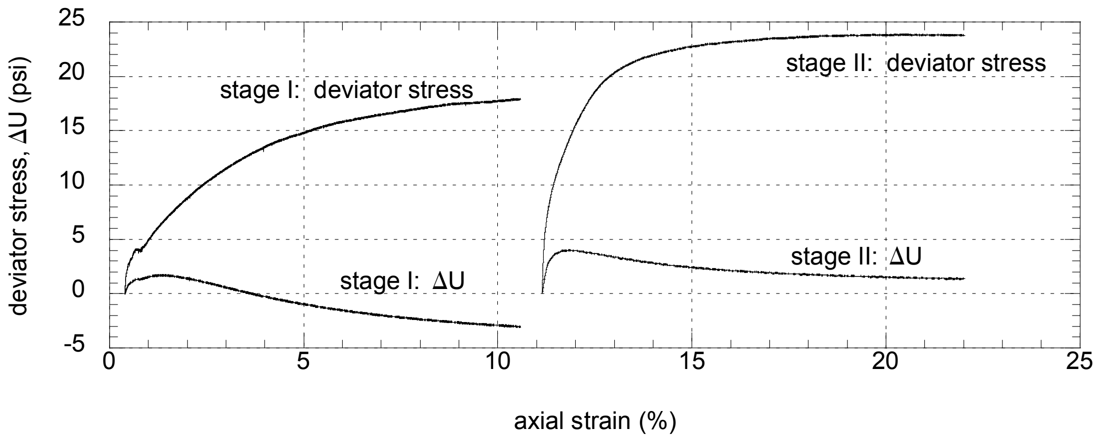
THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY





REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
	AREA	Logan County—SR33/SR347		
	BORING ID	BH 8	SAMPLE NO.	2
	DEPTH/ELEV.	(5.0-7.0) ft	DATE	March 22, 2005
	TECH	CMK	CHECKED	
2 of 2	**TRIAXIAL COMPRESSION TEST REPORT**			

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY



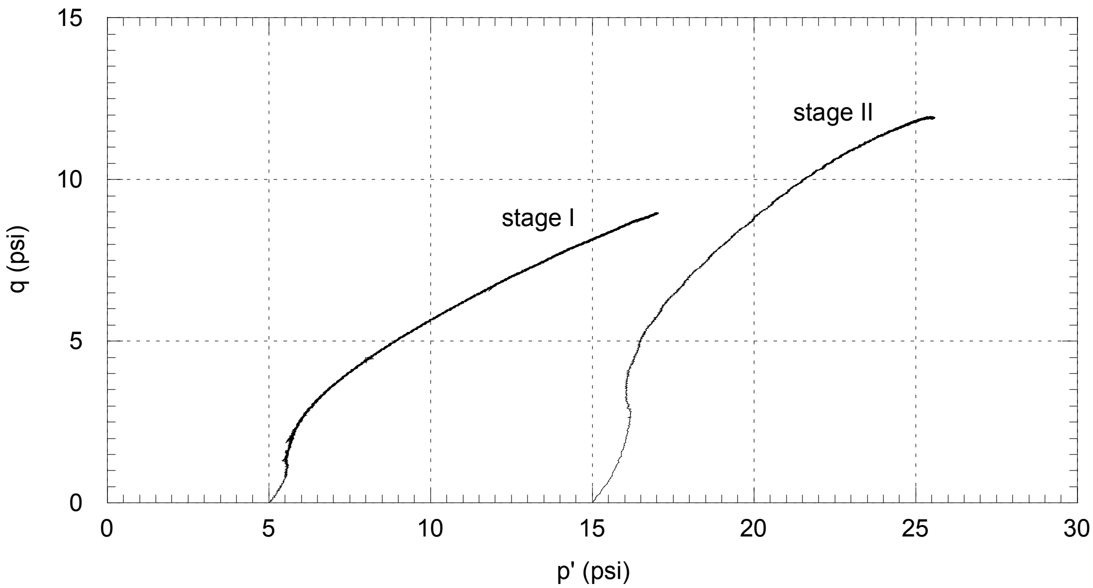
INITIAL	SPECIMEN WEIGHT	W_0 [g]	158.11	MULTI STAGE LOADING		I	II
INITIAL	SPECIMEN HEIGHT	H_0 [in]	3.0	CELL PRESSURE	σ_{cell} [psi]	19	29
INITIAL	SPECIMEN DIAMETER	D_0 [in]	1.4	BACK PRESSURE	u_0 [psi]	14	14
INITIAL	WATER CONTENT	w_0 [%]	23.30*	AXIAL STRAIN RATE	[$^{in}/_{min}$]	0.0010	0.0025
FINAL	SPECIMEN WEIGHT	W_f [g]	155.45	MAX DEVIATOR STRESS	$(\sigma_1 - \sigma_3)_{max}$ [psi]	17.93	23.89
FINAL	SPECIMEN HEIGHT	H_f [in]	2.68	AXIAL STRAIN, $(\sigma_1 - \sigma_3)_{max}$	ε_f [%]	10.58	20.75
FINAL	SPECIMEN DIAMETER	D_f [in]	1.47	MAX EFF. STRESS RATIO	$(\sigma_1'/\sigma_3')_{max}$	3.62	2.82
FINAL	WATER CONTENT	w_f [%]	23.2	AXIAL STRAIN, $(\sigma_1'/\sigma_3')_{max}$	ε_f [%]	3.17	14.88

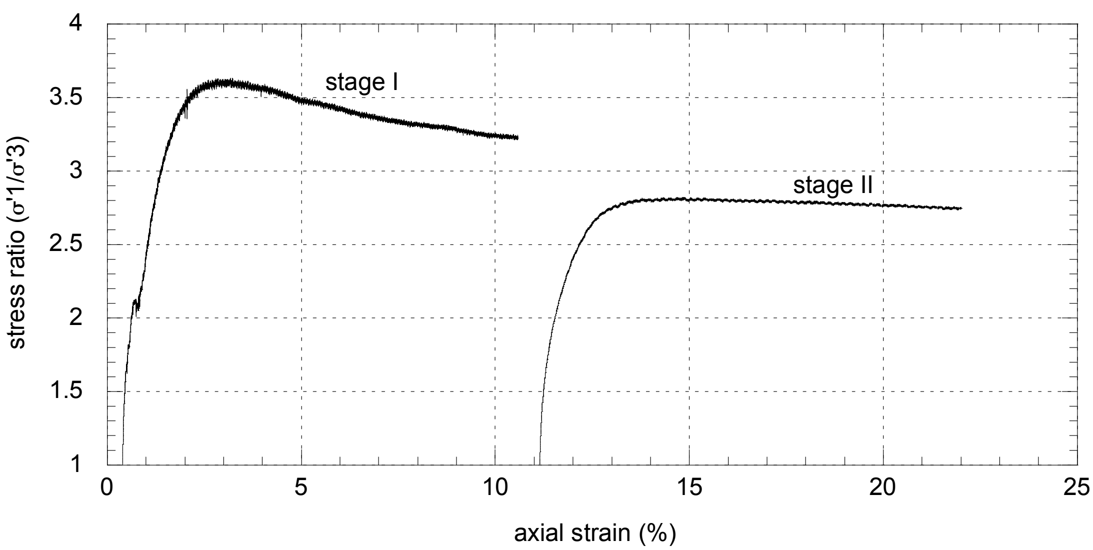
		STAGE I			STAGE II		
		INITIAL	AFTER CONSOLID.	AFTER SHEAR	INITIAL	AFTER CONSOLID.	AFTER SHEAR
DRY UNIT WEIGHT	γ_d [pcf]	104.07	105.32	←	105.32	107.11	←
VOID RATIO	e	0.6250	0.6057	←	0.6057	0.5788	←
WATER CONTENT	w [%]	23.30	22.35	←	22.35	21.36	←
SATURATION	S [%]	101.0	100.0	←	100.0	100.0	←

DESCRIPTION	gray clay (visual)				
CLASSIFICATION	A-4a (ODOT class.)			TYPE OF SPECIMEN	undisturbed
LL 32	PL 28	PI 4	G_S 2.71**	TYPE OF TEST	CU

REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
*w_0(%) from trimmings	AREA	Logan County—SR33/SR347		
**G_s assumed	BORING ID	BH 16	SAMPLE NO.	1-bottom
B-value = 0.95	DEPTH/ELEV.	(6.0-8.0) ft	DATE	February 11, 2005
	TECH	CMK	CHECKED	
1 of 2	**TRIAXIAL COMPRESSION TEST REPORT**			

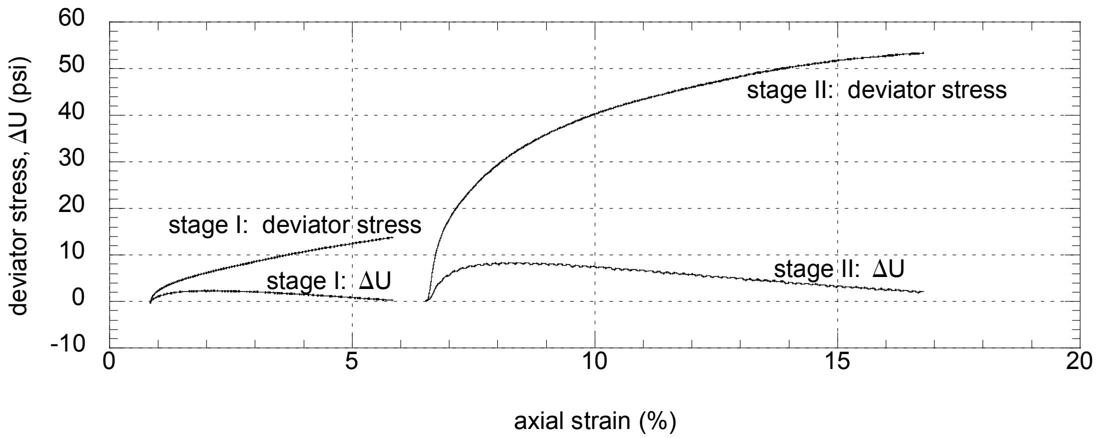
THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY





REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
	AREA	Logan County—SR33/SR347		
	BORING ID	BH 16	SAMPLE NO.	1-bottom
	DEPTH/ELEV.	(6.0-8.0) ft	DATE	February 11, 2005
	TECH	CMK	CHECKED	
2 of 2	**TRIAXIAL COMPRESSION TEST REPORT**			

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY



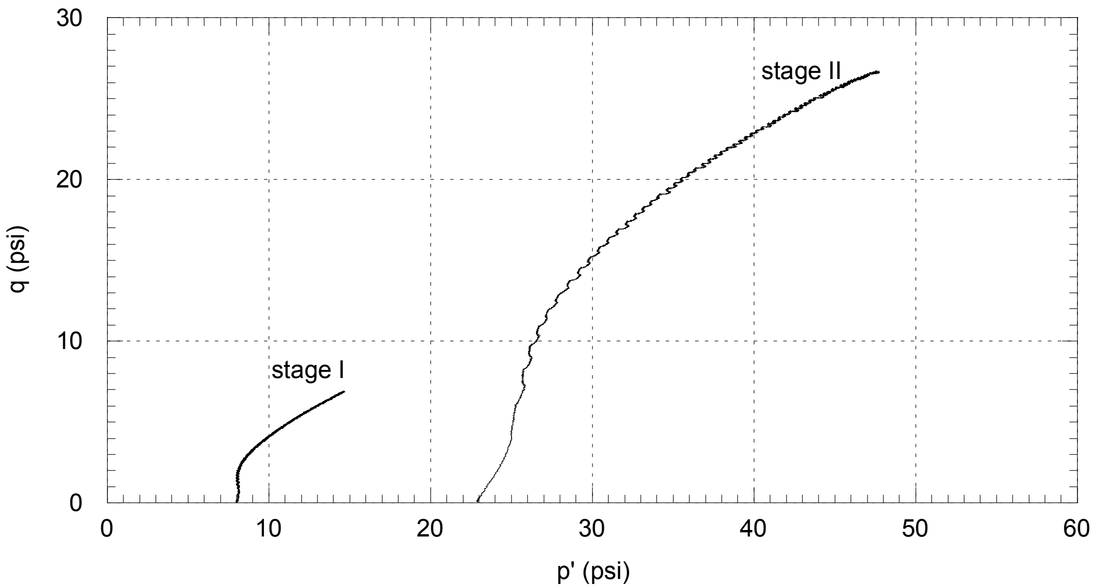
						I	II
INITIAL	SPECIMEN WEIGHT	W_0 [g]	156.56*	MULTI STAGE LOADING			
	SPECIMEN HEIGHT	H_0 [in]	3.0	CELL PRESSURE	σ_{cell} [psi]	22	41
	SPECIMEN DIAMETER	D_0 [in]	1.4	BACK PRESSURE	u_0 [psi]	14	18
	WATER CONTENT	w_0 [%]	21.87**	AXIAL STRAIN RATE	[$^{in}/_{min}$]	0.0001	0.0025
FINAL	SPECIMEN WEIGHT	W_f [g]	157.80	MAX DEVIATOR STRESS	$(\sigma_1 - \sigma_3)_{max}$ [psi]	13.79	53.59
	SPECIMEN HEIGHT	H_f [in]	2.80	AXIAL STRAIN, $(\sigma_1 - \sigma_3)_{max}$	ε_f [%]	5.81	16.77
	SPECIMEN DIAMETER	D_f [in]	1.43	MAX EFF. STRESS RATIO	$(\sigma_1'/\sigma_3')_{max}$	2.79	3.70
	WATER CONTENT	w_f [%]	22.83	AXIAL STRAIN, $(\sigma_1'/\sigma_3')_{max}$	ε_f [%]	5.68	12.70

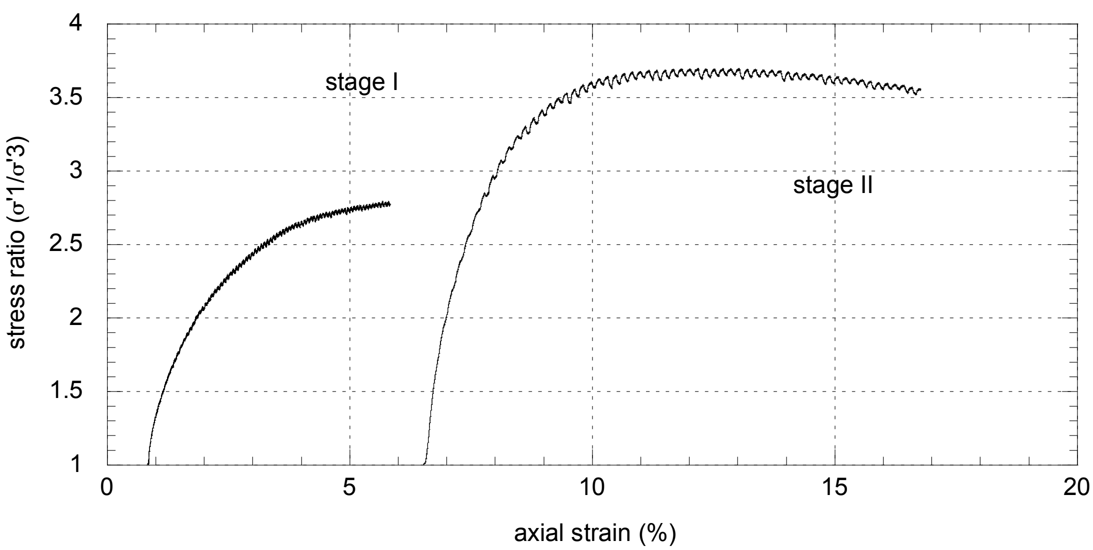
		STAGE I			STAGE II		
		INITIAL	AFTER CONSOLID.	AFTER SHEAR	INITIAL	AFTER CONSOLID.	AFTER SHEAR
DRY UNIT WEIGHT	γ_d [pcf]	105.93	108.66	←	108.66	110.91	←
VOID RATIO	e	0.5964	0.5563	←	0.5563	0.5247	←
WATER CONTENT	w [%]	21.87	20.53	←	20.53	19.36	←
SATURATION	S [%]	99.4	100.0	←	100.0	100.0	←

DESCRIPTION	gray clay (visual)						
CLASSIFICATION	A-6b (ODOT class.)					TYPE OF SPECIMEN	undisturbed
LL	33	PL	20	PI	13	G_s	2.71†
						TYPE OF TEST	CU

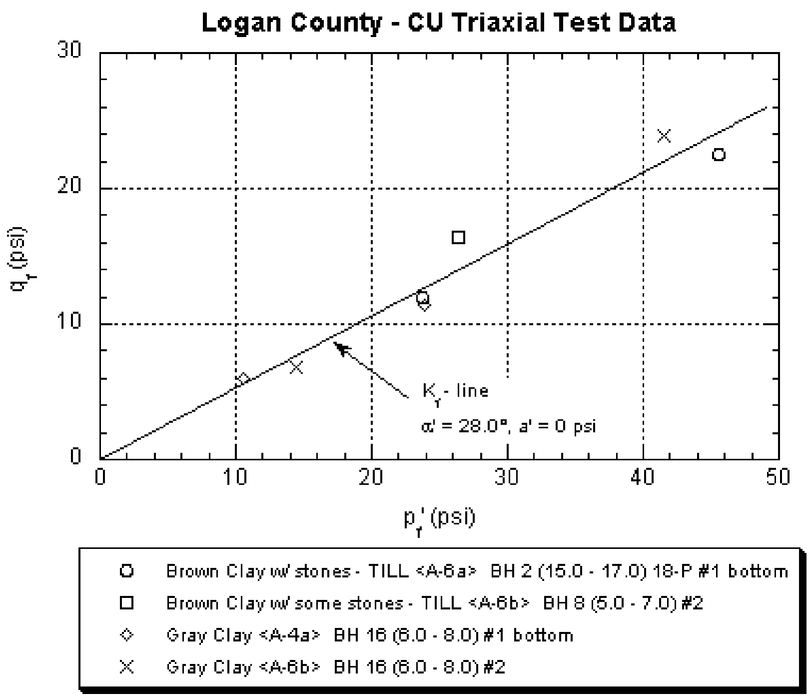
REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
*W_0 calc. from w_0(%)	AREA	Logan County—SR33/SR347		
**w_0(%) calc. from trimmings	BORING ID	BH 16	SAMPLE NO.	2
†G_s assumed	DEPTH/ELEV.	(6.0-8.0) ft	DATE	February 11, 2005
B-value = 0.88	TECH	CMK	CHECKED	
1 of 2	**TRIAXIAL COMPRESSION TEST REPORT**			

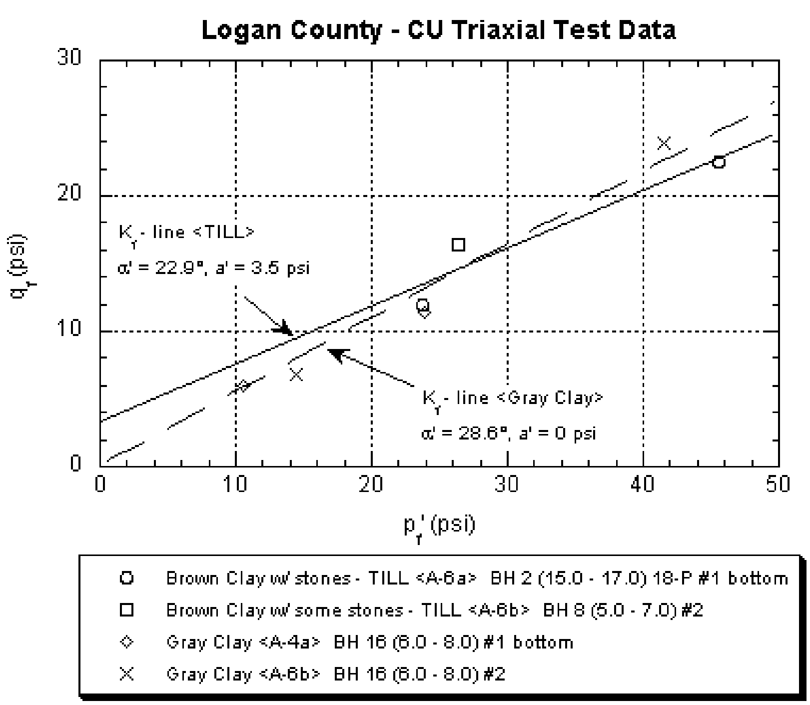
THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY





REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
	AREA	Logan County—SR33/SR347		
	BORING ID	BH 16	SAMPLE NO.	2
	DEPTH/ELEV.	(6.0-8.0) ft	DATE	February 11, 2005
	TECH	CMK	CHECKED	
2 of 2	**TRIAXIAL COMPRESSION TEST REPORT**			


Logan County - CU Triaxial Test Data
q_T (psi)
p_T' (psi)
K_T - line
α' = 28.0°, a' = 0 psi
Brown Clay w/ stones - TILL <A-6a> BH 2 (15.0 - 17.0) 18-P #1 bottom
Brown Clay w/ some stones - TILL <A-6b> BH 8 (5.0 - 7.0) #2
Gray Clay <A-4a> BH 16 (6.0 - 8.0) #1 bottom
Gray Clay <A-6b> BH 16 (6.0 - 8.0) #2


Logan County - CU Triaxial Test Data
q_T (psi)
p_T' (psi)
K_T - line <TILL>
α' = 22.9°, a' = 3.5 psi
K_T - line <Gray Clay>
α' = 28.6°, a' = 0 psi
Brown Clay w/ stones - TILL <A-6a> BH 2 (15.0 - 17.0) 18-P #1 bottom
Brown Clay w/ some stones - TILL <A-6b> BH 8 (5.0 - 7.0) #2
Gray Clay <A-4a> BH 16 (6.0 - 8.0) #1 bottom
Gray Clay <A-6b> BH 16 (6.0 - 8.0) #2

Determination of ϕ' and c' from CU Triaxial Test Results

<TILL>

K_f-line eqn: $q = 0.42166\,p' + 3.5297$

$$\alpha' = \tan^{-1} 0.42166 = 22.9°$$
$$a' = 3.5\ psi$$

$$\phi' = \sin^{-1} 0.42166 = 24.9°$$
$$c' = \frac{a'}{\cos\phi'} = \frac{3.5\,psi}{\cos 24.9°} = 3.9\,psi$$

<GRAY CLAY>

K_f-line eqn: $q = 0.54502\,p'$

$$\alpha' = \tan^{-1} 0.54502 = 28.6°$$
$$a' = 0\ psi$$

$$\phi' = \sin^{-1} 0.54502 = 33.0°$$
$$c' = 0\,psi$$

<ALL DATA POINTS>

K_f-line eqn: $q = 0.53271\,p'$

$$\alpha' = \tan^{-1} 0.53271° = 28.0°$$
$$a' = 0\ psi$$

$$\phi' = \sin^{-1} 0.53271 = 32.2°$$
$$c' = 0\,psi$$

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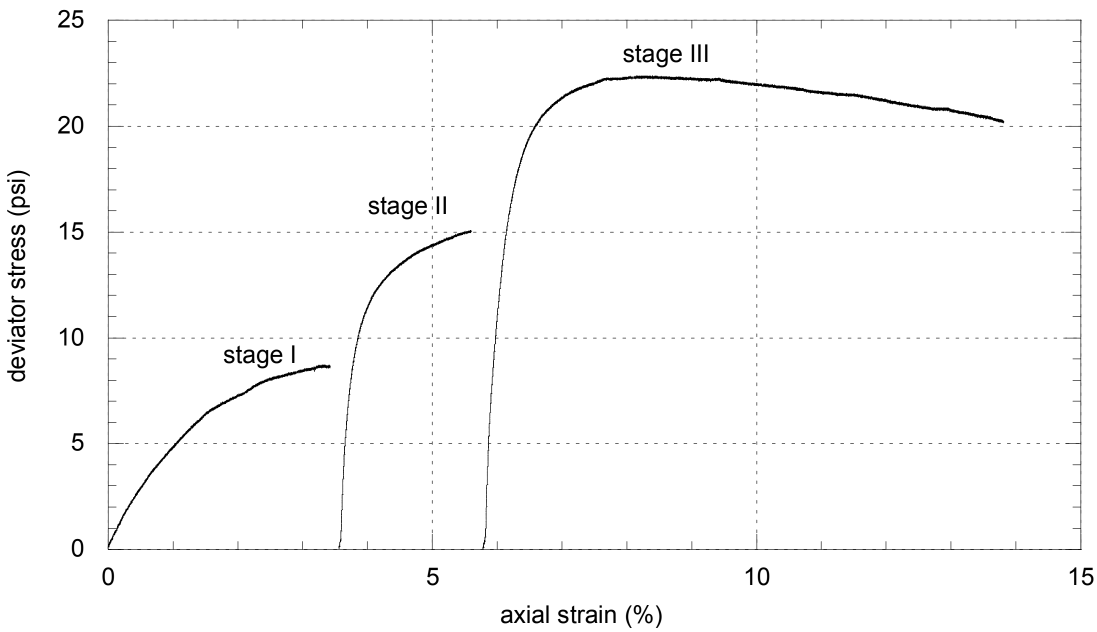
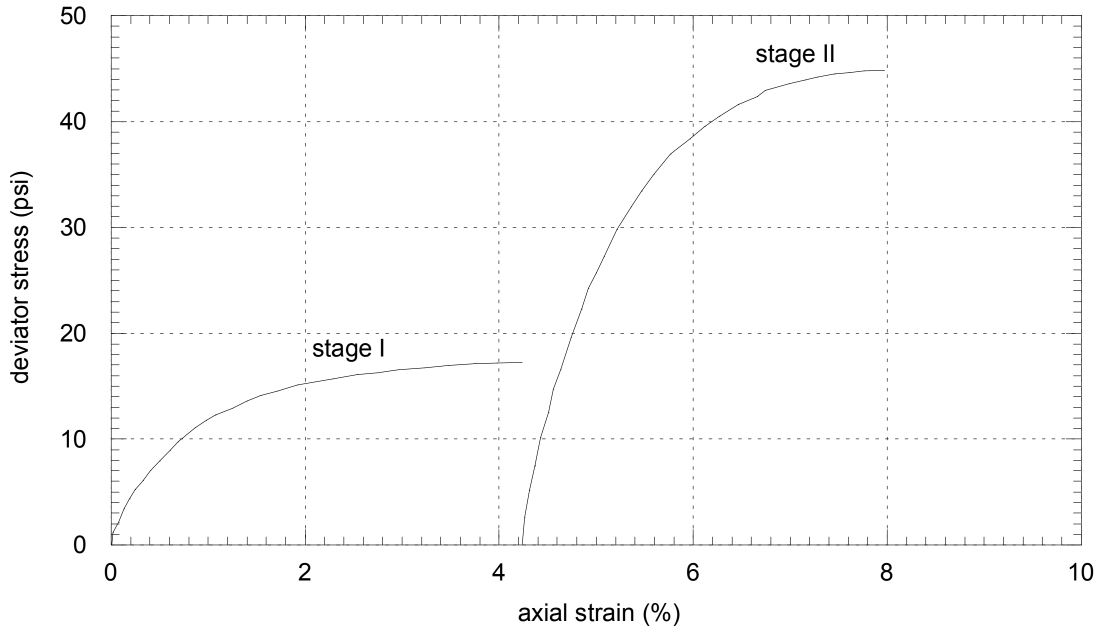


	INITIAL		FINAL		MULTISTAGE LOADING		I	II	III
SPECIMEN WEIGHT	W_0 [g]	181.0	W_f [g]	178.46	CELL PRESSURE	σ_3 [psi]	18	18	25
SPECIMEN HEIGHT	H_0 [in]	3.23	H_f [in]	2.75	PORE AIR PRESSURE	u_a [psi]	15	15	15
SPECIMEN DIAMETER	D_0 [in]	1.45	D_f [in]	1.51	PORE WATER PRESSURE	u_w [psi]	13	8	8
WATER CONTENT	w_0 [%]	17.18*	w_f [%]	16.2	NET CONFINING PRESSURE	$(\sigma_3 - u_a)$ [psi]	3	3	10
DRY UNIT WEIGHT	γ_{d0} [pcf]	109.35	γ_{df} [pcf]	118.43	MATRIC SUCTION	$(\sigma_a - u_w)$ [psi]	2	7	7
SATURATION	S_0 [%]	85.19	S_f [%]	102.60	MAXIMUM DEVIATOR STRESS	$(\sigma_1 - \sigma_3)_{max}$ [psi]	8.70	15.05	22.37
VOID RATIO	e_0	0.5465	e_f	0.4279	MAXIMUM STRESS RATIO	$(\sigma_1 / \sigma_3)_{max}$	1.48	1.84	1.89

DESCRIPTION	gray clay (visual)		
CLASSIFICATION	A-6a (ODOT class.)		
AXIAL STRAIN RATE	0.001 $^{mm}/_{min}$	TYPE OF SPECIMEN	undisturbed
LL 30	PL 17	PI 13	G_S 2.71**
		TYPE OF TEST	CD - unsaturated

REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
*w_0(%) from trimmings	AREA	Logan County—SR33/SR347		
**G_s assumed	BORING ID	BH 16	SAMPLE NO.	1-bottom
	DEPTH/ELEV.	(4.0-6.0) ft	DATE	February 5, 2006
	TECH	CMK	CHECKED	
1 of 1	**TRIAXIAL COMPRESSION TEST REPORT**			

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY



		INITIAL		FINAL	MULTISTAGE LOADING		I	II
SPECIMEN WEIGHT	W_0 [g]	170.47	W_f [g]	177.01	CELL PRESSURE	σ_3 [psi]	18	33
SPECIMEN HEIGHT	H_0 [in]	3.2	H_f [in]	-	PORE AIR PRESSURE	u_a [psi]	15	20
SPECIMEN DIAMETER	D_0 [in]	1.4	D_f [in]	-	PORE WATER PRESSURE	u_w [psi]	10	10
WATER CONTENT	w_0 [%]	11.36*	w_f [%]	16.25	NET CONFINING PRESSURE	$(\sigma_3 - u_a)$ [psi]	3	13
DRY DENSITY	γ_{d0} [pcf]	118.38	γ_{df} [pcf]	-	MATRIC SUCTION	$(\sigma_a - u_w)$ [psi]	5	10
SATURATION	S_0 [%]	71.86	S_f [%]	-	MAXIMUM DEVIATOR STRESS	$(\sigma_1 - \sigma_3)_{max}$ [psi]	17.24	44.87
VOID RATIO	e_0	0.4284	e_f	-	MAXIMUM STRESS RATIO	$(\sigma_1 / \sigma_3)_{max}$	1.96	2.36

DESCRIPTION	brown clay w/ few stones (visual)
CLASSIFICATION	A-4a (ODOT class.)
AXIAL STRAIN RATE	0.001 mm/min
TYPE OF SPECIMEN	undisturbed
LL	22
PL	16
PI	6
G_S	2.71**
TYPE OF TEST	CD - unsaturated

REMARKS		
*w_0(%) from trimmings	PROJECT	Bio-Engineering for Land Stabilization
**G_s assumed	AREA	Logan County—SR33/SR347
	BORING ID	BH 17
	SAMPLE NO.	4
	DEPTH/ELEV.	(8.0-10.0) ft
	DATE	November 9, 2005
	TECH	CMK
	CHECKED	

1 of 1

TRIAXIAL COMPRESSION TEST REPORT

ϕ^b from CU and CD triaxial test results (after Fredlund and Rahardjo 1993)

ϕ^b is determined theatrically as outline in Fredlund and Rahardjo (1993) by the following equations which are referenced by equation number as it appears in the text.

Graphically, the extending Mohr-Coulomb failure envelope and parameters:

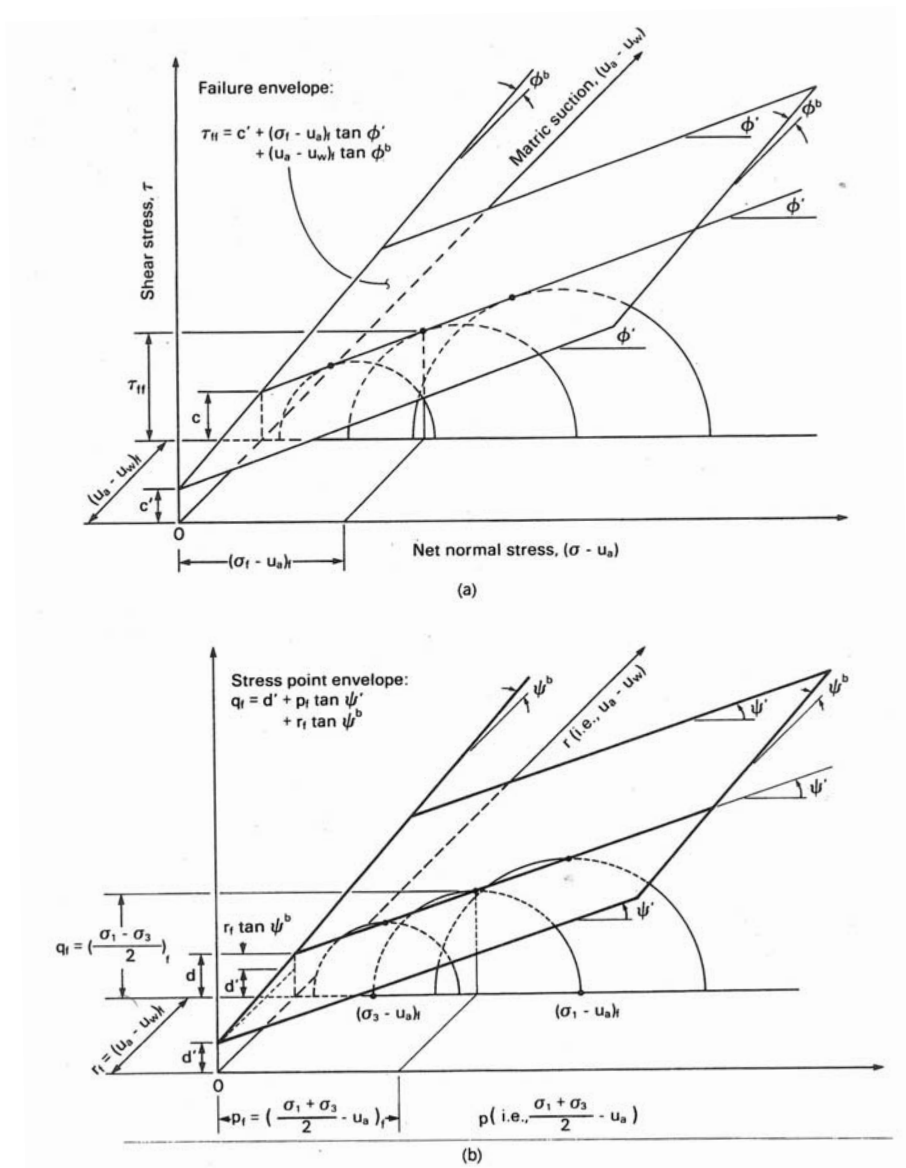


Figure 9.23 Comparisons of the failure envelope and the corresponding stress point envelope. (a) Extended Mohr–Coulomb failure envelope; (b) stress point envelope.

ϕ^b from CU and CD triaxial test results (after Fredlund and Rahardjo 1993)

(continued)

$$d = d' + r_f \tan\psi^b \tag{9.10}$$

$$\tan\psi' = \sin\phi' \tag{9.13}$$

$$d = c\cos\phi' \tag{9.15}$$

$$d' = c'\cos\phi' \tag{9.16}$$

$$\tan\psi^b \tan\phi^b \cos\phi' \tag{9.18}$$

$$c = \frac{q_f}{\cos\phi'} - p_f \tan\phi' \tag{9.20}$$

where:
- c = total cohesion intercept
- c' = effective cohesion
- d = ordinate intercept of the stress point envelope on the q axis at an r_f and p_f value equal to zero
- d' = intercept of the stress point envelope on the q axis when p_f and r_f are equal to zero
- P_f = mean net normal stress at failure
- q_f = half of the deviator stress at failure
- $(\sigma_f - u_a)_f$ = net normal stress state on the failure plane at failure
- u_{af} = pore-air pressure on the failure plane at failure
- u_{wf} = pore-water pressure on the failure plane at failure
- r_f = matric suction at failure [i.e., $(u_a - u_w)_f$]
- ψ' = slope angle of the stress point envelope with respect to the stress variable, p_f
- ψ^b = slope angle of the stress point envelope with respect to the stress variable, r_f
- ϕ' = angle of internal friction associate with the net normal stress state variable, $(\sigma_f - u_w)_f$
- ϕ^b = angle indicating the rate of increase in shear strength relative to the matric suction, $(u_a - u_w)_f$

ϕ^b **from CU and CD triaxial test results (after Fredlund and Rahardjo 1993)**

(continued)

Spreadsheet calculations

Sample	B-2 18P-1 (17')		B-16 #1 (6')			
Soil	gray clay		brown clay with few small stones			
Stage	I	II	I	II	III	
σ_3	18	33	18	18	25	
u_a	15	20	15	15	15	
u_w	10	10	13	8	8	
$\sigma_3 - u_a$	3	13	3	3	10	
$u_a - u_w$	5	10	2	7	7	
$(\sigma_{1/}\ \sigma_3)_{max}$	2.0	2.4	1.5	1.8	1.9	
σ_{1f}	35.3	77.9	26.6	33.1	47.3	
σ_{1f}-u_a	20.3	57.9	11.6	18.1	32.3	
p_f	11.6	35.4	7.3	10.6	21.1	
q_f	8.6	22.4	4.3	7.6	11.1	
ϕ'	32.2	32.2	32.2	32.2	32.2	*assumed from CU*
c'	0	0	0	0	0	*assumed from CU*
ψ'	28.1	28.1	28.1	28.1	28.1	*Eqn. 9.13*
d'	0	0	0	0	0	*Eqn. 9.16*
c	2.9	4.2	0.5	2.3	-0.2	*Eqn. 9.19 or 9.20*
d	2.4	3.6	0.4	1.9	-0.1	*Eqn. 9.15*
r_f	5	10	2	7	7	$r_f = u_a$-u_w
ψ^b	26.0	19.6	11.8	15.4	-1.1	*Eqn. 9.10*
ϕ^b	29.9	22.8	13.9	18.1	-1.3	*Eqn. 9.18*

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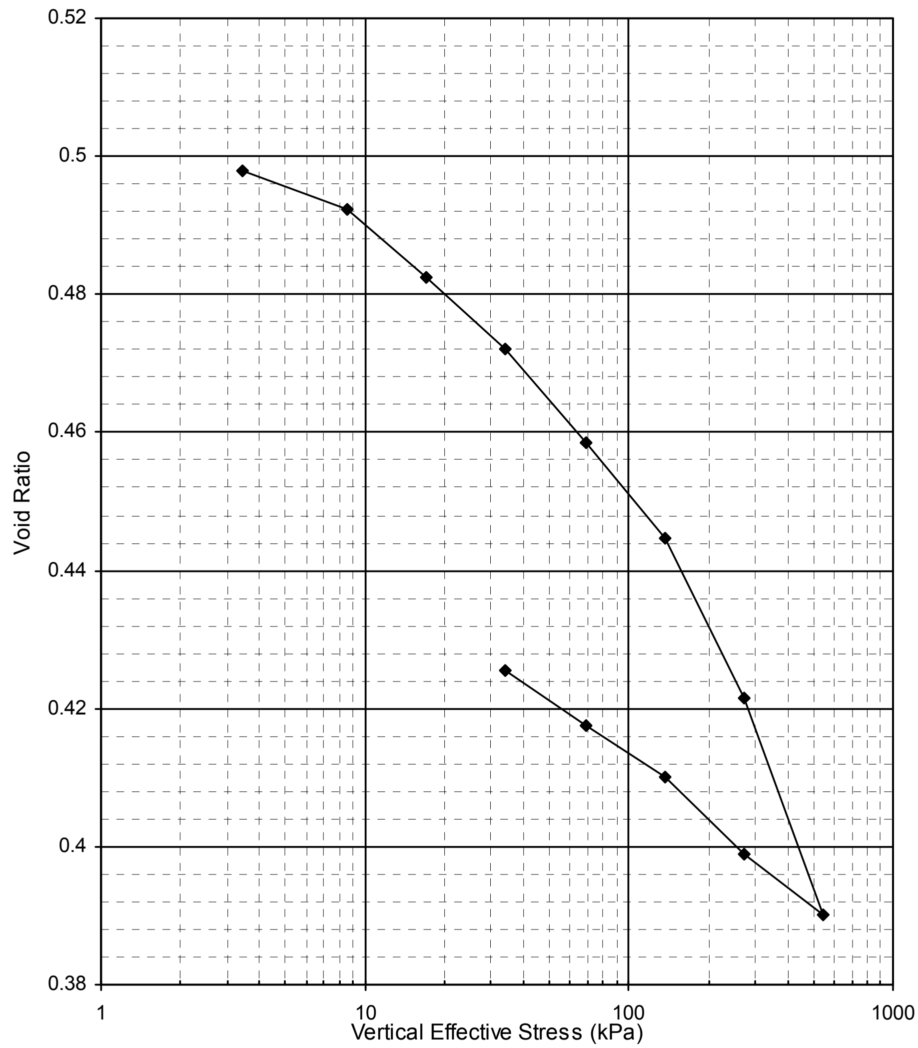
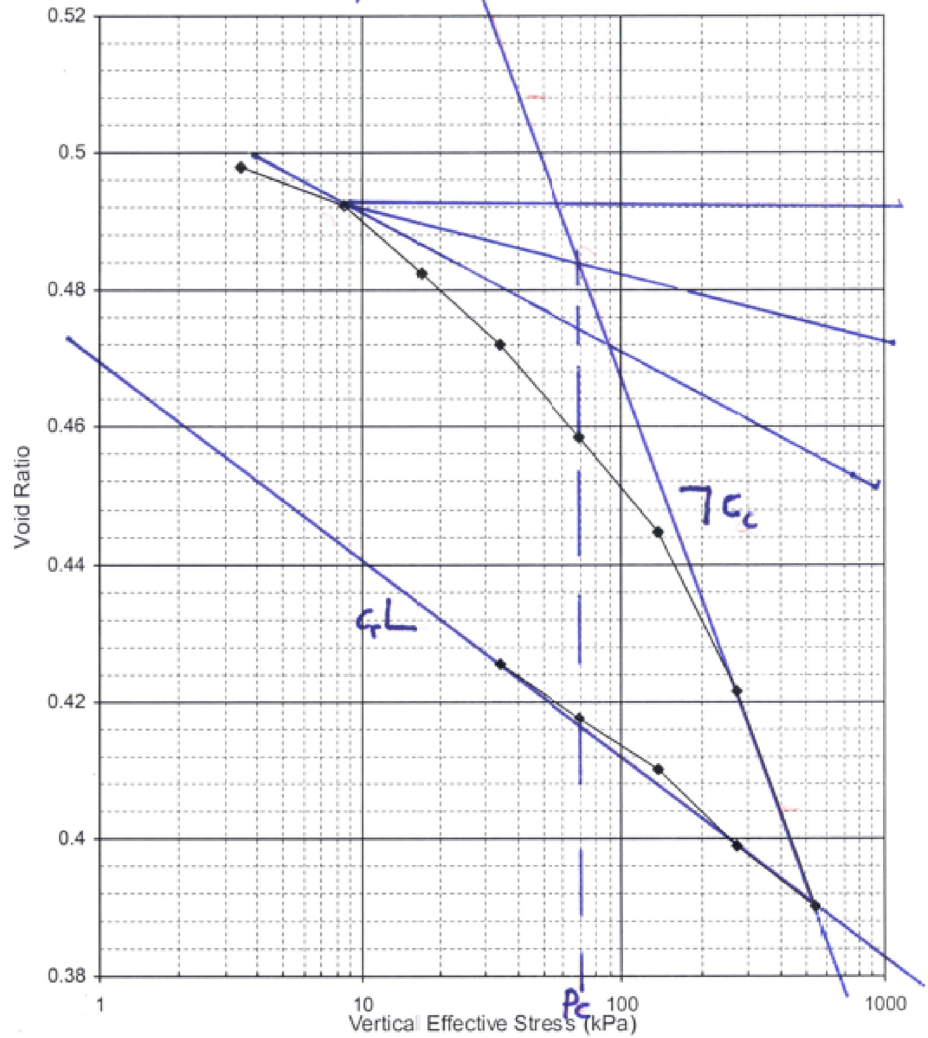


DESCRIPTION	Brown/gray clay w/ trace sand (visual)						
CLASSIFICATION	-					TYPE OF SPECIMEN	undisturbed
LL	-	PL	-	PI	-	w [%] 18	CONSOLIDOMETER

REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
	AREA	Logan County—SR33/SR347		
	BORING ID	BH 16	SAMPLE NO.	3-top
	DEPTH/ELEV.	(6.0-8.0) ft	DATE	February 24, 2005
	TECH	CMK, CHB, TM	CHECKED	
1 of 1	**CONSOLIDATION TEST REPORT**			

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY

Determination of Pc, Cc & Cr



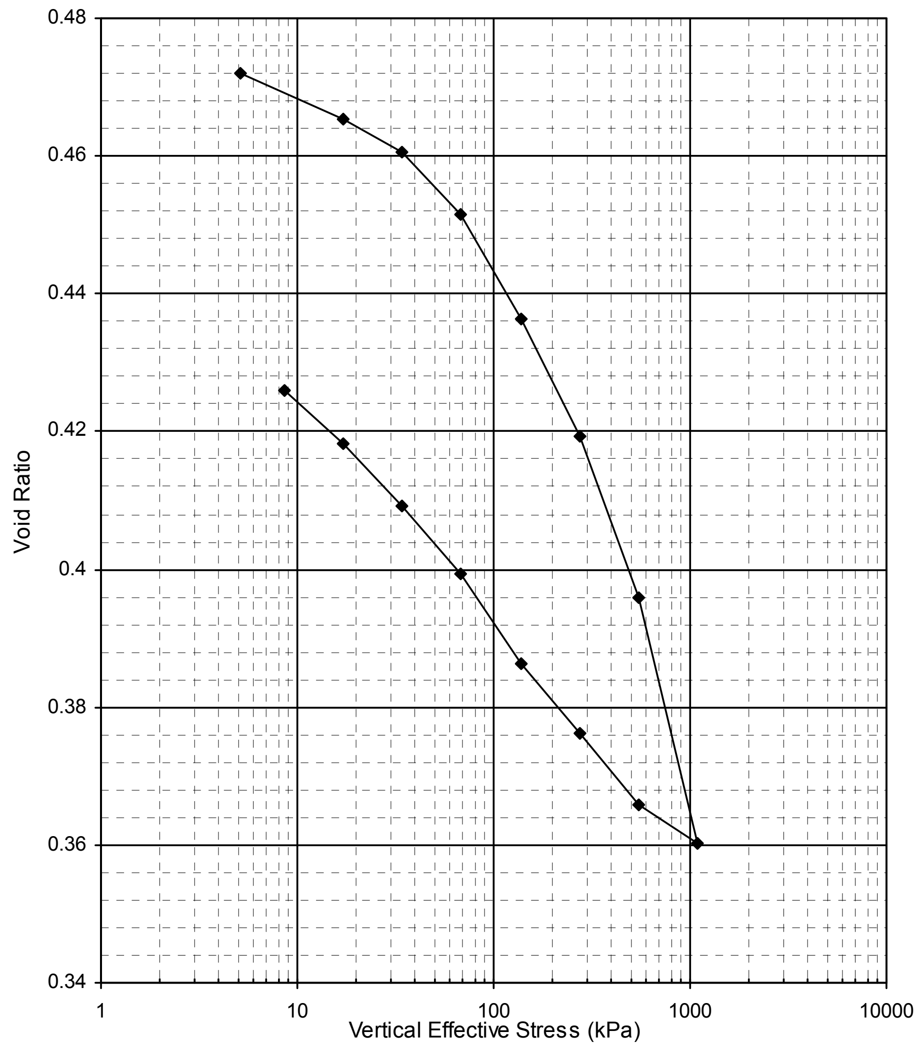
DESCRIPTION	Brown/gray clay w/ trace sand (visual)						
CLASSIFICATION	-					TYPE OF SPECIMEN	undisturbed
LL -	PL -	PI -	w [%] 18			CONSOLIDOMETER	
REMARKS		PROJECT	Bio-Engineering for Land Stabilization				
Pc = 70 kPa		AREA	Logan County—SR33/SR347				
Cc = 0.104		BORING ID	BH 16			SAMPLE NO.	3-top
Cr = 0.028		DEPTH/ELEV.	(6.0-8.0) ft			DATE	February 24, 2005
		TECH	CMK, CHB, TM			CHECKED	
1 of 1		CONSOLIDATION TEST REPORT					

Determination of OCR from Consolidation Test Results

Sample B-16 (6 ft.)

P_c = 1462.0 psf
γ_d = 111.0 pcf
S = 100%
H = 6 ft
H_w = 3 to 7 ft (piezo readings at P-15 range from -1 to 3 ft. bgs)
w = 18.0%
$\gamma = \gamma_d (1+w)$ = 111.0 pcf ·1.18 = 131.0 pcf
$\sigma_v = H \cdot \gamma$ = 6 ft · 131.0 pcf = 786.0 psf
$u = H_w \cdot \gamma_w$ = 3 ft · 62.4 pcf = 187.2 psf
= 7 ft · 62.4 pcf = 436.8 psf
$\sigma'_v = \sigma_v - u$ = 786.0 psf – 187.2 psf = 598.8 psf
= 786.0 psf – 436.8 psf = 349.2 psf
OCR = P_c/σ'_v = 1462.0 psf/598.8 psf = 2.4
= 1462.0 psf/349.2 psf = 4.2

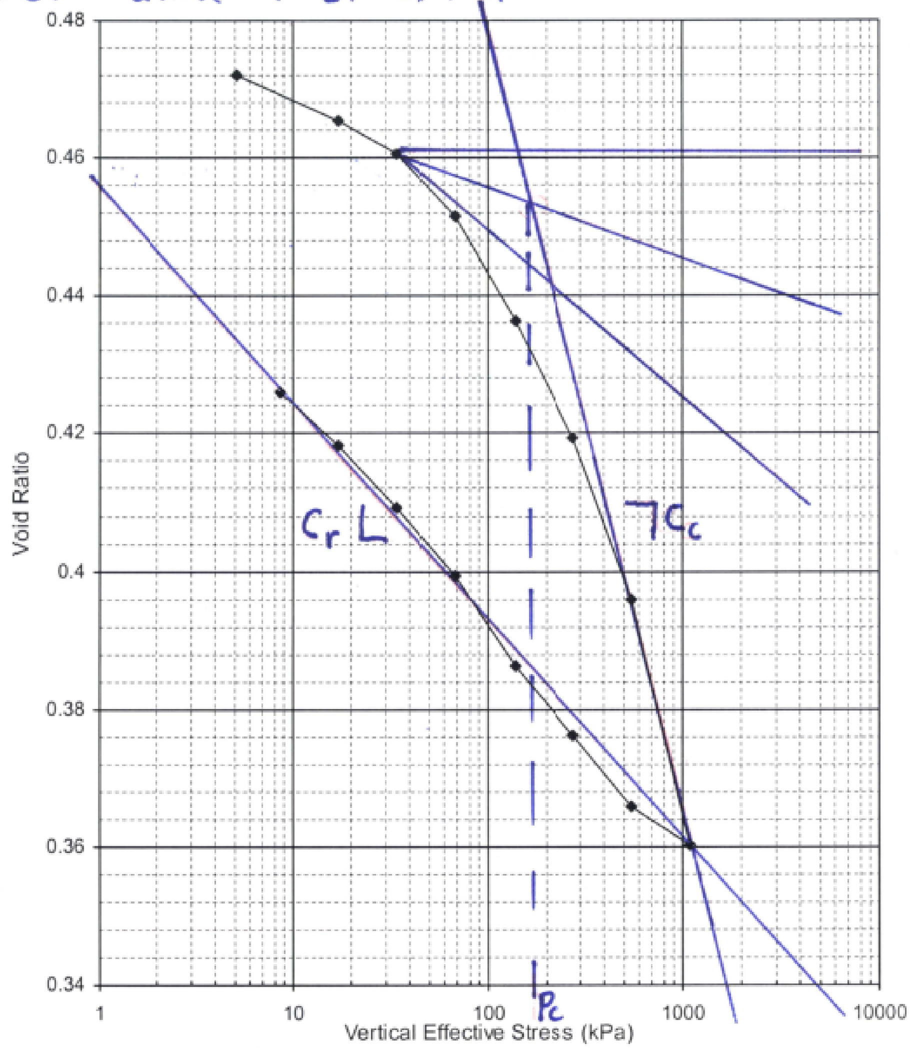
THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY



DESCRIPTION	Brown/gray clay (visual)							
CLASSIFICATION	-						TYPE OF SPECIMEN	undisturbed
LL	-	PL	-	PI	-	w [%] 20	CONSOLIDOMETER	13
REMARKS		PROJECT	Bio-Engineering for Land Stabilization					
		AREA	Logan County—SR33/SR347					
		BORING ID	BH 10				SAMPLE NO.	1-bottom
		DEPTH/ELEV.	(4.0-6.0) ft				DATE	June 24, 2005
		TECH	CMK, CHB				CHECKED	
1 of 1		CONSOLIDATION TEST REPORT						

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY

Determination of P_c, C_c, & C_r



DESCRIPTION	Brown/gray clay (visual)						
CLASSIFICATION	-					TYPE OF SPECIMEN	undisturbed
LL	-	PL	-	PI	-	w [%] 20	CONSOLIDOMETER 13

REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
P_c = 168 kPa	AREA	Logan County—SR33/SR347		
C_c = 0.114	BORING ID	BH 10	SAMPLE NO.	1-bottom
C_r = 0.032	DEPTH/ELEV.	(4.0-6.0) ft	DATE	June 24, 2005
	TECH	CMK, CHB	CHECKED	
1 of 1	**CONSOLIDATION TEST REPORT**			

Determination of OCR from Consolidation Test Results

Sample B-10 (6 ft.) – consolidometer 13

P_c = 3508.8 psf
γ_d = 105.0 pcf
S = 100%
H = 6 ft
H_w = 0.5 to 6 ft (piezo readings at P-9 range from 0 to 5.5 ft. bgs)
w = 20.0%
$\gamma = \gamma_d (1+w)$ = 105.0 pcf ·1.20 = 126.0 pcf
$\sigma_v = H \cdot \gamma$ = 6 ft · 126.0 pcf = 756.0 psf
$u = H_w \cdot \gamma_w$ = 0.5 ft · 62.4 pcf = 31.2 psf
= 6 ft · 62.4 pcf = 374.4 psf
$\sigma'_v = \sigma_v - u$ = 756.0 psf – 31.2 psf = 724.8 psf
= 756.0 psf – 374.4 psf = 381.6 psf
OCR = P_c/σ'_v = 3508.8 psf/724.8 psf = 4.8
= 3508.8 psf/381.6 psf = 9.2

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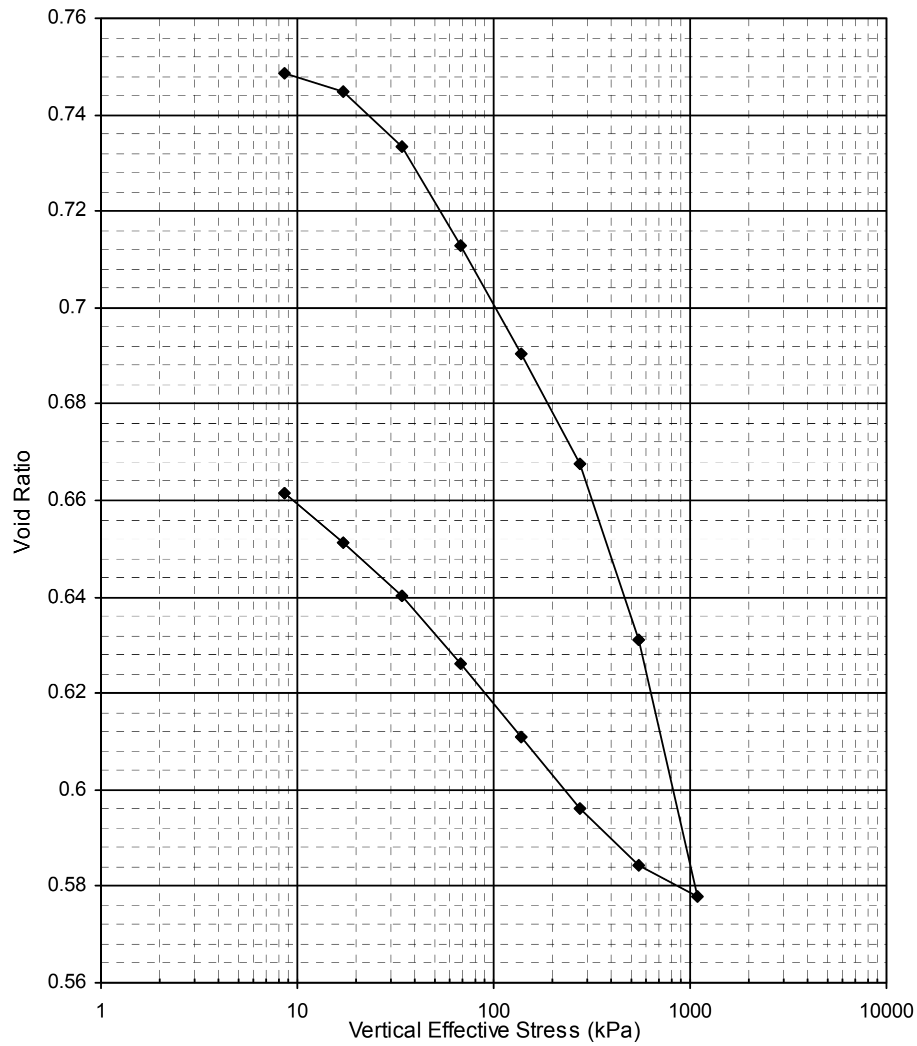
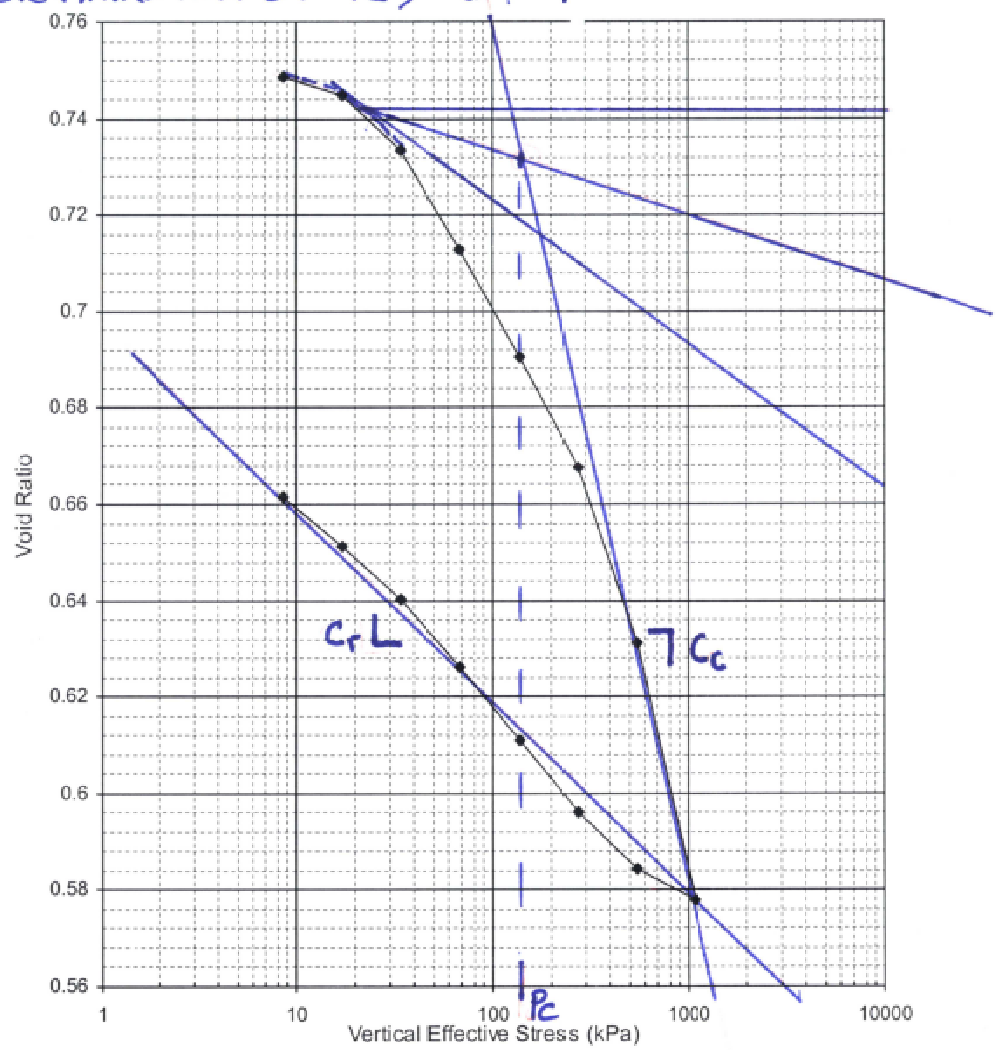


DESCRIPTION	Brown/gray clay (visual)						
CLASSIFICATION	-					TYPE OF SPECIMEN	undisturbed
LL -		PL -		PI -	w [%] 17	CONSOLIDOMETER	21

REMARKS	PROJECT	Bio-Engineering for Land Stabilization		
	AREA	Logan County—SR33/SR347		
	BORING ID	BH 10	SAMPLE NO.	1-bottom
	DEPTH/ELEV.	(4.0-6.0) ft	DATE	June 24, 2005
	TECH	CMK, CHB	CHECKED	
1 of 1	**CONSOLIDATION TEST REPORT**			

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY

Determination of P_c, C_c & C_r



DESCRIPTION	Brown/gray clay (visual)						
CLASSIFICATION	-					TYPE OF SPECIMEN	undisturbed
LL	-	PL	-	PI	-	w [%] 17	CONSOLIDOMETER 21
REMARKS		PROJECT	Bio-Engineering for Land Stabilization				
P_c = 145 kPa		AREA	Logan County—SR33/SR347				
C_c = 0.176		BORING ID	BH 10	SAMPLE NO.	1-bottom		
C_r = 0.042		DEPTH/ELEV.	(4.0-6.0) ft	DATE	June 24, 2005		
		TECH	CMK, CHB	CHECKED			
1 of 1		CONSOLIDATION TEST REPORT					

Determination of OCR from Consolidation Test Results

Sample B-10 (6 ft.) – consolidometer 21

P_c = 3028.4 psf
γ_d = 105.0 pcf
S = 100%
H = 6 ft
H_w = 0.5 to 6 ft (piezo readings at P-9 range from 0 to 5.5 ft. bgs)
w = 17%
$\gamma = \gamma_d\,(1+w)$ = 113.0 pcf ·1.17 = 132.2 pcf
$\sigma_v = H \cdot \gamma$ = 6 ft · 132.1 pcf = 792.6 psf
$u = H_w \cdot \gamma_w$ = 0.5 ft · 62.4 pcf = 31.2 psf
= 6 ft · 62.4 pcf = 374.4 psf
$\sigma'_v = \sigma_v - u$ = 792.6 psf – 31.2 psf = 761.4 psf
= 792.3 psf – 374.4 psf = 418.2 psf
OCR = P_c/σ'_v = 3028.4 psf/761.4 psf = 4.0
= 3028.4 psf/418.2 psf = 7.2

CAL MAR
SOIL TESTING LABS

02/13/06
LOGAN CO
SR 33/SR 347

Sample submitted by:
CHRISTOPHER KOKESH
6893 MEADOW OAK DR.
COLUMBUS, OH 43235

Test Results

Results are in pounds per acre except pH or as noted

LAB NUMBER 278.013 SAMPLE ID	pH		Phosphorus (P)	Potassium (K)	Magnesium (Mg)	Calcium (Ca)
	Soil	Buffer				
2233	7.5		4	191	788	10624

% Organic Matter	Cation Exchange Capacity	PERCENT BASE SATURATION			
		%Potassium	%Magnesium	%Calcium	%Hydrogen
1.5	30.1	0.8	10.9	88	

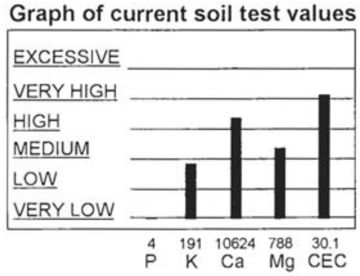
Recommendations

For optimum growth of your shrubs/bushes we recommend adding the following

NITROGEN (N)	PHOSPHORUS (P205)	POTASSIUM (K2O)
POUNDS / 1000 Sq. Ft. per season	POUNDS / 1000 Sq. Ft. per season	POUNDS / 1000 Sq. Ft. per season
2.8	3.4	4.6

Sulfur- Lbs/1000 Sq.Ft.		
SOIL DEPTH (inches)		
3	3	
6	5	
9	8	
12	11	
Type		

Graph of current soil test values



Please see notes on page two

CAL MAR SOIL TESTING LABS, 130 SOUTH STATE STREET, WESTERVILLE, OH 43081
PH: 614-523-1005 FAX: 614.523.1004

CAL MAR
SOIL TESTING LABS

02/13/06

LOGAN CO

SR 33/SR 347

Sample submitted by:
CHRISTOPHER KOKESH

6893 MEADOW OAK DR.
COLUMBUS, OH 43235

Notes

Soil pH

The pH level indicated by your soil test is above the optimum for your requested plant type. It will be necessary to apply sulfur in order to lower the current pH to the target pH level for your plant type. If practical, incorporate the sulfur in the soil to the rooting depth of your plant type. Recommendations have been made to assist you in applying the correct amount of sulfur depending on the expected rooting depth.

Nitrogen

Your nitrogen recommendation is based upon the plants to be grown and the soil organic matter. Your nitrogen recommendation has been increased slightly due to a low soil organic matter percentage. Nitrogen should be worked into the soil prior to planting when possible. For established plants, apply only a portion of the recommended nitrogen throughout the growing season. Be very cautious about applying nitrogen in very warm or dry weather, since this can damage the plant tissue.

Phosphorus

Your phosphorus value is very low and it may be affecting plant growth. It will be necessary to apply phosphorus in order to raise the soil phosphorus level. Phosphorus applications should be worked into the soil prior to planting if possible. If the plant material is already established, apply one-half of the recommended phosphorus to the soil surface, for now. The next time soil and phosphorus can be mixed thoroughly, apply the remaining amount recommended.

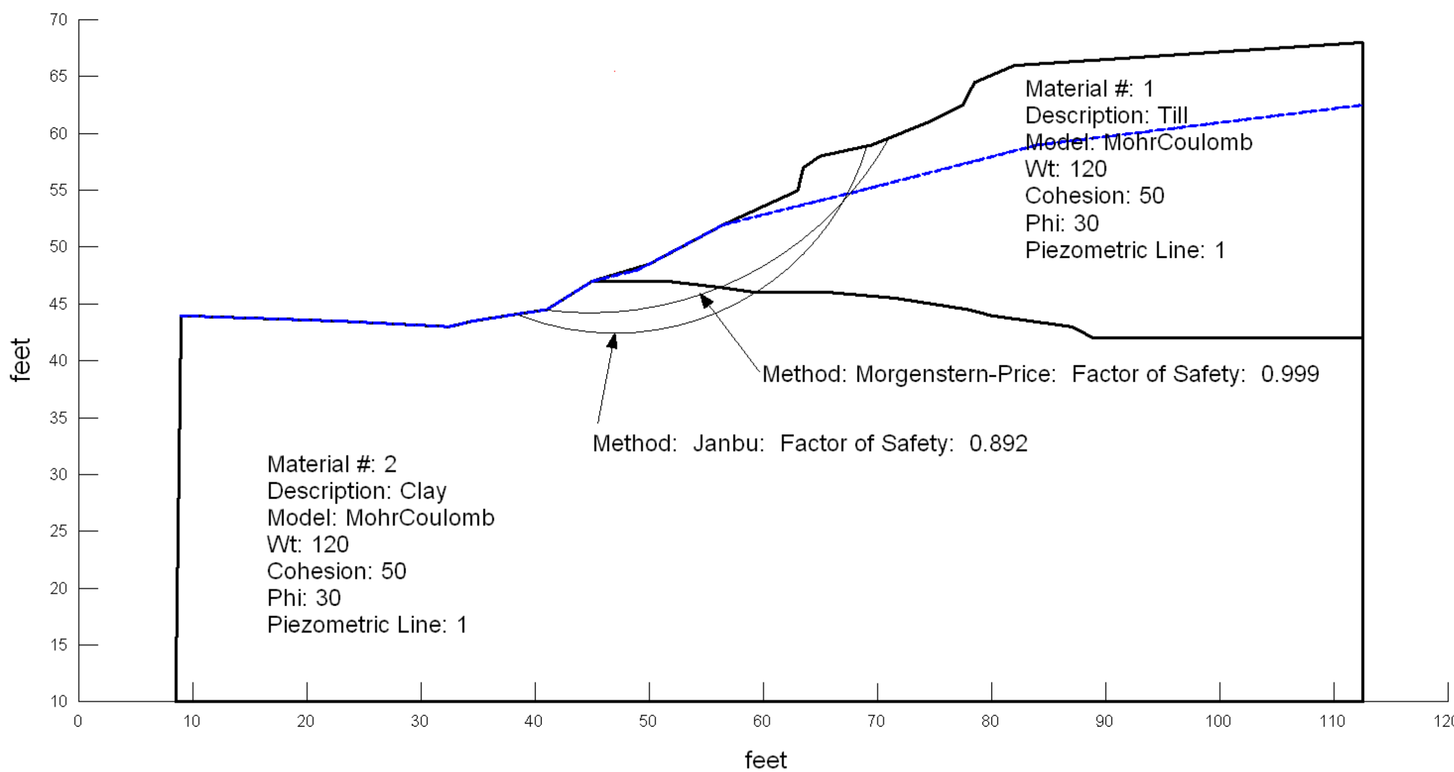
Potassium

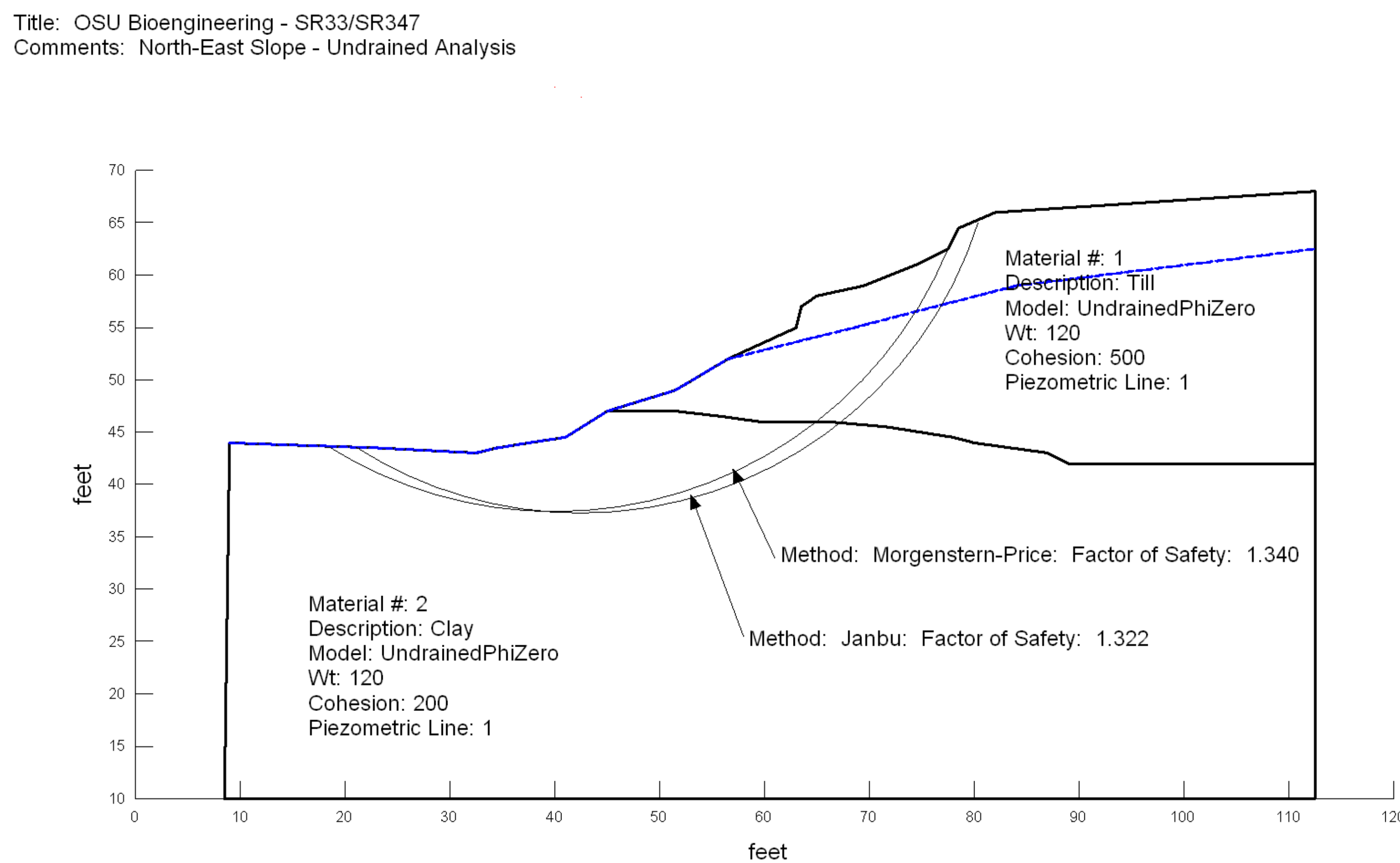
The potassium level indicated on this sample is below the optimum for the indicated plant type. It is necessary to apply potassium in order to raise the level in the soil to the optimum range for your plant type. The best way to apply the potassium is to thoroughly mix the it in the soil to the rooting depth of your plants. Recommendations have been made to assist you in applying the correct amount of potassium. Plant injury can be minimized if you water after applying potassium.

These notes refer to the recommendations and results on page one. Additional general information is enclosed.

CAL MAR SOIL TESTING LABS, 130 SOUTH STATE STREET, WESTERVILLE, OH 43081
PH: 614-523-1005 FAX: 614.523.1004

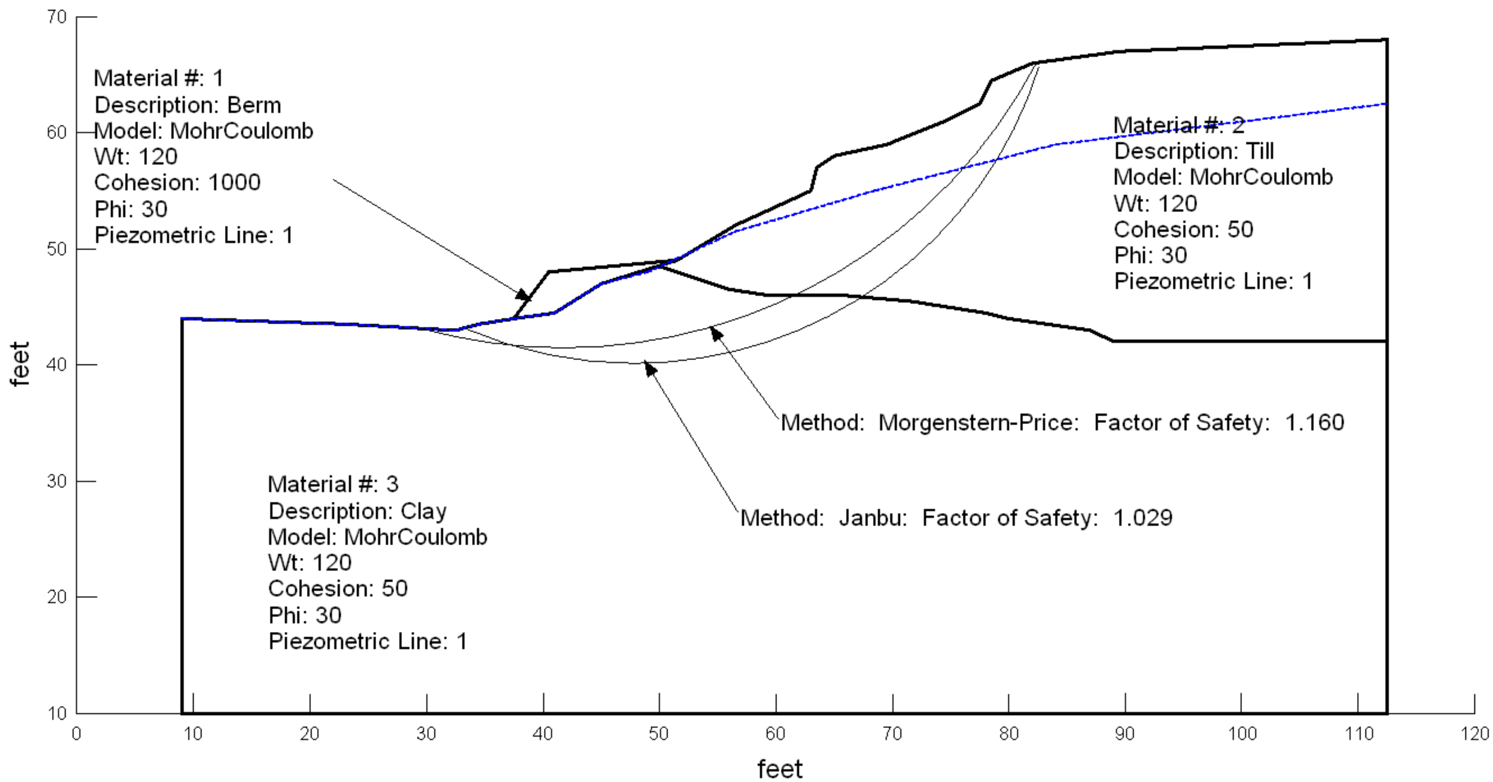
Title: OSU - Bioengineering - SR33/SR347
Comments: North-East Slope - Drained Analysis

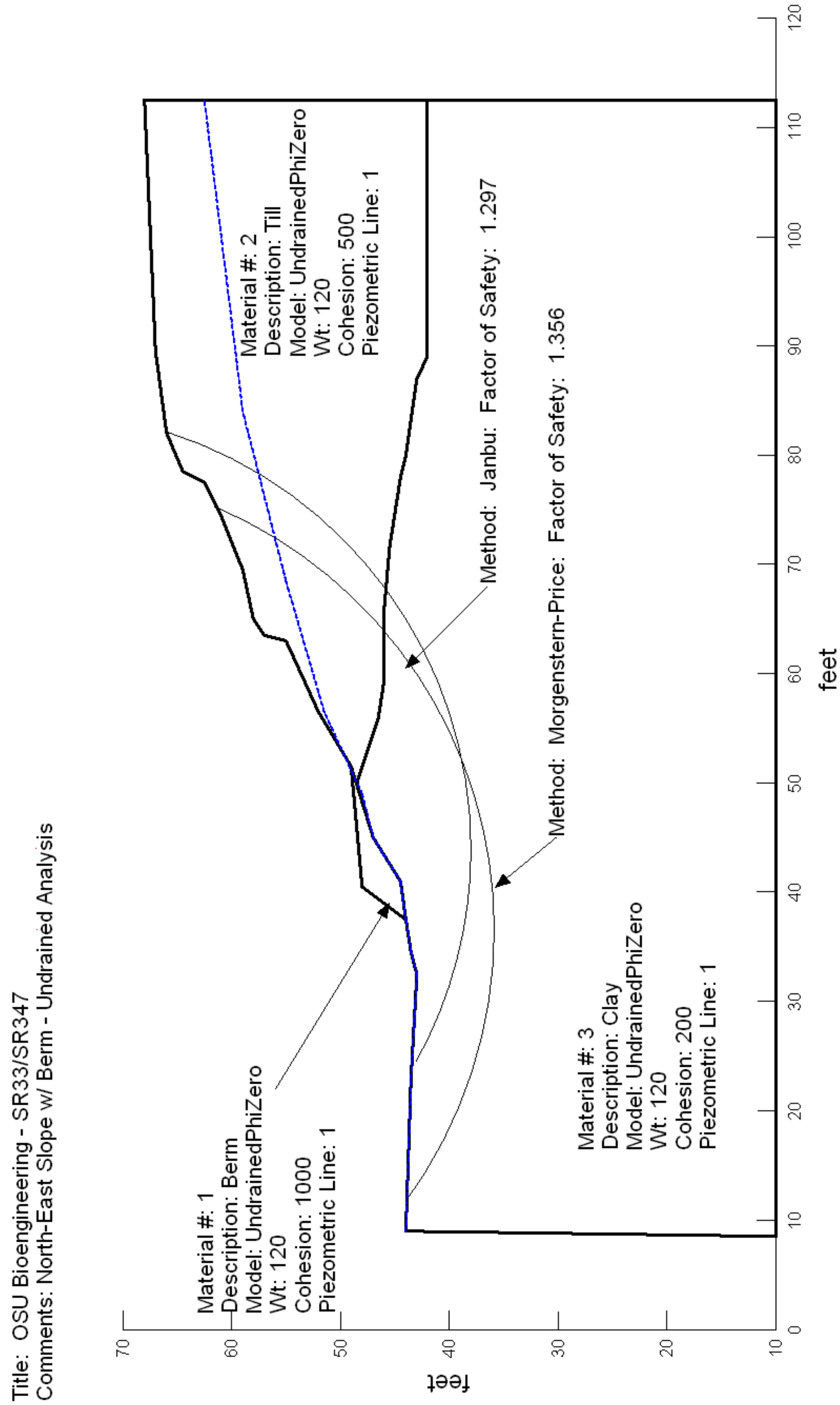




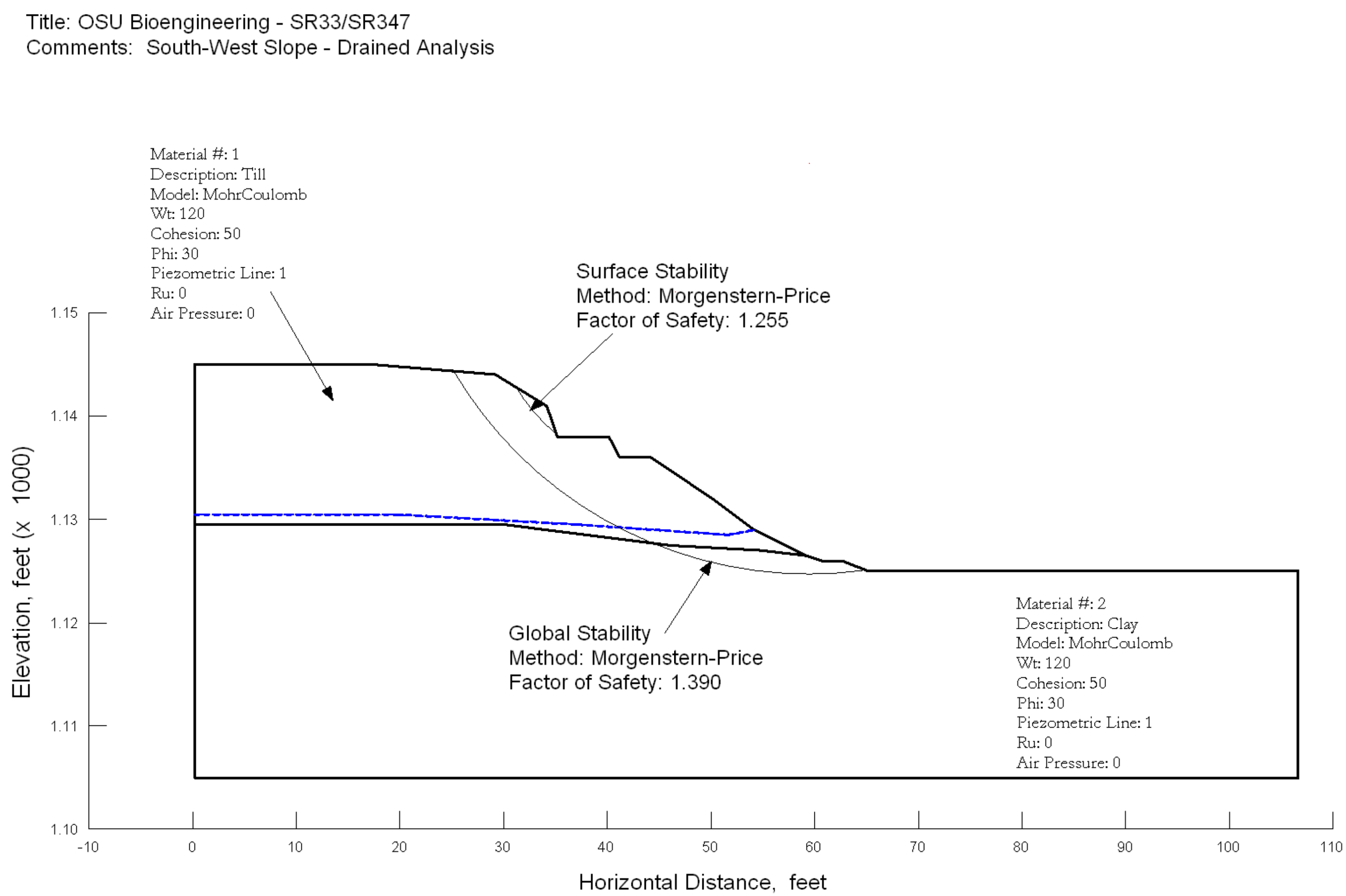
Title: OSU Bioengineering - SR33/SR347
Comments: North-East Slope - Undrained Analysis
Material #: 1
Description: Till
Model: UndrainedPhiZero
Wt: 120
Cohesion: 500
Piezometric Line: 1
Method: Morgenstern-Price: Factor of Safety: 1.340
Method: Janbu: Factor of Safety: 1.322
Material #: 2
Description: Clay
Model: UndrainedPhiZero
Wt: 120
Cohesion: 200
Piezometric Line: 1
feet
feet

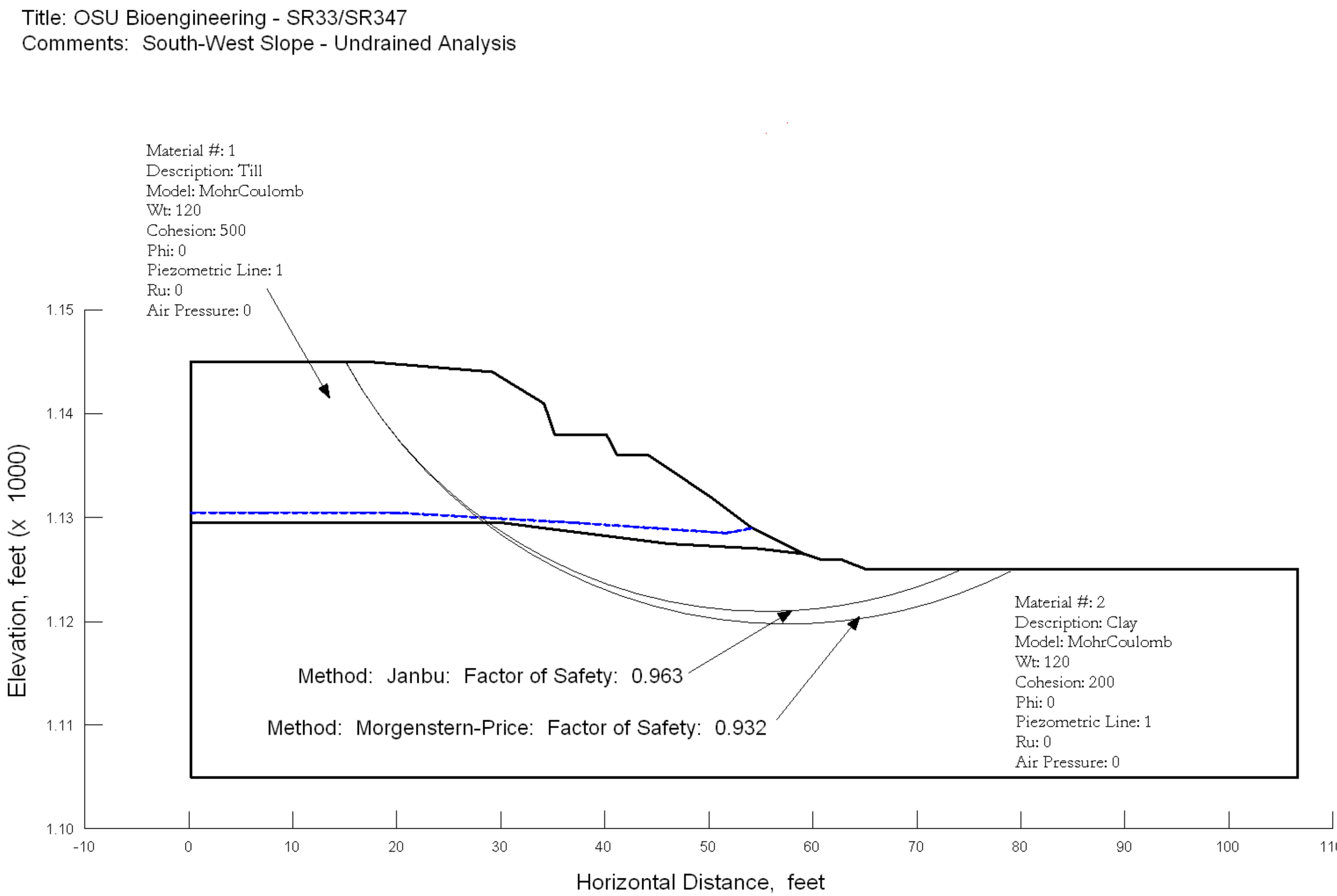
Title: OSU Bioengineering - SR33/SR347
Comments: North-East Slope w/ Berm - Drained Analysis





Title: OSU Bioengineering - SR33/SR347
Comments: North-East Slope w/ Berm - Undrained Analysis
Material #: 1
Description: Berm
Model: UndrainedPhiZero
Wt: 120
Cohesion: 1000
Piezometric Line: 1
Material #: 2
Description: Till
Model: UndrainedPhiZero
Wt: 120
Cohesion: 500
Piezometric Line: 1
Material #: 3
Description: Clay
Model: UndrainedPhiZero
Wt: 120
Cohesion: 200
Piezometric Line: 1
Method: Janbu: Factor of Safety: 1.297
Method: Morgenstern-Price: Factor of Safety: 1.356
feet
feet
0
10
20
30
40
50
60
70
80
90
100
110
120
10
20
30
40
50
60
70


Title: OSU Bioengineering - SR33/SR347
Comments: South-West Slope - Drained Analysis
Material #: 1
Description: Till
Model: MohrCoulomb
Wt: 120
Cohesion: 50
Phi: 30
Piezometric Line: 1
Ru: 0
Air Pressure: 0
Surface Stability
Method: Morgenstern-Price
Factor of Safety: 1.255
Global Stability
Method: Morgenstern-Price
Factor of Safety: 1.390
Material #: 2
Description: Clay
Model: MohrCoulomb
Wt: 120
Cohesion: 50
Phi: 30
Piezometric Line: 1
Ru: 0
Air Pressure: 0
Elevation, feet (x 1000)
1.15
1.14
1.13
1.12
1.11
1.10
-10
0
10
20
30
40
50
60
70
80
90
100
110
Horizontal Distance, feet


Title: OSU Bioengineering - SR33/SR347
Comments: South-West Slope - Undrained Analysis
Material #: 1
Description: Till
Model: MohrCoulomb
Wt: 120
Cohesion: 500
Phi: 0
Piezometric Line: 1
Ru: 0
Air Pressure: 0
Material #: 2
Description: Clay
Model: MohrCoulomb
Wt: 120
Cohesion: 200
Phi: 0
Piezometric Line: 1
Ru: 0
Air Pressure: 0
Method: Janbu: Factor of Safety: 0.963
Method: Morgenstern-Price: Factor of Safety: 0.932
Elevation, feet (x 1000)
1.15
1.14
1.13
1.12
1.11
1.10
-10
0
10
20
30
40
50
60
70
80
90
100
110
Horizontal Distance, feet

APPENDIX B

DATA AND ANALYSES: MUSKINGUM COUNTY I-70/SR-83

APPENDIX B CONTENTS

Climate Data for Station: 311997 CAMBRIDGE, OH - Source: MRCC/NOAA

Precipitation Summary
1971-2000 Normals

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
Precip	2.73	2.30	3.01	3.34	3.95	4.03	4.25	3.93	2.99	2.56	3.24	2.83	39.16

Precipitation Summary
1998-2007 Monthly (in inches)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
1998	3.66	2.61	2.26	4.69	3.89	15.4	3.19	1.97	3.93	3.50	1.59	2.05	48.74
1999	7.15	2.45	2.11	5.08	2.67	1.77	4.06	5.98	2.41	1.71	3.58	2.28	41.25
2000	2.88	3.15	3.25	3.72	4.68	2.99	1.77	3.27	3.49	2.93	1.70	3.02	36.85
2001	1.93	1.40	2.82	4.75	3.64	3.84	3.55	5.14	2.03	3.47	3.42	–	--
2002	2.00	0.96	3.63	3.72	5.44	6.18	2.20	1.70	3.81	4.34	3.01	2.40	--
2003	2.20	3.93	1.94	2.19	5.85	3.30	6.13	6.23	5.74	2.95	4.02	2.88	47.36
2004	5.52	1.82	3.04	4.58	7.18	5.97	–	5.54	12.43	2.47	3.97	3.61	--
2005	7.98	1.87	3.45	4.53	3.88	0.67	2.30	5.49	2.66	3.76	3.76	1.33	41.68
2006	4.04	1.48	2.09	4.29	3.87	4.04	6.53	3.05	5.23	6.56	1.73	1.60	44.51
2007	5.15	2.15	6.41	2.95	1.53	2.87	5.15	4.83	1.82	2.41	2.45	4.41	42.13

Climate Data for Station: 311997 CAMBRIDGE, OH - Source: MRCC/NOAA

Temperature Summary
1971-2000 Normals

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
Max °F	37.9	42.5	53.4	64.9	74.5	81.8	85.0	83.3	76.8	65.9	53.3	42.3	63.5
Min °F	20.3	22.7	30.8	39.3	49.0	57.8	62.2	60.9	53.9	41.9	33.8	25.4	41.5
Mean °F	29.1	32.6	42.1	52.1	61.8	69.8	73.6	72.1	65.4	53.9	43.6	33.9	52.5

Growing Season Summary
Derived from 1971-2000 Averages

Base Temp °F	Date of Last Spring Occurrence Median	Early	90%	10%	Late	Date of First Fall Occurrence Median	Early	90%	10%	Late
32	5/2	4/10	4/20	5/20	6/12	10/11	9/24	9/29	10/21	11/4
30	4/26	4/9	4/11	5/10	6/12	10/17	10/3	10/7	10/29	11/4
28	4/15	3/24	4/3	4/28	5/4	10/27	10/3	10/8	11/12	11/21
24	4/8	3/16	3/19	4/21	4/27	11/5	10/14	10/21	11/23	12/2
20	3/24	3/12	3/13	4/10	4/14	11/19	10/21	11/3	12/13	12/17
16	3/13	2/8	2/18	3/27	4/10	12/3	11/5	11/14	12/23	1/2

State of Ohio
Department of Transportation
Office of Geotechnical Engineering

1/3

LOG OF BORING

Date Started 3/8/05 Sampler: Type SS Dia. 1 3/8 Water Elev. DRY Project Identification: MUSKINGUM
Date completed 3/8/05 Approx. MUS-70-26.87
Boring No. B-1 * Station & Offset 732+49, 271' LT. (APPROX.) Surface Elev. 952.0' LANDSLIDE INVESTIGATION (RESEARCH PROJECT)

Elev.	Depth	Std. Pen./ R.Q.D.	Rec. ft	Loss ft	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
952.0	0														
951.5		AUGERED			TOPSOIL	-	-	-	-	-	-	-	-	-	VISUAL
	2														
	4	AUGERED			BROWN & RED CLAY (DRILLER'S DESCRIPTION)	-	-	-	-	-	-	-	-	-	VISUAL
947.0	6	6/8/12			BROWN & RED GRAVELLY CLAY (REC. 1.5')	1	13	5	7	32	43	37	16	21	A-6b
	8														
942.0	10				ATEMPTED TO TAKE A PRESS SAMPLE BUT FAILED										
	12														
	14														
937.0	16	21/27/31			RED SILTY GRAVELLY SAND W/BOULDERS	2	37	25	15	14	9	43	9	10	A-2-5
	18														
932.0	20														
	22	14/24/26			RED SANDY CLAY (REC. 1.5')	3	13	8	9	29	41	42	20	19	A-7-6
	24														
927.0	26	14/15/18			RED SILTY CLAY	4	7	4	5	35	49	40	19	21	A-6b
	28														
922.0	30														
920.5	—	11/13/14			RED & BROWN SILTY CLAY (REC. 1.5')	5	9	4	5	33	49	40	19	22	A-6b

BOTTOM OF BORING

Particle Sizes: Agg= >2.00mm, Coarse Sand= 2.00-0.42mm, Fine Sand= 0.42-0.074mm, Silt= 0.074-0.005mm, Clay= <0.005mm

* NOTE: INCLINOMETER TUBING INSTALLED IN THIS HOLE.

State of Ohio
Department of Transportation
Office of Geotechnical Engineering

2/3

LOG OF BORING

Date Started 3/9/05 Sampler: Type SS Dia. 1 3/8 Water Elev. DRY Project Identification: MUSKINGUM
Date completed 3/9/05 Approx. MUS-70-26.87
Boring No. B-2 * Station & Offset 732+38, 154' LT. (APPROX.) Surface Elev. 995.5' LANDSLIDE INVESTIGATION (RESEARCH PROJECT)

Elev.	Depth	Std. Pen./ R.Q.D.	Rec. ft	Loss ft	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
							Physical Characteristics								
995.5	0														
	2														
	4														
990.5	6	4/5/6			BROWN AND RED SILT AND CLAY (REC. 1.5')	6	8	4	7	40	41	34	14	22	A-6a
	8														
985.5	10	P.S. **			BROWN SANDY CLAY (DRILLER'S DESCRIPTION) (REC. 2.0')	7-P	-	-	-	-	-	-	-	-	VISUAL
	12														
	14														
980.5	16	4/7/8			BROWN AND RED CLAY	8	13	3	3	38	43	42	22	17	A-7-6
	18														
975.5	20	P.S. **			BROWN AND RED CLAY (DRILLER'S DESCRIPTION) (REC. 1.6')	9-P	-	-	-	-	-	-	-	-	VISUAL
	22														
	24														
970.5	26	5/9/10			BROWN AND RED CLAY (REC. 1.5')	10	0	0	2	41	57	41	22	20	A-7-6
	28														
965.5	30	P.S. **			BROWN AND RED CLAY (DRILLER'S DESCRIPTION) (REC. 1.9')	11-P	-	-	-	-	-	-	-	-	VISUAL
963.5	32														

BOTTOM OF BORING

Particle Sizes: Agg= >2.00mm, Coarse Sand= 2.00-0.42mm, Fine Sand= 0.42-0.074mm, Silt= 0.074-0.005mm, Clay= <0.005mm

* NOTE: WATER MONITORING WELL INCLUDING 2.0' OF SCREEN & 32.0' OF SOLID INSTALLED IN THIS HOLE.
** NOTE: P.S. = PRESS SAMPLE. (THESE SAMPLES WERE HANDED OVER TO THE OHIO STATE UNIVERSITY.)

State of Ohio
Department of Transportation
Office of Geotechnical Engineering

3/3

LOG OF BORING

Date Started 3/9/05 Sampler: Type SS Dia. 1 3/8" Water Elev. - Project Identification: MUSKINGUM
Date completed 3/9/05 Approx. MUS-70-26.87
Boring No. B-3 * Station & Offset 741+42, 112' LT. (Approx.) Surface Elev. 986.0' LANDSLIDE INVESTIGATION (RESEARCH PROJECT)

Elev.	Depth	Std. Pen./ R.Q.D.	Rec. ft	Loss ft	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
986.0	0														
	2														
	4														
981.0	6	10/11/12			RED AND BROWN CLAY (REC. 1.5')	12	14	2	1	34	49	40	21	17	A-6b
977.5	8														
976.0	10	P.S. **			RED & BROWN CLAY (DRILLER'S DESCRIPTION) (REC. 1.5')	-	-	-	-	-	-	-	-	-	VISUAL

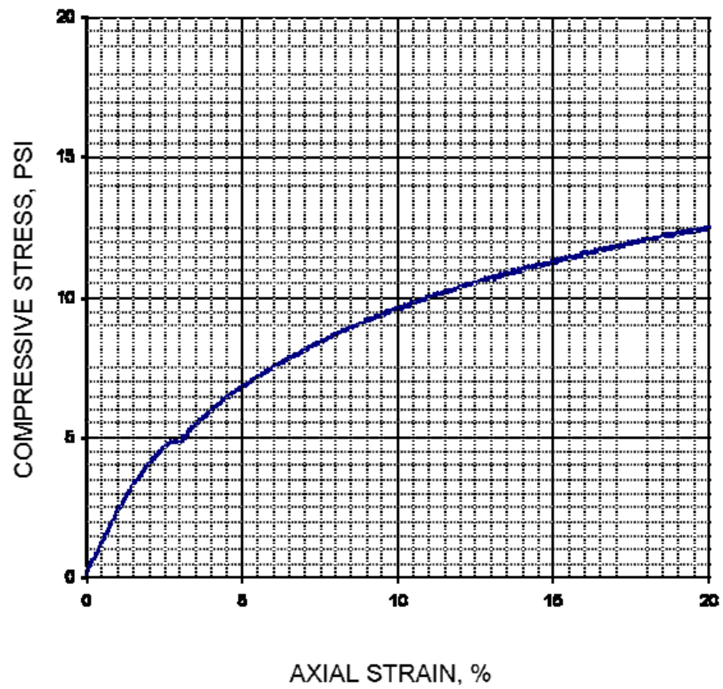
BOTTOM OF BORING

Particle Sizes: Agg= >2.00mm, Coarse Sand= 2.00-0.42mm, Fine Sand= 0.42-0.074mm, Silt= 0.074-0.005mm, Clay= <0.005mm

* NOTE: WATER MONITORING WELL INCLUDING 2.0' OF SCREEN & 10.0' OF SOLID INSTALLED IN THIS HOLE.

** NOTE: P.S. = PRESS SAMPLE. (THIS SAMPLE WAS HANDED OVER TO THE OHIO STATE UNIVERSITY.)

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY

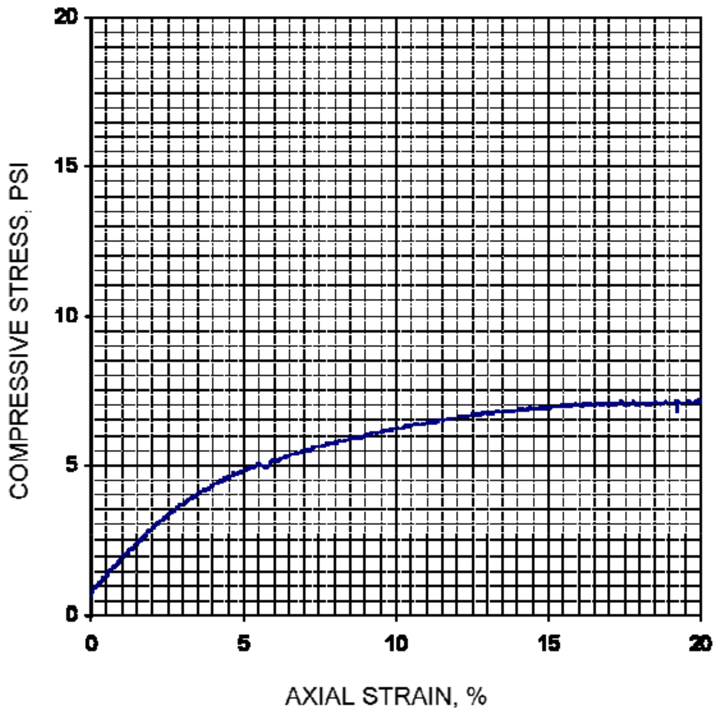


DESCRIPTION	red clay with few small stones				INITIAL	HEIGHT	3.1 in
SPECIMEN TYPE	undisturbed					DIAMETER	1.488 in
AXIAL STRAIN RATE	1%/min					HEIGHT/DIAMETER	2.08
SHEAR STRENGTH	5.65 psi	UCS	11.29 psi			WATER CONTENT	29.05%
STRAIN AT FAILURE	15%					DRY DENSITY	-
CLASSIFICATION	-					VOID RATIO	-
LL -	PL -	PI -	G_S 2.65			SATURATION	-

REMARKS G_s assumed	PROJECT	Bio-Engineering for Land Stabilization		
	AREA	Muskingum County—I70/SR83		
	BORING ID	TP-1	SAMPLE NO.	-
	DEPTH/ELEV.	1.5 ft	DATE	February 17, 2006
	TECH	CMK	CHECKED	
	UNCONFINED COMPRESSION TEST REPORT			

THE OHIO STATE UNIVERSITY SOIL MECHANICS LABORATORY

SPECIMEN AT FAILURE



DESCRIPTION	red clay with few small stones			INITIAL	HEIGHT	3.25 in
SPECIMEN TYPE	undisturbed				DIAMETER	1.489 in
AXIAL STRAIN RATE	1%/min				HEIGHT/DIAMETER	2.18
SHEAR STRENGTH	3.47 psi	UCS	6.93 psi		WATER CONTENT	-
STRAIN AT FAILURE	15%				DRY DENSITY	-
CLASSIFICATION	-				VOID RATIO	-
LL -	PL -	PI -	G_S 2.65		SATURATION	-

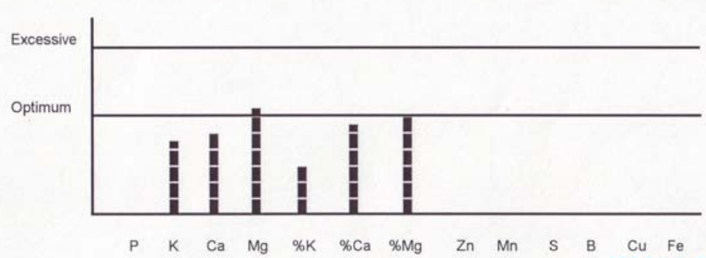
REMARKS G_s assumed	PROJECT	Bio-Engineering for Land Stabilization		
	AREA	Muskingum County—170/SR83		
	BORING ID	TP-2	SAMPLE NO.	B
	DEPTH/ELEV.	1.5 ft	DATE	February 22, 2006
	TECH	CMK	CHECKED	
	UNCONFINED COMPRESSION TEST REPORT			

CAL MAR SOIL TESTING LABS

Sample Submitted By	Grower	Date of Report	Report Number	Record Number	Lab #	Field	Sample
CHRISTOPHER KOKESH 6893 MEADOW OAK DR. COLUMBUS, OH 43235	MUSKINGUM CO I 70/SR 83	02/13/06	278.014	20184	02233B		

Organic Matter %	Phosphorus Bray P1	Potassium K	Magnesium Mg	Calcium Ca	pH		Cation Exchange Capacity	Percent Base Saturation			Sulfur S	Zinc Zn	Manganese Mn	Boron B	Copper Cu	Iron Fe
					Soil	Buffer		%K	%Mg	%Ca						
1.7	5	307	1513	6830	8.4		24.1	1.6	26.1	71						

Crop/Yield	Previous Crop	Nitrogen Lbs/A	P_2O_5 Lbs/A	K_2O Lbs/A	Lime		Magnesium Mg Lbs/A	Sulfur S Lbs/A	Zinc Zn Lbs/A	Manganese Mn Lbs/A	Boron B Lbs/A	Copper Cu Lbs/A	Iron Fe Lbs/A
					T/A	Type							
SHRUBS/BUSHES 1 NA		120	150	135	0.0								
HARDWOOD TREES 1 NA		120	150	135	0.0								



Comments:

Test results reported in pounds per acre

Soluble salts: 0.375 mmhos/cm

Na #/A = 167
% Na = 1.5

130 South State Street
Westerville, OH 43081
614-523-1005

CAL MAR
SOIL TESTING LABS

02/13/06
MUSKINGUM CO
I 70/SR 83

Sample submitted by:
CHRISTOPHER KOKESH

6893 MEADOW OAK DR.
COLUMBUS, OH 43235

Test Results

Results are in pounds per acre except pH or as noted

LAB NUMBER 278.014 SAMPLE ID	pH Soil	pH Buffer	Phosphorus (P)	Potassium (K)	Magnesium (Mg)	Calcium (Ca)
2233	8.4		5	307	1513	6830

% Organic Matter	Cation Exchange Capacity	PERCENT BASE SATURATION %Potassium	%Magnesium	%Calcium	%Hydrogen
1.7	23.8	1.7	26.5	72	

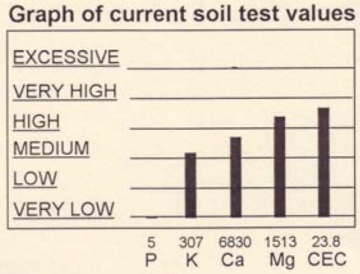
Recommendations

For optimum growth of your shrubs/bushes we recommend adding the following

NITROGEN (N)	PHOSPHORUS (P205)	POTASSIUM (K2O)
POUNDS / 1000 Sq. Ft. per season	POUNDS / 1000 Sq. Ft. per season	POUNDS / 1000 Sq. Ft. per season
2.8	3.4	3.1

Sulfur- Lbs/1000 Sq.Ft. SOIL DEPTH (inches)		
3	5	
6	11	
9	16	
12	21	
Type		

Graph of current soil test values



Please see notes on page two
page 1

CAL MAR SOIL TESTING LABS, 130 SOUTH STATE STREET, WESTERVILLE, OH 43081
PH: 614-523-1005 FAX: 614.523.1004

CAL MAR

SOIL TESTING LABS

02/13/06

MUSKINGUM CO

I 70/SR 83

Sample submitted by:
CHRISTOPHER KOKESH

6893 MEADOW OAK DR.
COLUMBUS, OH 43235

Notes

Soil pH

Your pH value is very high and may be affecting plant growth. It will be necessary to apply sulfur or other acidifying products in order to lower the soil pH. Sulfur applications should be worked into the soil. If the plant material is already established, use the shallow application amount (3-inches) recommended for now. The next time the soil and sulfur can be mixed thoroughly, apply the remaining amount (i.e.: the difference between the 3-inch recommendation and the 9-inch recommendation).

Nitrogen

Your nitrogen recommendation is based upon the plants to be grown and the soil organic matter. Your nitrogen recommendation has been increased slightly due to a low soil organic matter percentage. Nitrogen should be worked into the soil prior to planting when possible. For established plants, apply only a portion of the recommended nitrogen throughout the growing season. Be very cautious about applying nitrogen in very warm or dry weather, since this can damage the plant tissue.

Phosphorus

Your phosphorus value is very low and it may be affecting plant growth. It will be necessary to apply phosphorus in order to raise the soil phosphorus level. Phosphorus applications should be worked into the soil prior to planting if possible. If the plant material is already established, apply one-half of the recommended phosphorus to the soil surface, for now. The next time soil and phosphorus can be mixed thoroughly, apply the remaining amount recommended.

Potassium

The current potassium level is just slightly below the optimum range for the indicated plant type. Your current soil potassium level is not adversely affecting the health of your plants. You may apply a small amount of potassium at this time to adjust the level in your soil. If practical, when you apply potassium, try to thoroughly mix the application into the soil or water in after application.

These notes refer to the recommendations and results on page one. Additional general information is enclosed.

page 2

CAL MAR SOIL TESTING LABS, 130 SOUTH STATE STREET, WESTERVILLE, OH 43081
PH: 614-523-1005 FAX: 614.523.1004

Live Pole Vertical Pullout Test Data.

Pole ID (column-#)	Pullout Test (pull)	Measured Resistance for 0.1 ft. of Pullout (lbf)								
8-9	1	207	193	165	150	140	133	120	86	
	2	133	100	81	63	50	38	31		
8-10	1	190	186	151	144	171	180	188	177	163
	2	128	136	135	138	137	138	143	125	
	3	80	81	77	78	78	22			
8-11	1	286	210	186	165	108	37	38	39	
8-12	1	357	294	278	262	241	237	223		
	2	146	143	157	177	206	145	141	131	111
	3	93	84	65	60	56	53	57	56	39
9-2	1	31	223	226	204	155				
	2	159	145	150	149	149	114			
	3	85	106	78	78	60	55			
	4	45	40	35	30	30	25	22		
9-3	1	98	90	85	77	77	73	70	67	62
	2	30	36	31	25	24	23	18	23	14
9-4	1	178	171	137	90	79	74			
	2	45	51	56	55	30				
9-5	1	66	62	63	60	57	51	49		
	2	27								
10-3	1	90	110	124	121	133	154			
	2	126	170	138	126	118	112	84		
	3	78	77	74	62	26	20	18	18	
10-4	1	370	287	300	238	200				
	2	207	156	120	93	66	56	34	25	
10-5	1	250	243	216	150					
	2	103	44	33	31	30	22			
10-8	1	148	135	129	134	141	121	121	114	
	2	118	118	105	104	116	122	121	94	
	3	123	86	65	53	41	33	31	29	28
10-9	1	129	156	147	153	158	154	165		
	2	189	185	163	154	155	157	148	134	
	3	142	117	100	78	58	55	52	50	45
10-10	1	367	304	295	286	236				
	2	235	233	231	217	190	180	83		
	3	78	58	49	44	50	55	63	37	28

Live Pole Vertical Pullout Test Data (continued).

Pole ID (column-#)	Pullout Test (pull)	Measured Resistance for 0.1 ft. of Pullout (lbf)								
11-2	1	212	283	304	320	300	291			
	2	218	220	208	201	198				
	3	173	205	182	174	165	149	123		
	4	89	112	111	110	108	131	112	123	
	5	60	57	56	41	32	27	21	13	
11-3	1	187	187	160	148	129	117	110	105	
	2	72	99	103	98	91	81	74	67	
	3	51	55	57	52	50	48	47		
11-4	1	58	71	75	70	66				
	2	44	36	37	38	37	30			
11-8	1	283	308	315	272					
	2	166	281	245	212	211	214			
	3	250	244	255	247	236	234	217		
	4	233	229	226	198	189	174			
	5	181	145	134	118	101	53			
11-9	1	299	327	326	292	243	187	144	86	
	2	61	49	42	35	34	31	28	30	
12-3	1	385	306	235	190	152				
	2	113	111	97	84	44	43	24		
12-4	1	700	480	468	456	429				
	2	491	476	507	480	442				
	3	391	366	330	326	304				
	4	283	250	239	201	220	194			
	5	122	120	103	54					
13-2	1	148	182	235	290	480				
	2	155	149	133	1112	94				
	3	98	99	101	96	84	69	60	55	48
13-3	1	316	250	169	146	139	114			
	2	109	108	95	80	74	74	73	70	57
13-4	1	68	63	56	39	34	20			
14-3	1	312	392	403	385	355	331	282	221	
	2	212	170	142	120	180	98	89		
	3	81	77	81	104	127				
14-4	1	210	190	170	161	141	130	114		
	2	81	89	97	97	99	98	99	52	45
17-3	1	152	160	129	147	163	145			
	2	78	88	83	112	129	157	165	149	
	3	64	82	69	78	71	86	64	70	

Live Pole Vertical Pullout Test Data (continued).

Pole ID (column-#)	Pullout Test (pull)	Measured Resistance for 0.1 ft. of Pullout (lbf)						
18-2	1	108	189	64	34	24	24	
18-3	1	212	195	182	143			
	2	104	132	132	120	89		
	3	74	83	78	71	96	120	
19-2	1	185	237	294	251	271	264	
	2	239	219	168	157	132	97	85

Live Pole Vertical Pullout Test Pole Dimensions and Calculations.

Pole ID (column-#)	Bottom Diameter (in.)	Top Diameter (in.)	Pole Length (ft.)	Depth of Embedment (ft.)	Pullout (lb)	Pullout f (psf)	s_u * (psf)
8-9	2.83	2.18	4.92	3.80	207	1502	3003
8-10	2.22	1.63	4.84	3.96	190	2247	4494
8-11	2.62	1.97	5.15	4.39	286	2176	4352
8-12	2.13	1.43	5.16	4.34	357	4476	8952
9-2	2.89	2.54	4.64	3.58	226	1525	3050
9-3	1.79	1.54	4.62	3.66	98	1717	3434
9-4	1.87	1.39	5.18	3.99	178	2880	5761
9-5	2.31	1.66	4.64	3.50	66	811	1622
10-3	2.72	2.00	4.77	3.70	170	1414	2828
10-4	1.72	1.41	5.57	4.47	370	5961	11922
10-5	2.00	1.67	5.02	4.30	250	3086	6171
10-8	2.39	1.69	4.81	3.99	148	1543	3085
10-9	2.29	1.61	5.12	4.00	189	2114	4228
10-10	2.48	1.83	5.25	4.26	367	3216	6431
11-2	2.42	1.61	4.58	3.62	320	3676	7351
11-3	2.11	1.65	4.99	3.96	187	2330	4661
11-4	1.96	1.31	5.00	4.04	75	1181	2363
11-8	2.40	1.80	5.12	4.55	315	2789	5578
11-9	2.21	1.67	5.32	4.44	327	3428	6856
12-3	2.59	1.77	5.20	4.39	385	3194	6387
12-4	2.68	2.07	5.66	4.78	700	4576	9151
13-2	1.84	1.09	5.17	4.17	480	8927	17855
13-3	2.55	1.86	5.27	4.32	316	2609	5218
13-4	1.87	1.62	5.34	4.03	68	981	1962
14-3	2.74	1.88	5.10	4.03	403	3183	6365
14-4	2.19	1.73	5.21	4.23	210	2268	4536
17-3	1.62	1.13	5.09	4.55	165	3387	6775
18-2	1.33	0.95	5.06	3.95	189	6283	12565
18-3	1.53	0.89	5.58	4.55	212	5304	10609
19.2	1.92	1.45	5.07	4.16	294	4344	8687
AVERAGE					258	3104	6208

Note: * Calculation methodology presented in section 3.7.

APPENDIX C

WU, T.H., FOX, P.J., TRENNER, B.R., KOKESH, C.M., BEACH, K., AND BARKER, D.H. (2008)

Soil-Bioengineering for Slope Stabilization in Ohio

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ABSTRACT: This paper describes the use of live poles for the stabilization of shallow slope failures in clay-silt soils. Design and construction techniques are summarized. Long-term monitoring of the slope performance is currently in progress. Measured performance data includes displacement, pore pressure and suction, moisture content, survival of willow poles, and lateral resistance provided by the poles. Preliminary results and potential benefits and problems are presented.

INTRODUCTION

Soil-bioengineering has been used mostly for erosion control but has been shown to be successful in the stabilization of shallow slope failures. A recent development is the use of live poles, with diameters up to 50 mm and lengths up to 2 m, to stabilize shallow slips on road embankments in the UK (Barker 1997). This paper describes the use of live (i.e., willow and poplar) poles for the stabilization of shallow slips on three slopes constructed by the Ohio Department of Transportation (ODOT) as a research project. The purpose of the project is to investigate soil-bioengineering as a cost-effective alternative to conventional stabilization methods.

The lateral resistance of the poles contributes to shearing resistance along the sliding surface and increases the safety factor. In addition, the root systems of established poles can contribute an additional component to the shearing resistance. On the other hand, if the poles die and decay over time, their contribution to stability will also disappear. Thus, the long-term vitality of the poles is critical. Long-term monitoring of slope performance is currently being conducted and results obtained to date are presented. Construction time and costs are compared with those of conventional repair methods used by ODOT.

SITE CONDITIONS

Principle features of the three test slopes are given in Table 1. Profiles of the slopes are shown in Figs. 1-3. Indications of instability include bulges and cracks and soil blocks that have moved down slope. These blocks are 0.5 -1.0 m wide and less than 0.6 m in depth and the soil is generally wet.. Movements of about 50 mm took place during winter-spring 2004-2005. The soils at the sites are CL to ML. Strength properties from laboratory and in-situ tests are also given in Table 1.

Case histories of slope failures in Ohio (Wu et al. 1993) have shown that slopes on stiff clays or compacted clays generally deteriorate with age. The layer near the surface is subject to wet-dry and freeze-thaw cycles and its strength gradually approaches the state of $c' = 0$, $\phi' > 0$. During the wet season, the ground becomes saturated. Under vertical seepage, a 2H:lV slope would have a safety factor close to l.0. Local slips develop because of non-uniform soil and seepage conditions. Such slips are not usually noticed by maintenance crews. Over a period of several years, however, these slips grow, coalesce, and extend to greater depths. The condition becomes critical when the upper limit of a failure reaches the shoulder. To protect the roadway, the conventional repair measure is to excavate the soft material and replace with compacted fill, which is fairly costly. Then the cycle is repeated. The service life of such slopes ranges between 10 to 20 years, depending on the site conditions.

DESIGN AND CONSTRUCTION

To stabilize the slopes at New Concord and East Liberty, live poles were installed in the zones with shallow slips. At Marysville, the slips are located close to the bridge abutment and ODOT decided to repair this section with their conventional method. Willow poles were installed in the adjacent section, which had no slips, as a preventive measure. In the present study, live poles approximately 2 m long and 25-50 mm in diameter were installed vertically in a grid pattern approximately 1 m on center. The poles consisted of stems cut from willow and poplar trees located within 20 km from the sites, during early spring (March-April) and prior to sprouting of leaves. The poles were trimmed to the required dimensions. The installation at New Concord was delayed until late May 2005 and it became necessary to store the

Table 1. Principal Features of the Test Slopes.

Highway Routes	I-70/SR83	US33/SR347	US33/US36
Location	New Concord, Ohio	E. Liberty, Ohio	Marysville, Ohio
Type	embankment fill	cut slope	embankment fill
Pre-existing condition	shallow slides	shallow and deep rotational slides	shallow slides
Soil Type	compacted residual clay	till	compacted till
Cohesion, c’	0-30 kPa	0-20 kPa	
Friction angle, ϕ’	27-30°	30-34°	
Undr. strength, s_u	20-40 kPa	<50 kPa	
Pole species	willow, poplar	willow	willow

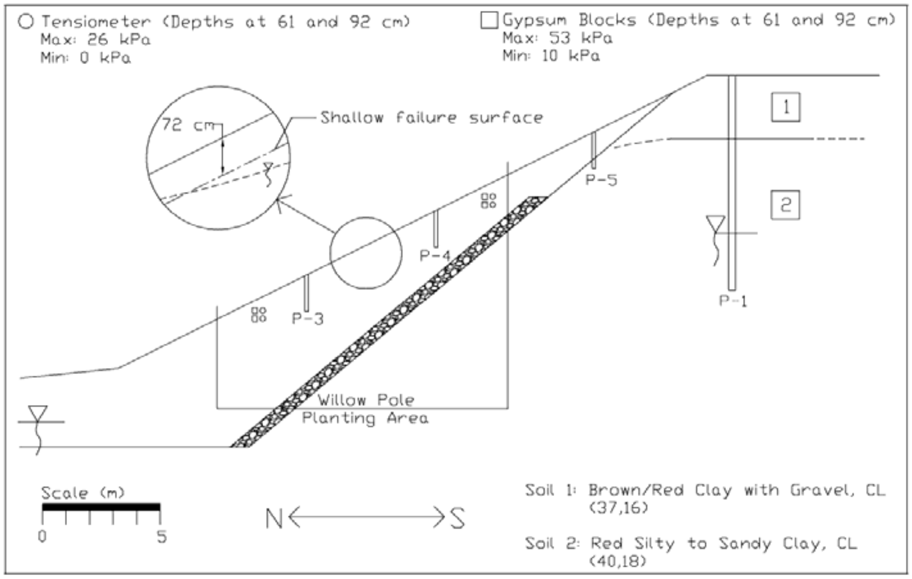

Fig. 1. New Concord Site.

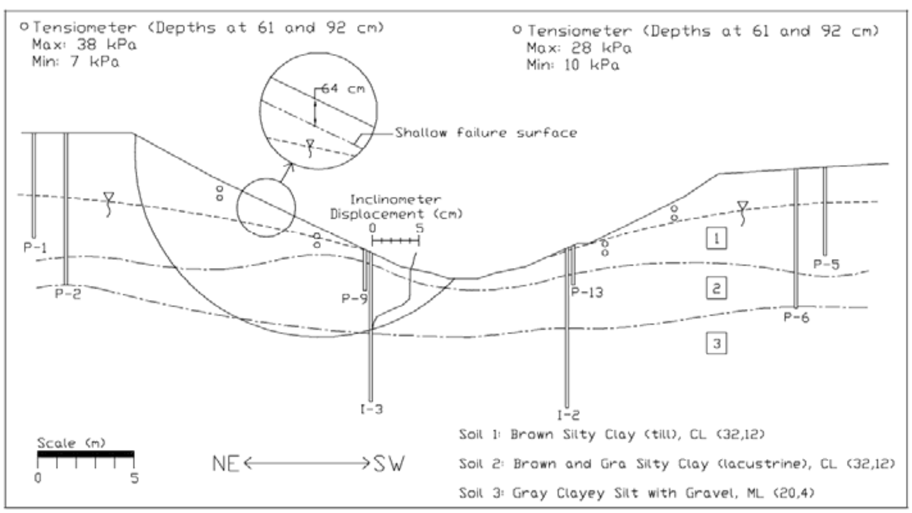

Fig. 2. East Liberty Site.

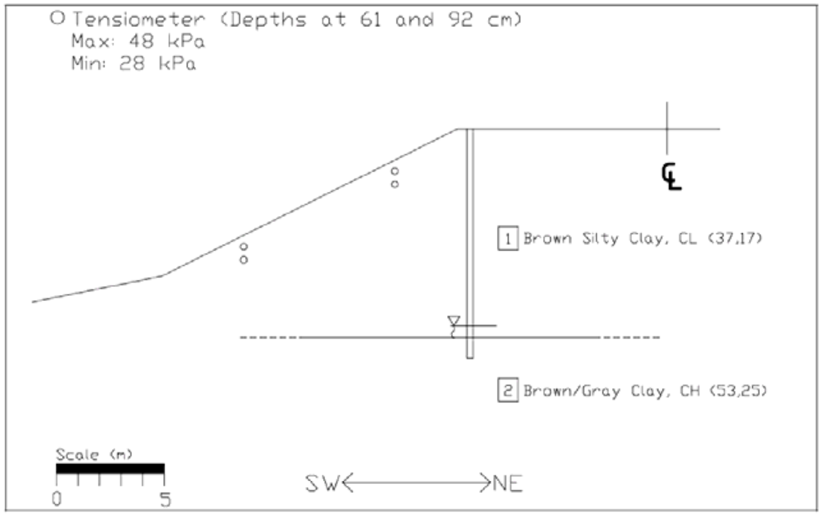

Fig. 3. Marysville Site.

harvested stems in refrigeration (2°C) as has been done in the UK (Barker 1997, Steele et al. 2004) until installation. At the other two sites the poles were transported to the site and installed within two days after harvest according to the procedures of Gray and Sotir (1996) and Barker (1997).

All poles were placed in pre-made 75 mm diameter holes. Two installation methods were used. Method A is considered to be the best method and is also the most labor-intensive. In this method, each pole is cut to a bevel at the bottom, the top is wrapped with wire to prevent splitting, the pole is driven 150 mm into the bottom of the hole with a mallet, and then the top is cut off, rewired and painted with a protective sealant. For Method B, which is simpler and less labor-intensive, the poles are simply dropped into pre-made holes, backfilled and painted with a protective sealant. Other details are shown in Fig. 4. Both installation methods were used at East Liberty to provide a side-by-side comparison. Only Method A was used at New Concord and only Method B was used at Marysville. The sand-gravel backfill and vent pipe provides aeration and should encourage root growth. To test the importance of aeration, about half of the holes at New Concord were backfilled with the embankment material and had no vent. A 0.6 m diameter biodegradable cover was placed around each pole to reduce competition from other vegetation (e.g., grass).

The infinite slope model was used to estimate the stability of the shallow slides before and after stabilization. Measured displacements (next section) indicated that the depth of the slip surface was at most 0.6 m and the measured suction was near 0 during the wet season, which indicated that the slopes were saturated with vertical seepage at East Liberty. At New Concord, piezometer levels suggested a perched water table near the surface and seepage parallel to the slope. The shear strengths in Table 2 represent the estimated "softened" strength (Skempton 1970). Calculated safety factors for the initial condition (i.e., before stabilization) are given in Table 2.

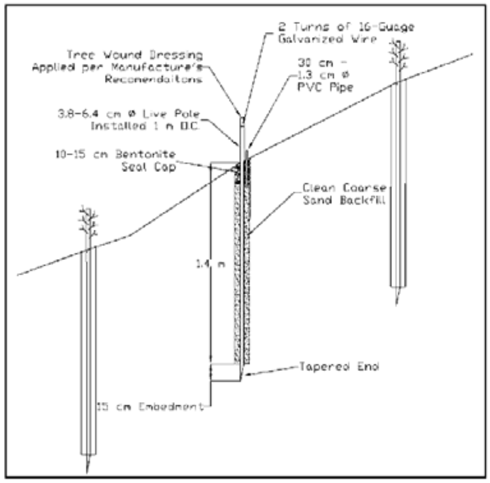

Fig. 4. Method A for live pole installation.

Table 2. Strength Parameters and Safety Factors at New Concord and East Liberty Sites.

Site	Seepage	c'	ϕ'	F_s (initial)	F_s (final)
New Concord	parallel to slope	2.4 kPa	27°	1.00	1.08
East Liberty	vertical	0	30°	1.15	1.40

Live poles installed vertically or perpendicular to the slope and extending to sufficient depth below the slip surface serve as soil reinforcements. The mechanism is analogous to that of piles. Immediately after installation, but with no root growth, the stability was calculated for the failure modes described by Vigiliante (1981) and Poulos (1995), which include the "flow mode" and "pile capacity mode". In the flow mode, the soil layer above the slip surface moves between the poles and resistance is the passive pressure against the poles. In the pile capacity mode, lateral resistance provided by the portion of the pole below the slip surface is overcome. For the flow mode, the passive pressure was calculated with the range in s_u given in Table 1. The s_u of the undisturbed embankment was used to calculate lateral resistance for the pile capacity mode. It was found that the flow mode controls. The increase in shear strength was taken as the pole resistance divided by the square of the pole spacing. Calculated final safety factors are approximately 1.1 and 1.4 (Table 2). This represents the most unfavorable condition. The strength of the soil is expected to increase with root growth and the safety factor should increase with time. The safety factor with respect to deep movement at East Liberty was approximately 1.0. The construction of a toe berm at the same time of pole installation should increase the safety factor to about 1.25. These safety factors are small compared to those used in practice. However, for research purposes these factors of safety are acceptable.

MEASUREMENT OF PERFORMANCE

Slope performance before and after stabilization has been monitored using inclinometers, piezometers, tensiometers, and moisture blocks. Load tests on the poles were conducted to measure vertical and lateral resistances. Survival rates have obtained been obtained periodically. Results obtained to date are summarized below.

Pore Pressure and Moisture

Shallow and deep piezometers were used to measure pore pressures at the locations shown in Figs. 1-3. Tensiometers were used to measure total soil suction at shallow depths (< 2 m). Measurements from 2005 to present show that suctions developed in the top 0.5 m during the summer and autumn. From November to March, the ground was often near saturation. This agrees with the ranges in water levels observed in piezometers as shown in Figs. 1 and 2.

Displacements

Slope indicator tubes were installed at locations shown in Figs. 1-3 to measure deep movements. No such movements have occurred at New Concord and Marysville. At East Liberty, significant horizontal displacements occurred prior to stabilization at about 3 m below the bottom of the slope. This suggests that rotational failure occurred along the deep slip surface shown in Fig. 2.

To monitor the shallow slips, copper tubes and plastic tubes 10 mm in diameter were placed in predrilled holes. The positions of their tops were monitored. Excavations were then made on one side to expose the tubes. This showed that the movements were limited to the top 0.6 m. In addition, a row of stakes were placed along the slope and their movements were monitored relative to fixed benchmarks. The measurements showed that shallow movements of 5 cm occurred during the winter of 2005-6 at New Concord and East Liberty. After the repairs at New Concord, the movements were minimal during the last wet season, although soil moisture remained high and near saturation.

Survival Rates

Observed survival rates of live poles are given in Table 3. The low survival rates for the first installation (May 05) at New Concord (50%) are attributed to the late date of installation, which was followed by a period of hot weather. In addition, the poplar poles had a very low survival rate as compared to the willow poles, which reduced the overall rate. Most of the dead poles at New Concord were replaced with willow poles during the following spring (Feb 06). These poles were installed shortly after harvest and had a good survival rate (90%). Survival rates at East Liberty are also low, with differences for the SW and NE slopes attributed to differences in soil moisture. The method of installation did not influence the survival rate. Survival rates at Marysville are high 4 months after installation. Comparison of the survival rates at East Liberty and Marysville also indicates that installation method did not influence survival rate.

Table 3. Survival Rates of Live Poles.

	New Concord		East Liberty		Marysville
			NE	SW	
Installation	May 05	Feb 06	Mar 07	Mar 07	Mar 07
1st Spring	**74%**	**90%**	**49 %**	**72 %**	**91 %**
1st Autumn	**50 %**	**90 %**			
2nd Spring	**34 %**	**90 %**			
2nd Autumn	**32 %**				

Load Tests

Vertical pull-out and lateral load tests were performed on dead poles at New Concord to verify the pile equations used for predictions. Dead poles represent the worst case scenario, which is the state with no roots. The vertical load at pullout is compared with that using the undrained soil shear strength. The lateral resistance is compared with that calculated using Broms (1964) solution. The results are shown in Table 4, where the range in calculated resistance represents the range in undrained shear strength. The measured lateral resistance is within the range of the calculated values. The low measured vertical resistance, relative to the predicted resistance, is attributed to poor contact between the pole and the backfill material.

Table 4. Measured and Calculated Resistances for Load Tests of Live Poles.

	Measured	Calculated
Lateral Resistance	1.09-2.43 kN	0.89-3.11 kN
Vertical Resistance	0.29-3.11	1.47-8.90

CONSTRUCTION COST AND TIME

At New Concord, ODOT operators graded the slope and students installed the live poles. Construction at East Liberty and Marysville was done by a private contractor. Cost and construction times for the three sites are given in Table 5. The cost and time

Table 5. Comparison of Construction Cost and Time.

Site	Method	Cost ($/m^2)*	Time (days)
New Concord	Soil-bioengineering	52	15
East Liberty	Soil-bioengineering	115	7
Marysville	Soil-bioengineering	93.5	3
Marysville	Conventional	155	60

*areas measured from plan view.

for the conventional repair at Marysville are also provided. The cost of soil-bioengineering repair is 30% lower. This cost is expected to decrease significantly as local contractors become familiar with the techniques, as indicated by the cost at New Concord.

SUMMARY AND CONCLUSIONS

Results obtained to date indicate that live poles, if they can be established, can be effective for stabilization of shallow slides at depths of approximately 1.5 m or less. The low survival rate at the East Liberty site is of concern and will be studied further. The potential benefits of soil-bioengineering are reduced time and cost of repair and minimum or zero interruption of traffic. In a larger sense, this approach represents a proactive slope maintenance strategy. It is not only economical but is a step towards sustainable development that minimizes environmental impact and uses renewable materials. The principal limitations are constraints on construction time and the procedures for installing plant materials, which are critical but unfamiliar to many civil engineers and contractors. Also, the survival of poles requires additional and maintenance beyond what is typically necessary for conventional stabilization repairs.

ACKNOWLEDGMENTS

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REFERENCES

Barker, D. H. (1997). "Live willow pole for slope stabilization on the A249 at Iwade." Report PR/CE/133/97, TRL Ltd., Crowthorne, UK.

Broms, B. B. (1964). "Lateral resistance of piles in cohesive soils." *J. Soil Mechanics and Foundations Division*, Vol. 90:27-64.

Gray, D. H., and Sotir, R. B. (1996). *Biotechnical and Soil Bioengineering Slope Stabilization.* John Wiley and Sons, New York.

Poulos, H. G. (1995). "Design of reinforcing piles in increase slope stability." *Canadian Geotechnical J.*, Vol. 32(5):808-818.

Skempton, A. W. (1970). "First-time slides in over-consolidated clays." *Geotechnique*, Vol. 20:320-324.

Steele, D. P., MacNeil, D. J., Barker, D., and McMahon, W. (2004). "The use of live willow poles for stabilizing highway slopes." Report TRL619, TRL Ltd., Berkshire, UK.

Viggiani, C. (1981). "Ultimate lateral load on piles used to stabilize landslides." *Proc., 10th Int. Conf. Soil Mechanics and Foundation Engrg.*, Stockholm, Vol. 3:555-560.

Wu, T. H., Randolph, B. W., and Huang, C-S. (1993). "Stability of shale embankments." *J. Geotechnical Engrg.,* Vol. 119(1):127-146.