To the Graduate Council:

I am submitting herewith a thesis written by Clyde Raymond Tant III entitled "Stabilization of Mine Waste Through the Application of Soil Nailing." I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfilment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

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Stabilization of Mine Waste Through the Application of Soil Nailing

A Thesis
Presented for the
Master of Science
Degree
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Clyde Raymond Tant III
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DEDICATION

This thesis is dedicated to my parents, and grandparents

Clyde Raymond Tant Jr.,

Jeannie Burns Arnold,

Clyde Raymond Tant Sr.

And

Mai Ellen Tant

without them, none of this could have happened

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I wish to thank Betsy for her love, encouragement, and for providing me with an incentive to finish this thing up.

Finally, I wish to thank a power greater than myself.

"Praise be to God, to whom all praise belong"

John Coltrane

Abstract

The traditional soil nailed structure incorporates grouted or driven nails, and a wire mesh reinforced shotcrete facing to increase the stability of a slope or wall. This thesis describes the construction and monitoring of a full-scale demonstration of nailing to stabilize coal mine spoil (waste from contour surface mining). The purpose of the investigation is to evaluate the performance of nailed slopes in mine spoil using methods proven for the stabilization of soil walls and slopes. The site in eastern Tennessee is a 8 meter high slope of dumped fill, composed of residual soil, weathered shale chips, sandstone, and coal. The slope was formed by "pre-regulatory" contour surface mining operations and served as a work bench during mining. The material varies in particle size from clay to boulders, and has a small amount of cohesion. Portions of the slope have experienced slope instability and erosion which have hampered subsequent reclamation activities.

Three different nail spacings and three different nail lengths were used in the design of the stabilized sections. A fourth unreinforced control section was excavated and later observed to have failed. The study sections were instrumented and monitored for nail stresses and slope deflection. Measurements from the instrumentation revealed slight increases in nail stress during the study period, with little slope movement detected. Following completion of the slope, an analysis of the as-built soil nailed structure was performed and a method was devised to create and analyze an equivalent slope from the as-built nail locations.

The study suggests soil nailing is an applicable stabilization method for slopes composed of mine spoil. The major advantage of soil nailing is the ability to vary the design as differing subsurface conditions are encountered. The proposed method permits variable nail spacings to be easily incorporated in most analysis methods.

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Chapter 1

Introduction

1.1 Problem Statement

Throughout the Southeastern United States, surface contour strip mining has been widely used to extract coal. Usually, the mining activity leaves behind massive spoil piles of dumped uncompacted waste. If compacted, it is done with little effort, reducing the density of the material, and often results in a steep, marginally stable slope. It is not unusual for homes, businesses, and roadways to be constructed on or near these slopes. Failures of the slopes can be a violent and massive displacement or a slow creeping failure that slowly overtakes land and roadways. Mine waste differs greatly from soil due to the random nature of the particles which can vary in size from boulders to clay. The waste may also include debris from the mining process such as roots and trees. It has become very expensive to stabilize strip mine waste slopes using tie back walls, and concrete retaining walls. Many of the slopes in question are located in remote locations making the use of tie back walls, and retaining walls more difficult to apply. An alternative is soil nailing.

Soil nailing has been proven to be a practical and successful method of stabilizing soil slopes in Europe and the United States. Soil nailing is an appealing alternative to other methods because it is an inexpensive and convenient method of slope stabilization. Since little excavation is needed, and equipment for the process is not too cumbersome, soil nailing can be used on remote and difficult sites. The cost benefits of soil nailing also make it very appealing compared to other methods. Basic equipment and supplies can be obtained easily. Although soil nailing has been widely used for the stabilization of earth slopes, its effectiveness has not been demonstrated in mine waste.

1.2 Purpose

The purpose of this research is to evaluate the performance and stability of nailed slopes in mine spoil using methods proven for the stabilization of soil walls and slopes. The application of these techniques to stabilize mine spoil materials has not been previously demonstrated or investigated, and guidelines for design are needed.

1.3 Components of Investigation

The steps to achieve the above purpose were as follows:

1. **Review literature pertinent to soil nailing and mine spoil**. The focus of the review was directed towards information pertaining to methods for the design and construction of soil nailed walls and slopes, and material properties of mine spoil in the eastern coal province. (Chapter 2)

2. **Site identification, description and determination of material properties(1994/1995).** A site was chosen in east Tennessee that had been subjected to extensive strip mining. Photographic, topographic, and surveying measurements were used to describe the study site. Various soil tests and analysis were used to determine the properties of the mine spoil. An important part in the design process of soil nailing is the determination of soil strength parameters. (Chapter 3)

3. **Design and construction of soil nailed wall (1995/1996).** Four methods were

used to analyze the overall stability of the original slope and the soil nailed slope. Different methods were used to compare the factors of safety and to verify the stability of the design slope. The slope was constructed with 3 different nail schemes, and an un-nailed section. (Chapter 4)

4. **Construction and Instrumentation (1995/1996)**. The soil nailed structure was instrumented and monitored using three different methods: resistance strain gages, inclinometers, and optical surveys. (Chapter 5)

5. **Evaluation and Interpretation of Measurements**. Following construction, the slope was monitored for increases in stresses, and movement in the slope. (Chapter 6)

6. **Conclusions and Recommendations** . Chapter 7 provides some Design Guidelines for soil nailing in mine waste, and draws conclusions from the study.

Chapter 2

Review of Literature Pertinent to Soil Nailing and Mine Spoil

2.1 Introduction

The application of earth inclusions, or soil nailing, is a proven and effective method for the stabilization of soil walls and slopes (Gässler 1988, Elias and Juran 1991, Munfakh et al., 1987, Stocker et al., 1990, Thompson et al., 1990). Soil nails are bars or cables, generally steel, used to reinforce a vertical or sloping face, and are usually driven or grouted into place. The construction sequence is depicted in Figure 2-1 (Clouterre 1991). The nails are not tensioned and although shotcrete is often applied to the face, the face treatment plays a minor role in the overall stability. Construction usually occurs from the top down, with the nails gradually picking up load as excavation progresses and the soils deform. Numerous field studies and load tests have been conducted to determine the effects of surcharge loading, variation of soil properties, application in sandy soils, and unusual terrains on soil nails (Juran 1987, Tsubouchi et al., 1988, Okuzono et al., 1988,

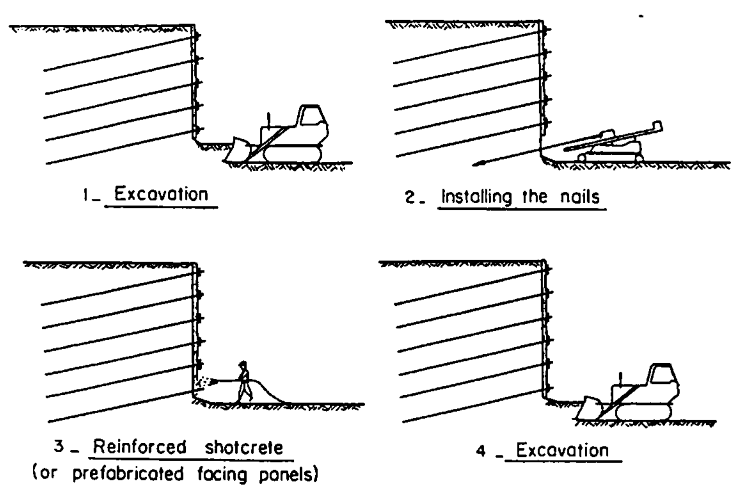

Figure 2-1 Soil Nailing Process (Clouterre, 1991)

Mino et al., 1988, Sano et al., 1988, Kitamura et al., 1988, and Kim et al., 1995). Furthermore, tests have been conducted on inclusions other than typical soil nails such as stone columns, root piles, bamboo dowels, and welded wire (Munfakh et al., 1987, Aurilio 1987, Poorooshasb et al., 1988, and Anderson et al., 1987). Soil nailing has been applied in many different situations such as stabilizing steep excavations adjacent to buildings and roads (Nicholson 1986, MacCara 1993, Wurster & Rhodes 1994, and Powers & Morgan 1995), the reinforcement of ancient retaining walls (Schwing & Gudehus 1988), and the anchoring of landfills (Kroschel et al., 1996). There are many analysis techniques used, and the construction technique is often governed by site conditions, making the technique somewhat site dependent. However, this is a major advantage of soil nailing, since the design (nail inclination, spacing, and length of nail) can easily be changed during construction based on the observed performance (Mitchell and Villet 1987).

There are four major design methods in use today, and several different computer codes have been written based on the design methods. These four methods are the Shen, German, French, and Kinematic Limit Analysis methods. There are other design methods in use or under development such as a numerical technique developed by Abdi et al., (1994) for the critical height of reinforced soil walls, and a general analytical procedure for the design of a soil nailed wall (Sabhahit et al., 1995). Furthermore, advice notes have been published for the application of soil nailing (Love & Milligan 1995). The four major design methods are discussed below.

2.2.1 Shen Method (Davis Method)

The Shen method (Shen et al., 1978, Shen et al., 1981; Shen et al.,) was developed at the University of California at Davis and is widely known as the Davis Method. It is a limit equilibrium design method for soil nailing. The design method assumes all failure surfaces are vertical axis parabolas with the vertices located at the bottom of the facing as shown in Figure 2-2.

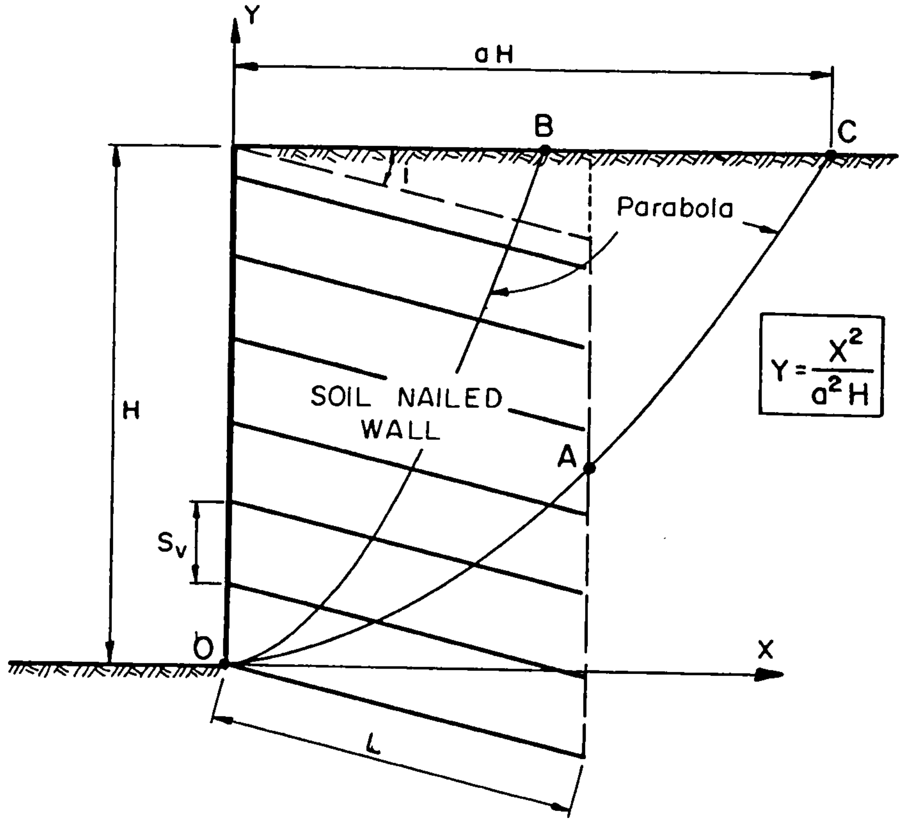

Figure 2-2 Shen Method Design Assumptions (Clouterre 1991)

The method is based on seven basic assumptions for design.

1. Homogenous soil with a Mohr-Coulomb failure criterion
2. The facing is vertical.
3. Horizontal ground surface.

4. Well drained so that pore pressure is eliminated.
5. Vertical and horizontal spacings are equal.
6. Uniform nail length.
7. The nail acts only in tension.

Since only tensile forces are considered, the breakage or force at yield of the nail, T_b, is calculated as follows:

$$T_b = A_s F_y \tag{1}$$

where A_s is the cross sectional area of the reinforcing member and F_y is the yield strength of the member. The frictional force, T_p, of the reinforcing member along the length of the nail is defined as follows:

$$T_p = \pi D L_a \frac{\sigma_n \tan\phi_m + C_m}{S_h} \tag{2}$$

T_p = force in the nail per unit length of wall

D = diameter of grouted reinforcement

L_a= adherence length of reinforcement in the passive zone

ϕ_m = mobilized friction angle ϕ/F_ϕ

C_m = mobilized cohesion C/F_c

σ_n = average normal stress on L_a

S_h = spacing of reinforcement

F_ϕ, F_c = factors of safety with respect to friction angle and cohesion

The minimum of the two tensile values (T_p, T_b) controls the resistance of the nail with the safety factor with respect to the nail pullout being equal to F_l/F_{lm}, where F_l is equal to the lateral shear stresses and F_{lm} equal to the mobilized shear stresses at the soil-nail interface. The resistance of each row is totaled to give an equivalent tensile force for the global stability.

The Shen method considers two failure conditions for the nailed structure. The first is when the failure surface lies entirely inside the reinforced zone. The second is when the failure surface lies partially outside the reinforced zone. The two scenarios as well as the freebody diagrams are shown in Figures 2-3 & 2-4.

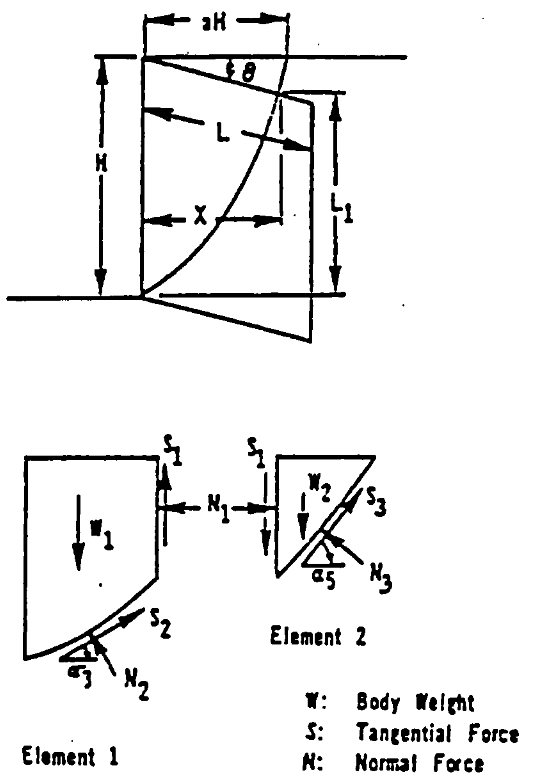

Figure 2-3 Case 1 Shen Method (Elias et al., 1991)

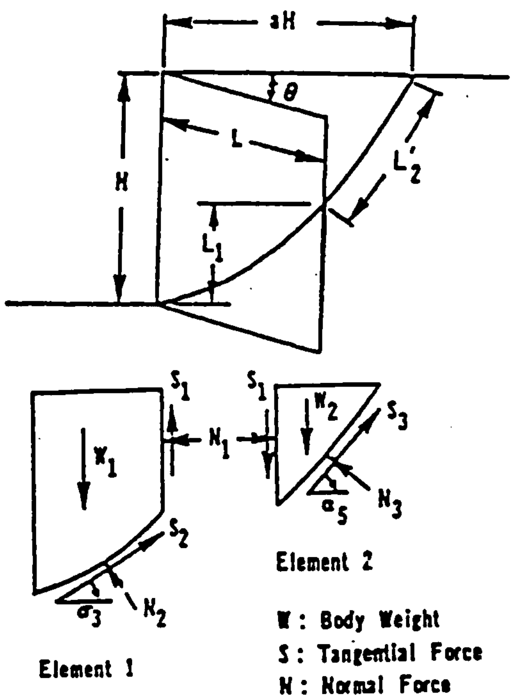

Figure 2-4 Case 2 Shen Method (Elias et al., 1991)

A limit equilibrium analysis is used to analyze the overall stability of the slope using two soil wedges. An active and passive zone are defined for each case. Simplifying the assumptions reduces calculations and provides the overall safety factor. A Mohr - Coulomb failure criterion is used to calculate the lateral shear stress at the interface where σ_v is the average normal stress along the adherence length.

$$\tau = \sigma_v \tan\phi_m + C_m \tag{3}$$

where

C_m = Mobilized Cohesion

ϕ_m = Mobilized soil friction

Shen et al., method utilizes a ratio K of the horizontal and vertical stresses at the interslice to calculate the force equilibrium which is usually taken as 0.4 for frictional soils and 0.5 for cohesive soils. Numerous field and laboratory tests have been conducted and confirm the validity of the design method (Shen et al., 1981).

2.2.2 German Method

In the mid 1970's, Stocker, along with Riedinger, Gudehus, and Gassler, proposed a design procedure for the application of soil nails. The method was based on the performance of laboratory and field test models begun in 1975 (Stocker & Riedinger, 1990) and used a systematic approach to determine the critical failure surface by analyzing four general failure surfaces.

- Rotation of one rigid body (slip circle) (ROT-I), Figure 2-5 a
- Translation of two rigid bodies (TRA-II), Figure 2-5 b,d
- Translation of a rigid body (failure wedge) (TRA-I), Figure 2-5 c
- Rotation of two rigid bodies (ROT-II), not pictured

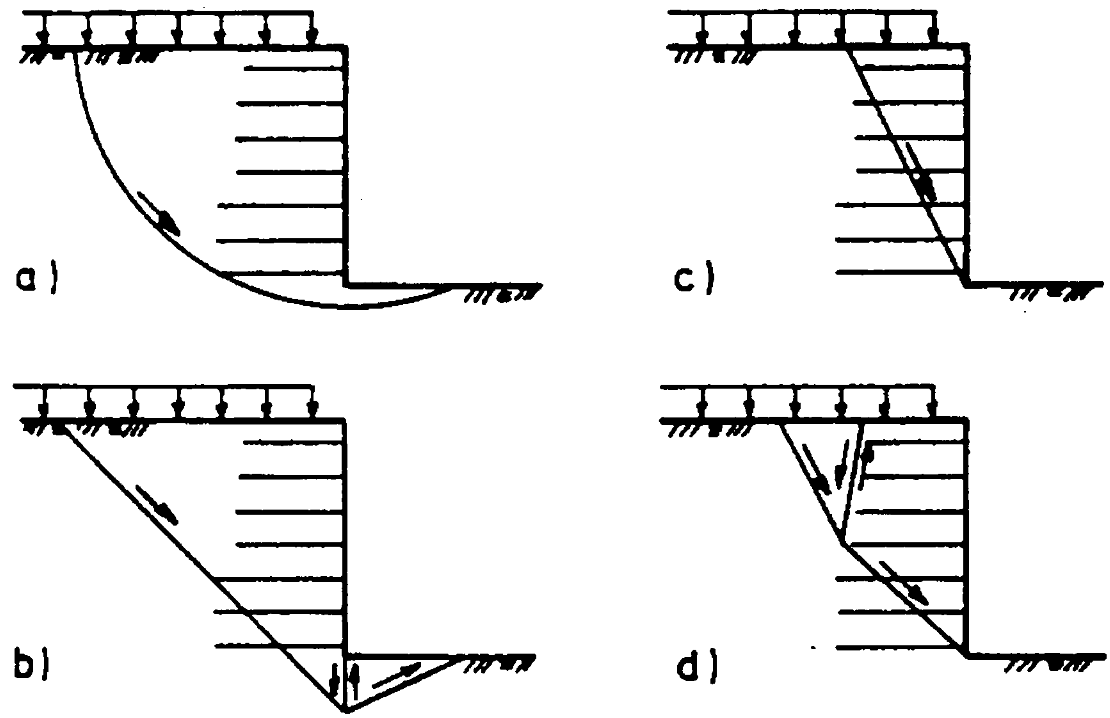

Figure 2-5 Possible failure mechanisms (a) ROT-I (b) TRA-II (c)TRA-I (d) TRA-II (Stocker et al., 1979)

The study included the definition of two failure mechanisms for the failure surfaces which are external and internal failures. An external failure occurs when there are insufficient forces to stabilize the wall and the entire wall fails by sliding, overturning, or overall failure. An internal failure occurs if there is insufficient tensile and shear strength in the reinforcing material. The German design method results in a global safety factor for internal failures. Stocker et al. determined the TRA-II failure surface to reveal the lowest factor of safety. However, this is not consistent with the failure surfaces observed in laboratory and field testing. The test models were loaded using surcharges to induce failure. When conducting hand or computer calculations to find the lowest safety factor the TRA-II surface results in the lowest calculated safety factor. In tests when failures occurred, the observed surface resembled a ROT-I surface.

To calculate the global factor of safety, Figure 2-6, the ratio, η_N of the sum of the resisting limit nail forces Z_a determined from nail pullout tests to the total force required Z_g to maintain limit equilibrium, is determined. It should be noted that the shearing resistance of the soil as defined by Mohr-Coulomb's failure criterion is fully mobilized

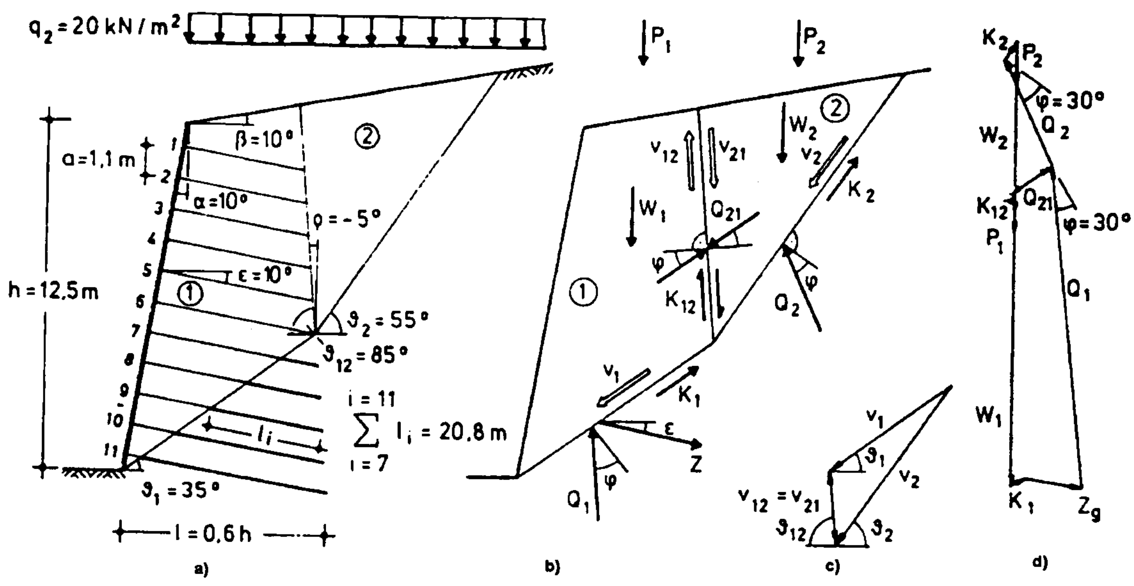

Figure 2-6 Bi-linear failure analysis. (a) failure mechanism (b) acting forces and displacements © hodograph (d)force polygon (Gässler 1988)

$$\eta_N = \frac{Z_a}{Z_g} \quad \textbf{(4)}$$

along the failure surface. The inclination $\theta_{1,}$, of the lower wedge passing though the base of the wall as shown in Figure 2-6 is determined through an iterative process to find the most critical failure surface, and the second line segment θ_2, (Figure 2-6) is taken to be:

$$\theta_2 = \frac{\pi}{4} - \frac{\phi}{2} \quad \textbf{(5)}$$

Gässler (1988) suggests the bilinear failure surface in cohesionless soils results in a more conservative factor of safety compared to deep and steep sliding circles, but with cohesive soils there is little difference between the two surfaces.

To determine the resulting critical failure surface, many iterations of calculations are necessary. A computer code (Stocker & Riedinger 1990) was later developed to eliminate the cumbersome hand calculations used during the process.

Once located, the critical surface can be used to design the nailed wall. The sum of nail forces Z, are determined by using the following equation.

$$Z = \frac{1}{b}\sum_{i=j}^{i=n} N_i = \frac{T_m}{b}\sum_{i=j}^{i=n} l_i \quad \textbf{(6)}$$

b = horizontal distance between nails

$N_i = T_m \bullet l_i$ = internal nail force for nail I

T_m = mean shear force per unit length of nail

l_i = section length of nail I behind slip plane where j denotes the uppermost row, and n the lowest row that is intersected by the slip plane (Gassler, 1988). The sum of the nail forces can also be expressed by using the following:

$$\bar{Z}=\left(\frac{1}{h}\right)^2(\tan\theta_1+\tan\epsilon)\frac{\cos\alpha}{\cos(\alpha-\epsilon)}\frac{T_m}{\gamma ab} \tag{7}$$

with

$$\bar{Z}=\frac{Z}{\gamma h} \tag{8}$$

α = wall inclination

h = height of wall

ϵ = nail inclination

a = horizontal nail spacings

b = vertical nail spacings

γ = specific weight of material

The last part of the equation ($T_m/\gamma ab$) represents the specific nailing density μ.

$$\mu = \frac{T_m}{\gamma \cdot a \cdot b} \tag{9}$$

If the nail density is calculated for both the equilibrium state, μ_g and the design state, μ_a an equivalent factor of safety can be defined in terms of nail density as:

$$\eta_\mu = \frac{\mu_a}{\mu_g} \tag{10}$$

From testing and repeated trial runs it has been shown that there is no difference in the factor of safety determined by the two methods (Gassler, 1988).

A computer method was devised (Gassler, 1988) to develop design charts for TRA-II, and ROT-1 failure surfaces in cohesionless and cohesive soils. From numerous calculations and investigations the following conclusions were made about the characteristic failure surfaces.

a. In cohesionless soils the TRA-II mechanism controls when the nailed wall is nearly vertical (α<10) and constructed of uniform nail lengths. However, when the wall is more inclined (α>10) then the ROT-I mechanism controls, and for ϕ >35° the TRA-I, and TRA-II mechanisms are almost the same.

b. For soils with little cohesion the TRA-II and the ROT-I, II mechanisms are nearly equivalent. With medium to high cohesion the slip circle is the most critical (Gassler, 1988).

Over the last 15 years progress has been made to modify the German method to accommodate partial factors of safety such as those for dead loads, live loads, soil friction, cohesion, and nail friction (Stocker & Riedinger, 1990). A computer code to accommodate these factors has not been developed.

2.2.3 French Method

From 1986 to 1990 under the direction of Francois Schlosser, the French National Project "CLOUTERRE" (Clou- nail, Terre- soil) was developed to investigate soil nailing. From the research the French Design Method was developed, and the TALREN computer code was written to utilize the aforementioned design method. The design method considers three failure criteria: Failure in the soil characterized by a Mohr-Coulomb failure criterion, failure in the nails, and failure of the soil/nail interface. Various failure mechanisms are considered in the design process such as bilinear wedges and circular failures. However, since the method of slices is used to analyze the soil mass in question, the design method is not limited to these surfaces. It is suggested that drained soil parameters be used regardless of the actual soil conditions. However, for temporary walls undrained soil parameters can be used.

The nail inclusions are evaluated based on the nail resistance to simple tension R_n, shear forces R_c, and the bending stiffness M_o of the nail inclusion (Clouterre, 1991). However, the strength of the grouting material is not usually taken into account.

To evaluate the overall stability of the soil nailed wall, four failure criteria are considered together with their interactions with one another.

Soil-nail skin friction criterion (C1)

This criterion relates the strength of the nailed structure to its ability to resist pullout based on the nail's skin friction. The nail tension, T_n is defined as:

$$T_n \leq q_s \cdot \pi \cdot D \cdot L_a \qquad \textbf{(11)}$$

q_s = soil-nail unit friction, or nail skin friction

D=borehole diameter (diameter of grouted nail)

L_a= grouted nail length extending beyond the failure surface.

However, the nail stress produced by the mobilization of the nail skin friction, must be less than the reinforcement's tensile strength as defined by the following nail failure criterion:

$$T_n < R_n$$

where

R_n = tensile strength of nail reinforcement

Therefore, the two conditions are satisfied by selecting the minimum value of the two criterion with applied safety factors.

$$T_n \leq \min\left[\frac{q_s \pi D L_a}{\Gamma_{m,q_s}}\ ,\ \frac{R_n}{\Gamma_{m,\sigma}}\right] \qquad \textbf{(12)}$$

where

$\Gamma_{m,\,qs}$ = partial safety factor for the unit skin friction

$\Gamma_{m,\sigma}$ = partial safety factor for the tensile strength of the reinforcing bar

Soil-nail lateral pressure criterion (C2)

The limiting factor for criterion 2 is ultimate lateral earth pressure of the soil p_u, which is exerted in the nail by the soil (Clouterre, 1991). The criteria can be defined as:

$$Tc \leq T_{c2,max}$$

where

$$T_{c2,\max} = \frac{D_c}{2}\, l_o\, p_u \tag{13}$$

where

$$l_o = \sqrt[4]{\frac{4EI}{k_s D}} \tag{14}$$

D_c= diameter of the nail (grout and reinforcement)

l_o= transfer length

p_u= ultimate lateral pressure

T_c = nail shear force

Criterion (C3) and (C4) Strength of the inclusion

Criterion C3 and C4 are governed by all three nail forces, the tension force T_n, the nail shear force T_c, and the moment M. All three are used with Anthoine's criterion (Clouterre, 1990) which represents the actual resistance in the nail:.

$$\left(\frac{T_n}{R_n}\right)^2 + \left(\frac{T_c}{R_c}\right)^2 + \left|\frac{M}{M_o}\right| - 1 \leq 0 \tag{15}$$

Criterion (C3)

Yield of the nail occurs at the point of maximum shear force O as shown in Figure 2-7, which corresponds to the intersection of the nail and failure surface. The bending

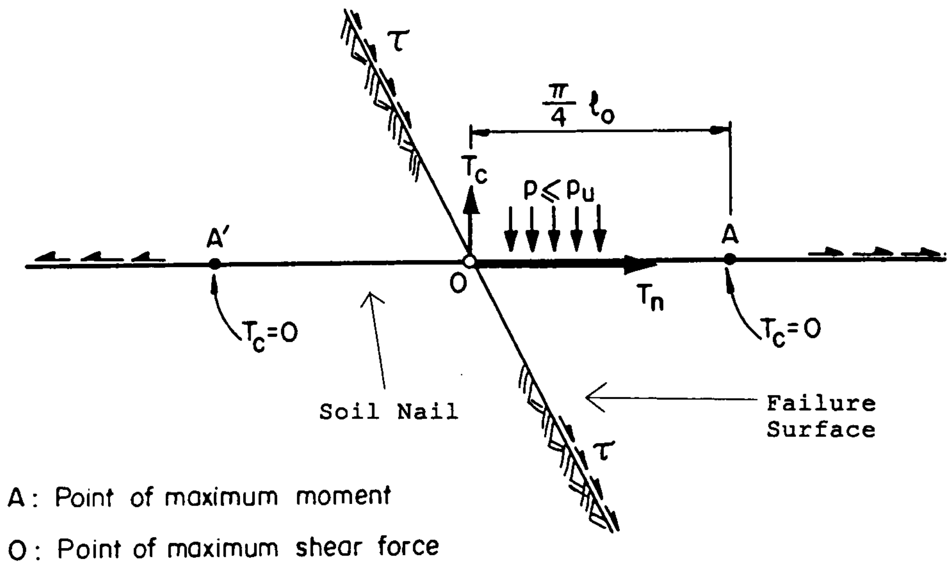

Figure 2-7 Diagram of Nail Forces, Where Point O Designates the Intersection of the Nail and Failure Surface. (Clouterre, 1991)

moment at this point is zero (M=0) and the criteria based on the general failure criteria of the nail are satisfied when:

$$\left(\frac{T_n}{R_n}\right)^2 + \left(\frac{T_c}{R_c}\right)^2 \leq 1 \qquad \textbf{(16)}$$

where

R_n = tension resistance of the nail

R_c = shear resistance of the nail (usually assumed to be one half the nail tensile strength $R_c = R_n/2$)

Criterion (C4)

The fourth failure criterion is based on the assumption that nail yielding due to bending occurs at the two points where maximum bending occurs along A-A' in Figure 2-7, which are located on both sides of the failure surface. Their location is empirically determined to be at an equal distance $l_p = \pi \cdot l_o/4$, with the assumption both the soil and nail are behaving elastically. At the two points of maximum moment the shear force is zero ($T_c = 0$) and the following criterion applies:

$$M \leq M_{max}$$

where

$$M_{max} = M_o\left[1 - \left(\frac{T_n}{R_n}\right)^2\right] \qquad \textbf{(17)}$$

which is calculated by the nail failure criterion and for this value the shear force at Point 0 yields the following equation:

$$T_{c0} = a\frac{M_0}{l_0}\left[1 - \left(\frac{T_n}{R_n}\right)^2\right] \qquad \textbf{(19)}$$

where a = 3.12:

However, from field testing and practice it was observed that initially as the nail yields there are two hinges at A and A' which move with the nail as it is deformed. Initially the value for l_p is equal to $\pi \cdot l_o/4$, but slowly changes during deformation. Therefore, for simplification the value of l_p can be assumed to remain constant, or equal to $\pi \cdot l_o/4$ even though the value varies with deformation. Based on this assumption the nail yielding would be represented by the following criterion :

$$T_c \leq T_{c4,\max}$$

$$T_{c4,\max} = b\,\frac{M_0}{l_0}\left[1-\left(\frac{T_n}{R_n}\right)^2\right] + c\,D_c\,l_0\,p_u \tag{19}$$

where b = 1.62 and c = 0.24

The French analysis method is considered to be comprehensive in the fact that it does consider the effect of soil stratification, seismic loading, and groundwater flow on the global stability of the nailed structure. It has been shown in numerous tests conducted for the CLOUTERRE project the design method accurately predicts the results of full scale models (Elias et al., 1991).

2.2.4 Kinematic method

In 1991, the Kinematic Limit Analysis design method was developed (Elias et al., 1991). This method is also referred to as the FHWA design method because the Kinematic Method was developed for the Federal Highway Administration. This method varies from previous design methods because earlier methods yielded solutions for the global stability of the slope and neglected the local stability of the reinforced slope. It has been suggested that "in soil nailed structures the local stability at the level of a sliding nail can be significantly more critical than the global stability with respect to general sliding in the structure and/or its environment"(Juran et al., 1990). The Kinematic Analysis Method takes into account the main design parameters of the structure's geometry, spacing, bending stiffness of the nails, and inclination and how these affect the shear and tension forces developed in the structure.

The Kinematic Limit Analysis design method is based on a limit-analysis solution which relates a legitimate kinematic failure mode with a legitimate static limit equilibrium solution. The six main design assumptions are as follows and are shown in Figure 2-8 (Elias et al., 1991):

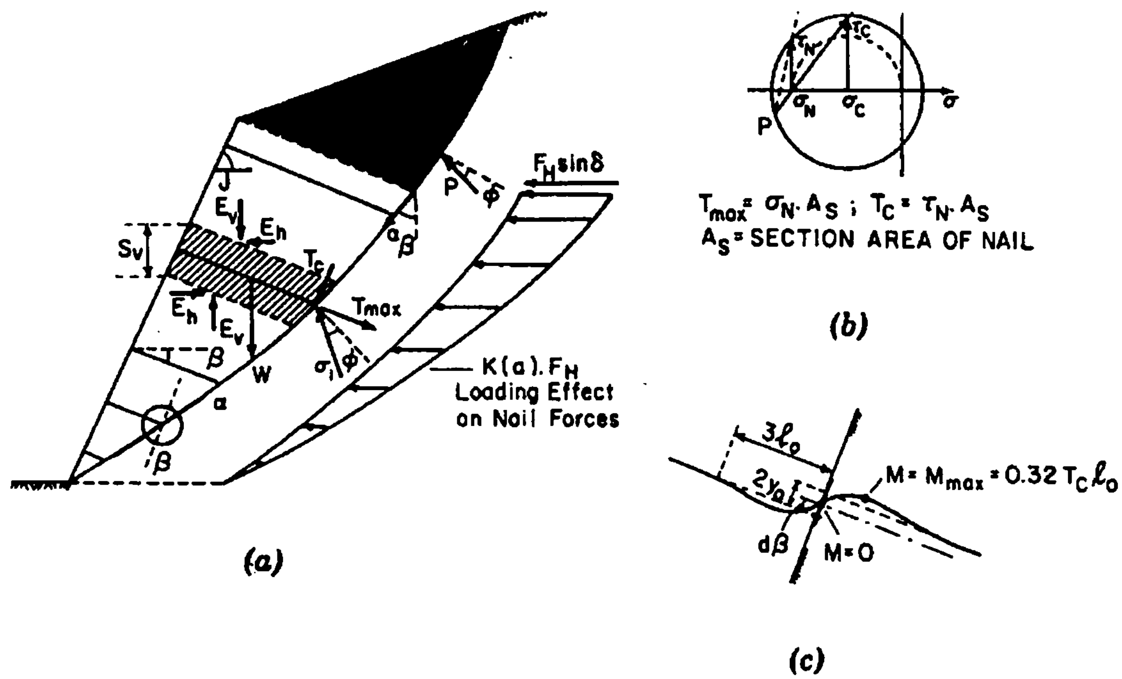

Figure 2-8 Kinematic Limit Analysis Approach: (a) Mechanics of Failure and Design Assumptions; (b) State of Stress In Inclusion; (c) Theoretical Solution for Infinitely long Bar (Elias et al., 1991)

1. Failure occurs by a quasi-rigid body rotation of the active zone that is limited by a log-spiral failure surface.

2. At failure, the locus of maximum tension and shear forces in the nails coincides with the failure surface developed in the soil.

3. The quasi-rigid active and resistant flow zones are separated by a thin layer of soil at a limit state of rigid plastic flow.

4. The shearing resistance of the soil, as defined by Coulomb's failure criterion, is entirely mobilized along the failure surface.
5. The horizontal components (E_h) of the interslice forces acting on both sides of a slice comprising a nail (Figure 2-8) are equal.
6. The effect of a slope above the nailed section (or horizontal surcharge, F_h) at the upper surface of the nailed soil mass on the forces in the inclusions decreases linearly along the failure surface (Juran 1990).

Effects of the nail bending stiffness on nail deformation (excluding the grout) is analyzed using a conventional "p-y" analysis, by treating the nails as laterally loaded, infinitely long piles. "The solution implies that at the failure surface, the bending moment in the nail is zero whereas the tension and the shear forces are maximum" (Juran et al., 1988). The solution incorporates a normalized non-dimensional bending stiffness parameter, N where:

$$N = \frac{K_h D_b L_o^2}{\gamma H S_h S_v} \tag{20}$$

and

L_o = transfer length characterized by relative rigidity of the nail to the soil

$$L_o = \left[\frac{4EI}{K_h D_b}\right]^{\frac{1}{4}} \tag{21}$$

H = height of the reinforced wall

D_b = diameter of the grouted nail

S_h, S_v = horizontal and vertical nail spacing

K_h = lateral modulus of soil reaction

γ = unit weight of soil

The modulus of soil reaction can be approximated by the use of charts found in the FHWA soil nailing manual (Elias et al., 1991) which consider the maximum value of (S/H), TN, and TS . Values for N typically range from 0.1 to 1.5 which suggests the nails behave like a flexible reinforcement. If N = 0, the nail is perfectly flexible & carries only tension. If N < 1, the nail is relatively flexible. If N is above 2, the nail begins to perform more like a rigid nail. When N= 10 the nail performs perfectly rigid.

A kinematical failure surface which satisfies the equilibrium conditions of the active zone is definable since the locus of maximum tension and shear forces coincide with the failure surface. Field observations for flexible nails show the failure surface is nearly vertical at the upper part of the structure. For perfectly rigid nails, it is almost perpendicular to the nail (Elias et al., 1991).

The maximum tension force (T_{max}) for the nail contained in the slice is determined by calculating the horizontal force equilibrium. When mobilized, values for T_{max} and T_c can be represented by normalized non-dimensional values at any relative depth Z/H:

$$TS = \frac{T_c}{\gamma H S_h S_v} \qquad \textbf{(22)}$$

$$TN = \frac{T_{\max}}{\gamma H S_h S_v} \qquad \textbf{(23)}$$

The analysis has been extended to accommodate stratified soils and groundwater. The design method evaluates local stability of each reinforcement by two criteria:

Criterion I. Failure by pullout of the reinforcement

For this mode of failure a nail length can be obtained with the design input parameters governed by:

$$\frac{T_{\max}}{D_g L_a \pi} < \frac{F_1}{F_p} \qquad \textbf{(24)}$$

F_1 = limit interface lateral stress (from pullout test)

F_p = safety factor with respect to pullout

L_a = adherence length

D_g = diameter of grouted nail

With this design criteria the L/H ratio can be verified for each reinforcement level as defined by:

$$\frac{L}{H} \geq \frac{S}{H} + F_p\left(\frac{TN}{\pi\mu}\right) \quad (25)$$

where
$$\mu = \frac{F_1 D_g}{\gamma S_h S_v} \quad (26)$$

S = nail length in the active zone

Criterion II. Failure by breakage of the reinforcement in tension or bending

Case (a) Breakage Failure of the inclusion by tension and shear forces.

Flexible nails withstanding only tension forces must meet the following criterion:

where
$$\frac{F_{all} A_s}{\gamma H S_h S_v} > TN \quad (27)$$

F_{all} = allowable tensile stress

A_s = cross sectional area of the inclusion

The design of rigid nails coupled with tension and shear forces should satisfy:

$$\frac{F_{alt}A_s}{\gamma H S_h S_v} > K_{eq} \tag{28}$$

$$K_{eq} = [TN^2 + 4TS^2]^{\frac{1}{2}} \tag{29}$$

Case (b).For failure by excessive bending the following criterion should be satisfied:

$$M_p > SF_m M_{max} \tag{30}$$

S_{fm} = factor of safety with respect to plastic bending (For allowable stress in tension zone S_{fm}=1, otherwise use S_{fm} = 1.8)

M_p = plastic bending moment of nail

An equivalent plastic bending moment resistance is calculated for a grouted nail assuming grout has a compressive strength f_y = 3,000 psi (21 MPa), and zero tensile strength (Christopher et al., 1989). Juran notes that detailed analysis requires a computer code to determine the optimum design parameters. However, in the FHWA manual "Soil Nailing For Stabilization of Highway Slopes and Excavations" (Elias et al., 1991), preliminary design charts for perfectly flexible nails and for No. 8 rebars are provided. The steps outlined for conducting a kinematic analysis (Juran et al., 1990) follow:

Step 1. Select the nail type (bending stiffness, EI, allowable tension stress, F_{all}, diameter, D, and spacings, S_v, S_h).

Step 2. For the specified soil properties (γ, K_s, c, ϕ), selected nail type (EI, F_{all}), nail inclination (β) and the structure height (H), determine the ratios S/H, TN, and TS, from the design charts.

Step 3. Verify that the selected reinforcement satisfies the breakage and excessive bending failure criteria.

Step 4. Select the limit interface lateral shear stress f_l from the field pull-out data or from available correlations with in situ tests for preliminary design.

Step 5. Establish structure geometry (L/H) to satisfy the pull-out failure criteria with the required value of the safety factor, F_p.

2.3 Comparison of Slope Analysis Methods

Each of the four methods discussed in Section 2.2 were examined and the results were compared for a simple wall structure as described in Table 2-1. Four computer programs were utilized to analyze the simple structures. The programs analyzed a simple geometry since some methods are limited to certain input parameters. Table 2-2 lists the design methods and the associated computer programs. Other computer codes that were not compared include Goldnail from Golder & Associates, and a design optimization code (Abdi et al. 1994).

Comparison of the different methods reveal variations in the overall factors of safety. These differences can be attributed to the design approach itself as discussed previously. Each method has its own distinctive calculation of a safety factor, and many of the methods have been developed and improved upon older methods. Differences between the hand calculations and the computer codes could be attributed to how the code is designed, and how factors are selected. Most hand calculations are conducted using design charts making the outcome somewhat subjective. Input files and hand calculations are shown in Appendix 5.

Table 2-1 Properties for Stability Analysis

Slope Characteristic		Value
Wall Height, h		6.1 m (20 ft)
Wall Inclination (From Vertical)		0°
Nail Length (3 rows)		4.6 m (15 ft)
Nail Inclination (From Horizontal)		10°
Unit Weight, γ		16.6 kN/m^3 (106 lb/ft^3)
Friction Angle, ϕ		30°
Nail Spacing	Horizontal	Sh = 1.5 m (5 ft)
	Vertical	Sv = 1.5 m (5 ft)
Cohesion, c		7.2 kPa (150 lb/ft^2)

Table 2-2 Summary of Factors of Safety from Stability Analysis

Method	Implementation	FS
German (Stocker and Riedinger 1990)	SNAIL (Caltrans 1992)	1.24
	EDINA (Law Environmental Inc. 1991)	2.03
	Hand Calculation	2.2
French (Schlosser et al. 1990)	Hand (design chart)	1.54
Kinematic (Juran et al. 1990)	Mathcad Template	1.34
	Hand (design chart)	1.34
Shen "Davis" (Shen et al. 1981)	NAIL (Geoconsult Inc. 1991)	1.11

Chapter 3

Site Description

3.1 Site Selection

The study site is in Anderson County, Tennessee, in the Windrock area north of Oliver Springs near the intersection of Anderson, Scott, and Morgan Counties. The site was selected after investigation of numerous sites in eastern Tennessee and southeastern Kentucky. The site is located on Buffalo Mountain as designated by the Windrock topographic map. Figure 3-1 shows the location of Buffalo Mountain Site. The land

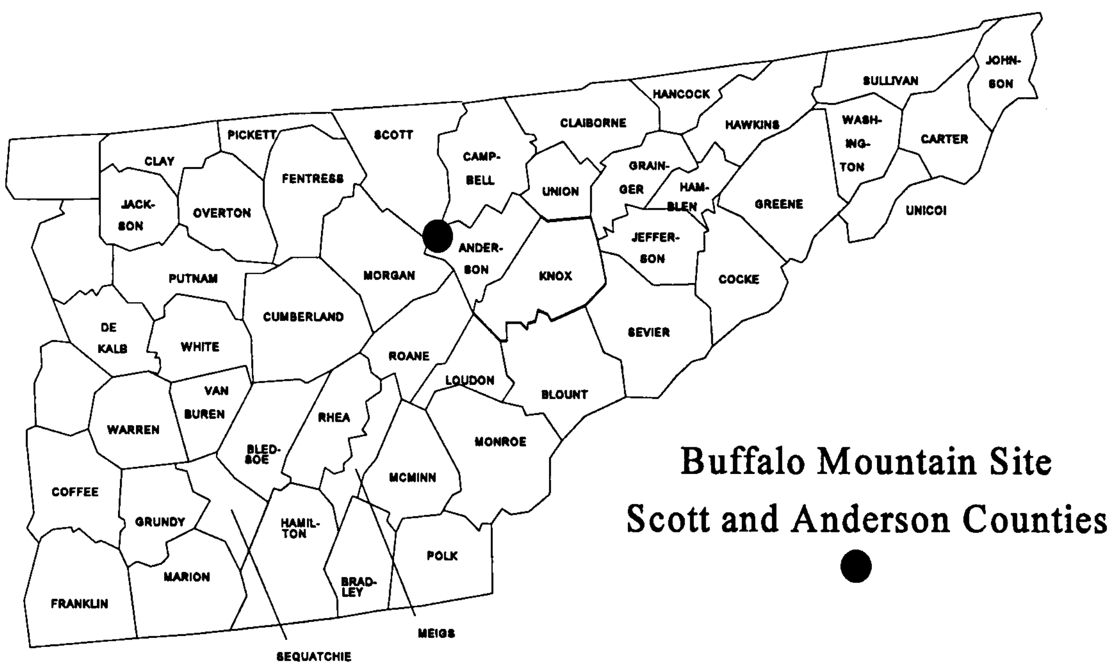

Figure 3 -1 Location of Study Site In East Tennessee

surrounding the site is a combination of reclaimed surface mine sites, partially reclaimed bond forfeiture sites, and pre-reclamation law sites. The site identified for the research project is a bond forfeiture site that has experienced one or more slope failures since the mining operations were completed.

3.2 Site Description & Geology

The site on Buffalo Mountain is comprised of loose uncompacted fill material which varies in size from silt size particles to large boulders and includes numerous tree trunks and root balls. The site is located near 914 m (3000 ft) above sea level. The appearance of the area changes dramatically from season to season. The site remains wet throughout the year, and from October to April, maintains a semi-frozen state. Figure 3-2 (Walker, 1995) shows the failure slope and remnants of the original slope, which has come to rest at the bottom of the scarp.

Figure 3 -2 Aerial Photograph of Buffalo Mountain Site

Numerous contour surface mine operations were conducted in the region. In an investigation of mine spoil stability in the eastern U.S. coal province, Swanson et al. (1983) included two nearby sites that are geologically similar to the Buffalo Mountain site. The sites (in Scott and Anderson counties) Figure 3-1, are in the Wartburg basin which is "bounded on the southeast by steeply dipping beds which form the boundary between the Cumberland Plateau and the Valley & Ridge Province"(Swanson et al., 1983). Buffalo Mountain, located in the Wartburg basin, is underlain by coal bearing rocks of Pennsylvanian age (Middle Pennsylvanian) comprised of shale, coal, sandstone, and clay layers which are essentially horizontal. The two sites described by Swanson et al. (1983) and the Buffalo Mountain site are located in the upper Vowell formation and lower portion of the Vowell formation. The coal removed from the study site was part of the Upper and Lower Grassy Spring Coal seam. The two seams are vertically about 12.2 m (40 ft) apart. The upper seam varies in thickness from 0-39 inches, and the lower seam varies greatly. The lower seam is usually found 9.1 to 15.2 m (30 to 50 ft) above the Frozen Head Sandstone which is a member of the Vowell Mountain Formation (Statler and Ronald, 1970). Cross sections of the two sites described by Swanson et al. (1983) are shown in Figures 3-3 and 3-4. A cross section of the Buffalo Mountain Site (Coal Creek Mining & Manufacturing Co., 1985) is shown in Figure 3-5. The upper and lower Grassy Spring Coal seam is part of the Cross Mountain Formation (Statler, 1970) where the slope

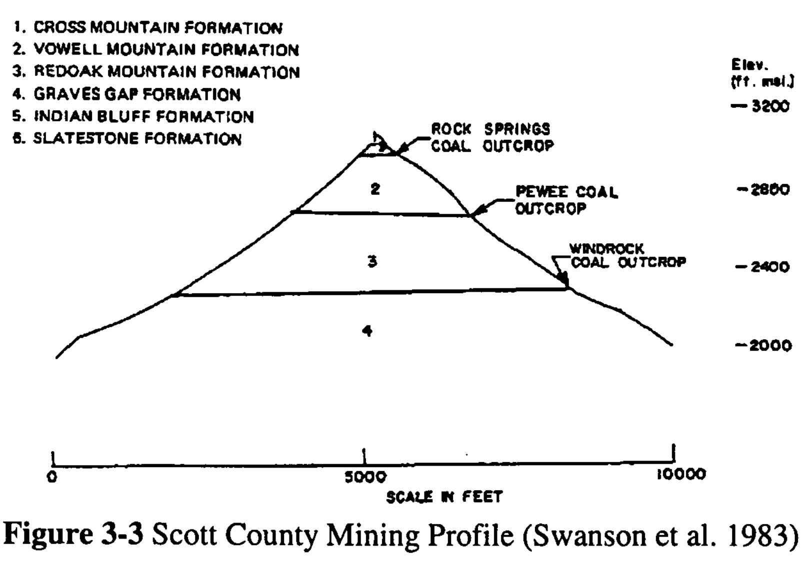

Figure 3-3 Scott County Mining Profile (Swanson et al. 1983)

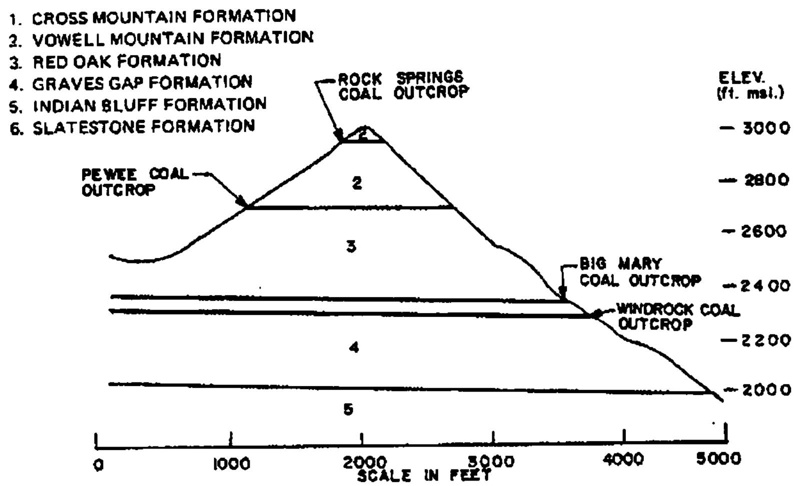

Figure 3-4 Anderson County Mining Profile (Swanson et al. 1983)

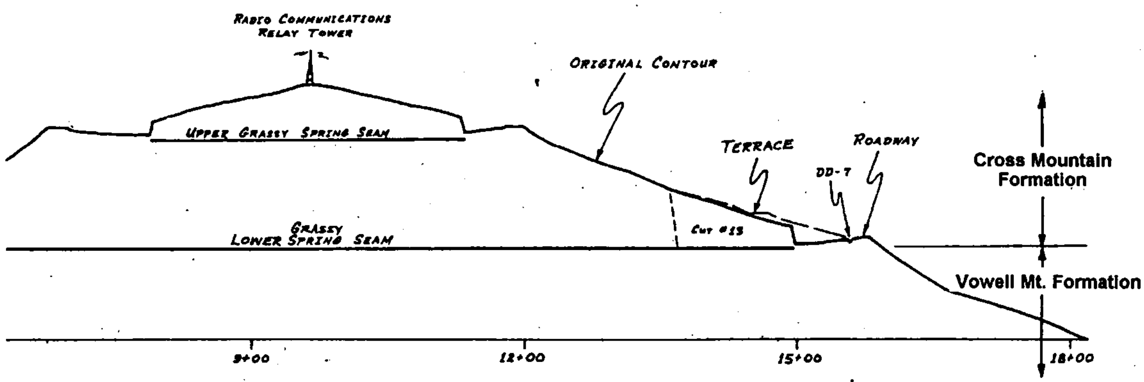

Figure 3-5 Geologic Profile of Buffalo Mountain with Strip Mine Highwalls (Coal Creek Mining & Manufacturing Co. 1985)

is spoil from the mining of the Grassy Spring seams. It can be seen that the same geologic layers were mined on the study sites as was mined on Buffalo Mountain. Swanson et al. (1983) included published material properties such as grain size distributions, shear strength, and compaction properties for the mine spoil.

As of 1970, the Tennessee Department of Conservation, Division of Geology reported Buffalo Mountain to be one of the only areas where the two seams had not been mined. This suggests the mining operation was conducted between 1970 and 1975, since the 1975 topographic map shows extensive mining operations. It is estimated that over 65 percent of the exposed Pennsylvanian strata are shale and clay (Statler, 1970), which is evidenced by the abundance of these materials on the site. The lithology or geologic structure of the site is shown in Figure 3-6, from Duncan Flats and Fork Mountain, with the Grassy Spring coal seams near the bottom of the Cross Mountain formation.

In a soil survey published in 1978 three soil types were identified on Buffalo Mountain: the Jefferson Soils (JSE), the Muskingum - Petros complex (MrE), and the Udorthents (UDE)(Moneymaker, 1978). The two soils on the site identified during construction are the MrE and UDE soils. The soil study, conducted following the completion of mining operations, concluded the UDE soil is the spoil material from the surface mining. The MrE soil is the residual soil above the mining benches and areas not mined. Moneymaker (1978) describes the UDE soils as consisting of the high walls formed and the heterogenous spoil excavated, during the mining. The color and texture of the spoil varies, with most of the material a yellowish brown shaly silty clay loam. The material ranges in color from gray to yellow, to numerous shades of brown. Moneymaker (1978) points out that the soil contains 35 - 80 percent rock fragments comprised mostly of shale with some sandstone and coal. The MrE soils are characterized as a dark grayish brown and light yellowish brown surface layer about 13 cm (5 inches) thick with a moderate erosion hazard. The subsoil is yellowish brown and about 66.0 cm (26 in) thick. Interlayered shale and sandstone are often included in the MrE soils. These materials were observed on the Buffalo Mountain site but in depths greater than 3.0 m (10 ft).

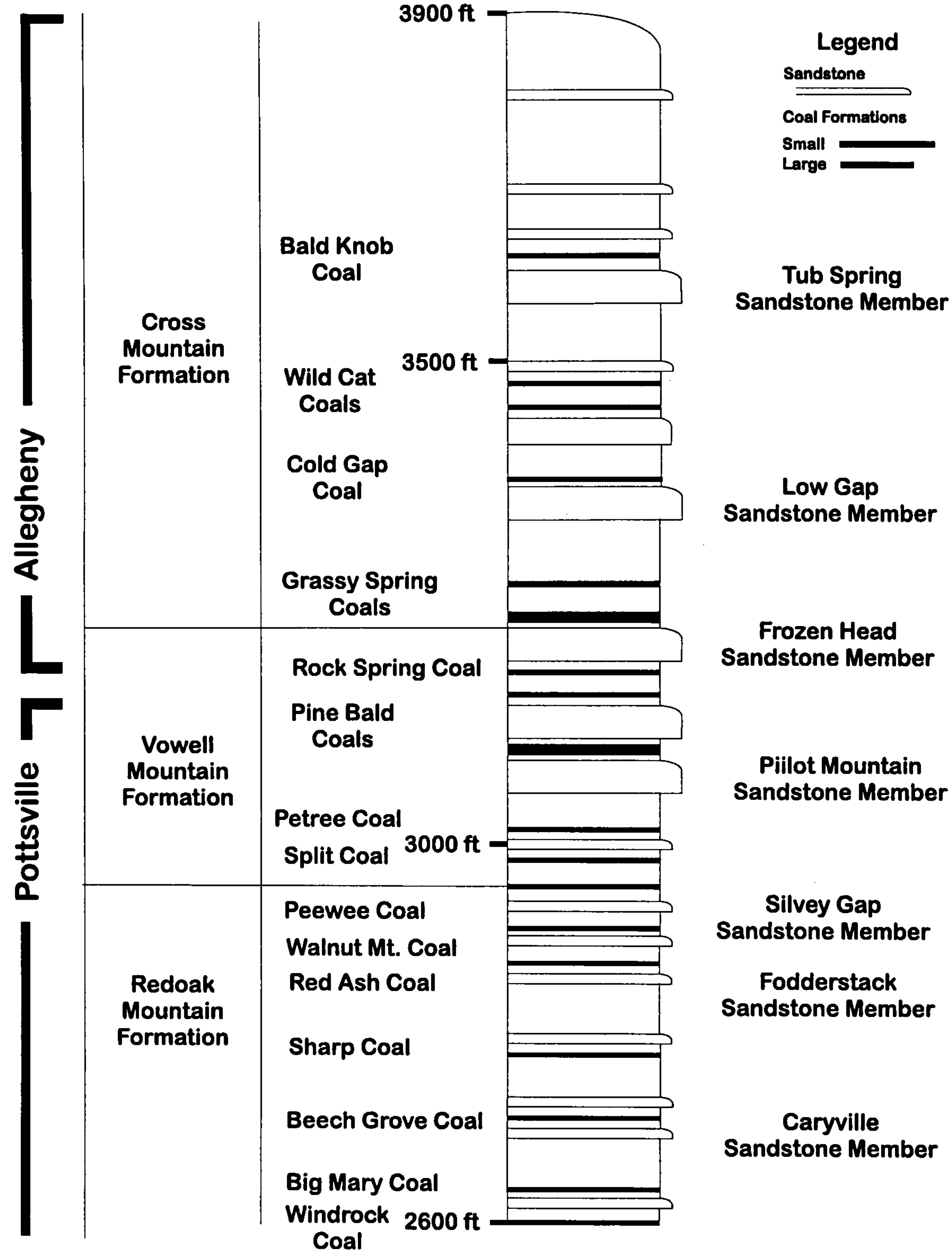

Figure 3-6 Lithology of Buffalo Mountain (Tisdale 1974)

3.3 Topographic maps from pre-mining and post mining operations

An investigation of the original site terrain was conducted to determine the effects of the mining on the site topography. To investigate these effects, topographic maps from 1952 (before mining) and 1975 (after mining) were obtained. Using the coordinates supplied by the mine operators, and coordinates obtained from a G.P.S. survey, the location of the road passing over the slope was found on the 1975 topographic map. Then the same point was located on the 1952 topographic map. Portions of the two topographic maps are shown in Figure 3-7 and Figure 3-8. Various line segments (such as that designated A-A) were drawn up and down slope of the road on both topographic maps. From the elevations and distances between contours along these lines, profiles of the pre-

Figure 3-7 1952 Topographic Map of Study Site

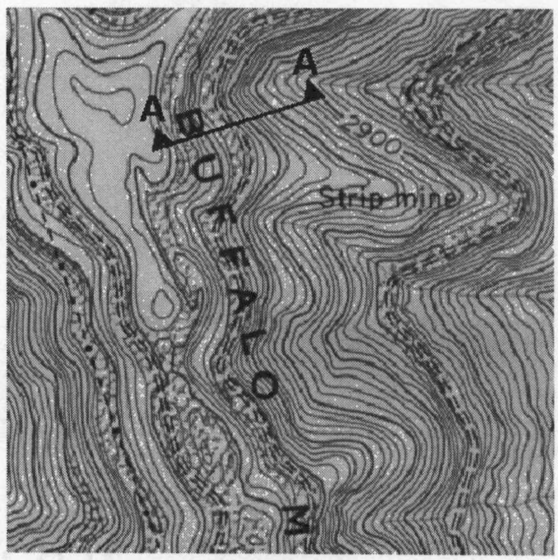

Figure 3-8 1975 Topographic Map of Study Site

mining and post mining slopes were created. Assuming that the difference in the two profiles was due to the contour mining, mine spoil with depths varying from about 3 m

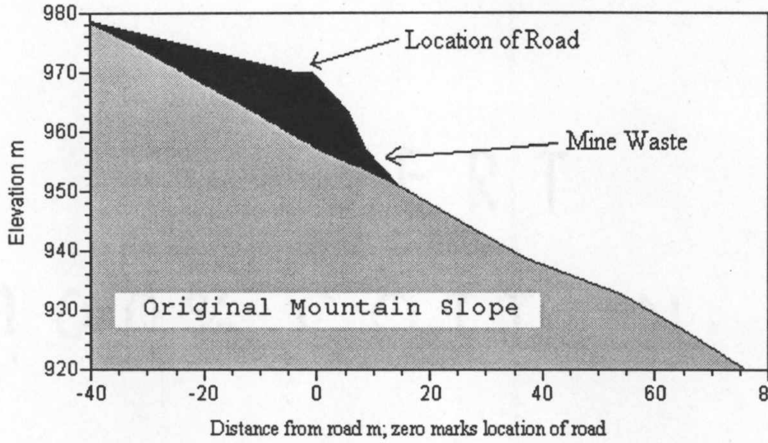

Figure 3-9 Profile of Buffalo Mountain after Mining (1975) as Estimated from Topographic Maps.

important for the erosion (10 ft) to 12 m (40 ft) deep was placed in the project area as depicted in Figure 3-9.

3.4 Geologic Structure & Cross Section of Profile

Throughout the project, several borings were conducted in various parts of the slope: four in the road at the top of the slope, and four in the slope. The first two borings (1994) in the road were terminated in the UDE soil (mine spoil) around a depth of 7.6 m (25 ft). These did not encounter residual soil or the mountain face. However, two later borings (1995) in the road were extended beyond that depth and four were drilled for the installation of the slope inclinometer. In all of the later borings, sandstone bedrock was encountered along the slope at about 8.2 m (26.8 ft) in depth. Based on rock outcrops in the area, the bedding planes of the sandstone strikes about N50W and dips 10 NE degrees.

The site preparation included the removal of a small vertical face at the top of the slope approximately 2.5 m (8 ft) high, to make the slope more nearly 1 to 1. The vertical part can be seen in Figure 3-10. Several buried trees and large boulders protruding from the dumped mine waste are also observed.

Figure 3-10 Slope Prior to Removal of Vertical Section

From the auger borings and notes recorded by the drilling operator during the installation, a more detailed estimate of the subsurface profile was created as shown in Figure 3-11. Based on the initial 1994 borings and the review of the topographic maps, a subsurface profile similar to that in Figure 3-9 was believed to exist. This relatively homogenous profile was used for the initial design of the nailed wall. As the excavation and nail installation took place, and data from the inclinometer borings was obtained, additional subsurface profiles were prepared. Cross sections for the other test sections are available in Appendix 2. From the cross sections it is evident that the mine waste material was placed randomly, and the zone of the silty clay material is significant. The encountered layers fit the descriptions provided by Moneymaker (1978) with the silty clay similar to MrE and the mine waste similar to the UDE. No immediate ground water was

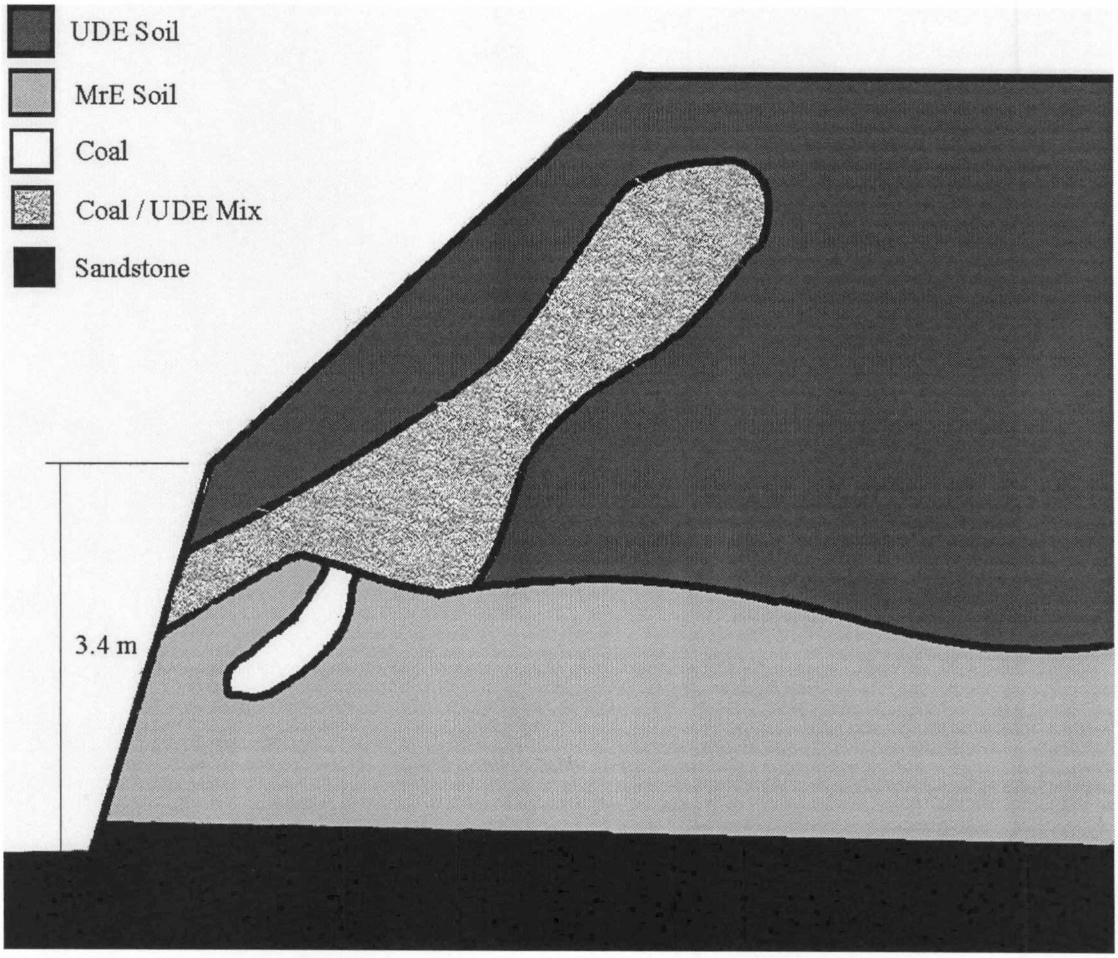

Figure 3-11 Schematic of Likely Subsurface Profile

encountered in any of the vertical boreholes. However, the inclinometer boreholes were left open for 24 hours, and a water table was detected in the boreholes but the depth was not able to be measured. It is noted from the schematics of the subsurface profile in Figure 3-11, that the thick layer of the MrE soil in the cut slope will have a significant impact on the stability of the nailed slope. This material was not identified in the initial investigation, nor considered in the initial design, but the soil nailing approach permits adjustments as unexpected subsurface conditions are encountered.

3.5 Insitu and Laboratory Testing of Site Materials

3.5.1 Nail Pullout Tests

Two vertical auger borings (February 1995) were drilled in the road crossing the crest of the slope. From the borings, bulk samples of the UDE spoil materials were obtained, and in March 1995 two soil nail pullout tests were conducted by Rembco Engineering to measure the maximum shear stress between the grout and the mine spoil. Both borings were terminated at a predetermined depth of 7.62 m (25 ft), which was within the mine spoil. Another pullout test was conducted by Rembco Engineering in July of 1996, to measure the ultimate skin friction for nails placed in the clayey MrE type soils. Test nails were installed in the bore holes, and after the grout cured the nails were loaded to failure. From the pullout tests, an ultimate friction value was obtained which is an important input parameter in the design of the nailing system. In all tests the grouted portion of the nail was 200 mm (8 in) diameter and 2.4 m (8 ft) in length. Table 3-1 lists the average results of the pullout tests.

Table 3-1 Nail Pullout Test Results

Results of Pullout Tests Conducted 3/13/95 UDE Material			
Applied Load at Failure	Surface Area of Soil Nail	Displacement at Failure	Ultimate Stress in Nail
52.6 kN (11.8 kips)	1.56 m^2 (16.8 ft_2)	0.15 mm (0.006 in)	33.7 kN/m^2 (704 lb/ft^2)
Results of Pullout Tests Conducted 7/12/96 MrE Material			
21.2 kN (4.76 kips)	1.56 m^2 (16.8 ft^2)	0.33 mm (.013 in)	13.6 kN/m^2 (283 lb/ft^2)

3.5.2 Laboratory Index Testing

Cuttings of the UDE material were collected from the boreholes and washed through a #200 sieve. A sieve analysis indicated at least 95% finer than #200. During the #200 wash, it was noticed that some of the spoil material was breaking down which suggests the material is subject to slaking. A comparison of the lab tests and published sieve analyses are available in Appendix 3. Atterberg limits were determined on a sample of the UDE mine waste from the southwest portion of the slope. The results are shown in Table 3-2. These results suggest that the mine spoil has a relatively narrow plasticity index, and is therefore unlikely to undergo significant volume change due to water content variation. The material would be classified as a low plasticity silt (ML) by the USCS classification system (Howard, 1984). The low PI also suggests that the nailed slope will not be subject to significant creep deformation and is well below the maximum value of 20 recommended for permanent soil nailed walls (Elias and Juran, 1991). Atterberg Limits were also run on the clayey MrE samples. The results are listed below in Table 3-2. The soil is a low plasticity silt (ML) based on the USCS classification system (Howard, 1984). For comparison, the published Atterberg Limits (Moneymaker 1978) are included in Table 3-2. Moneymaker (1978) did not report material properties for the UDE soils. There are some differences between the laboratory data and the published and could be explained by differences in technique used conducting the Atterberg Limits analysis.

Consolidated Undrained (CU) tests were performed (Yang 1996) on undisturbed tube samples taken in the MrE soils. Table 3-3 lists the results of the lab tests and the soil strength parameters.

Table 3-2 Published and Laboratory Atterberg Limits of Windrock Spoil

Sample Origin	LL Liquid Limit	PL Plastic Limit	PI Plasticity Index	USCS
MrE Soil (Moneymaker, 1978)	20-35	22-45	2-10	N/A
MrE Soil	44	28	16	ML
UDE Soil	31	26	5	ML

Table 3-3 (UC) Tests Results for Undisturbed MrE Samples

Soil Type	Cohesion, c	Unit Weight, γ	Friction Angle, ϕ
MrE	18.0 kPa (376 psf)	16.6 kN/m^3 (106 pcf)	28°

3.6 In-Place Testing

Due to the granular nature and the wide range of particle sizes of the UDE waste material, undisturbed tube samples for strength testing could not be obtained. Therefore, two in-place borehole shear tests (Lutenegger, 1987) were conducted. The results can be found in Appendix 3. The tests were conducted with various consolidation times to determine effective strength parameters. The two tests yielded nearly identical results, with a friction angle (ϕ) of about 30°. This was consistent with triaxial data reported by Swanson et al. (1983) on remolded samples of mine spoil from nearby mining operations.

3.7 Back Prediction of UDE Cohesion From Slope Stability Analysis

Inspection of the slope suggested that the UDE material possessed a small amount of cohesion, and it would not be measured in disturbed samples from lab and field tests.

To estimate the cohesion of the UDE soil, a profile of the slope prior to the failure (Figure 3-12) was estimated from the 1975 topographic map and evaluated using the STABL5 (USDOT, 1985; Carpenter, 1986) slope stability analysis program. The failure surface was assumed to be contained within the UDE material and STABL5 was used to back-calculate a value for the cohesion corresponding to a factor of safety, FS=1. The cohesion value was determined to be 7.9 kPa (165 psf).

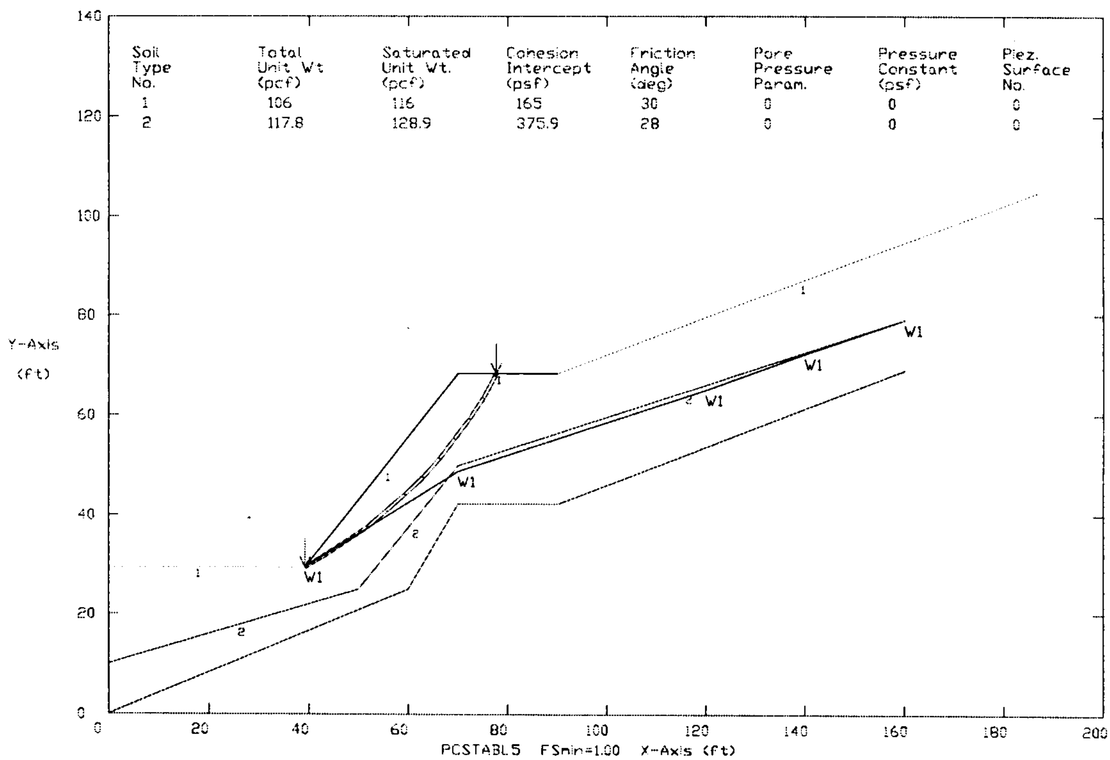

Figure 3-12 STABL Analysis of 1975 Slope Prior to Failure

Chapter 4

Stability Analysis and Soil Nail Slope Design

4.1 Slope Stability of 1995 Slope Profile

With the measured soil properties and an estimate of the cohesion it was possible to analyze the existing slope on Buffalo Mountain. A slope profile obtained from an optical survey (June 1995) was used in the analysis. The water table was assumed to flow along the original topography of the mountain beneath the spoil and through the toe of the slope. This assumption was based on the absence of water at a depth of 7.6 m (25 ft) in the auger holes in the road and the observation of water at 0.3 m (1 ft) below the slope surface in a hand augered borehole at the toe of the slope. The profile was identified as shown in Figure 3-11. From a STABL5 analysis, the factor of safety was found to be about 1.36. A factor of safety greater than one was not surprising since the slope had failed previously as indicated by the soil mass and trees found at the bottom of the slope, Figure 3-2. The slope had failed and naturally come to rest in a marginally stable condition. Figure 4-1 depicts the existing slope, and critical failure surface.

4.2 Analysis of Cut Slope

Since the 1995 slope profile was judged to be stable, it was decided to modify the slope to reduce the stability. By creating a steep cut in the toe of the slope, the stability could be reduced. Based on the assumed material properties, the highest unreinforced cut in the toe that could be made without instability is around 3.7 m (12 ft). Thus, it was determined that the lower 4.0 m (13 ft) of the slope would be cut from the existing 1H:1V

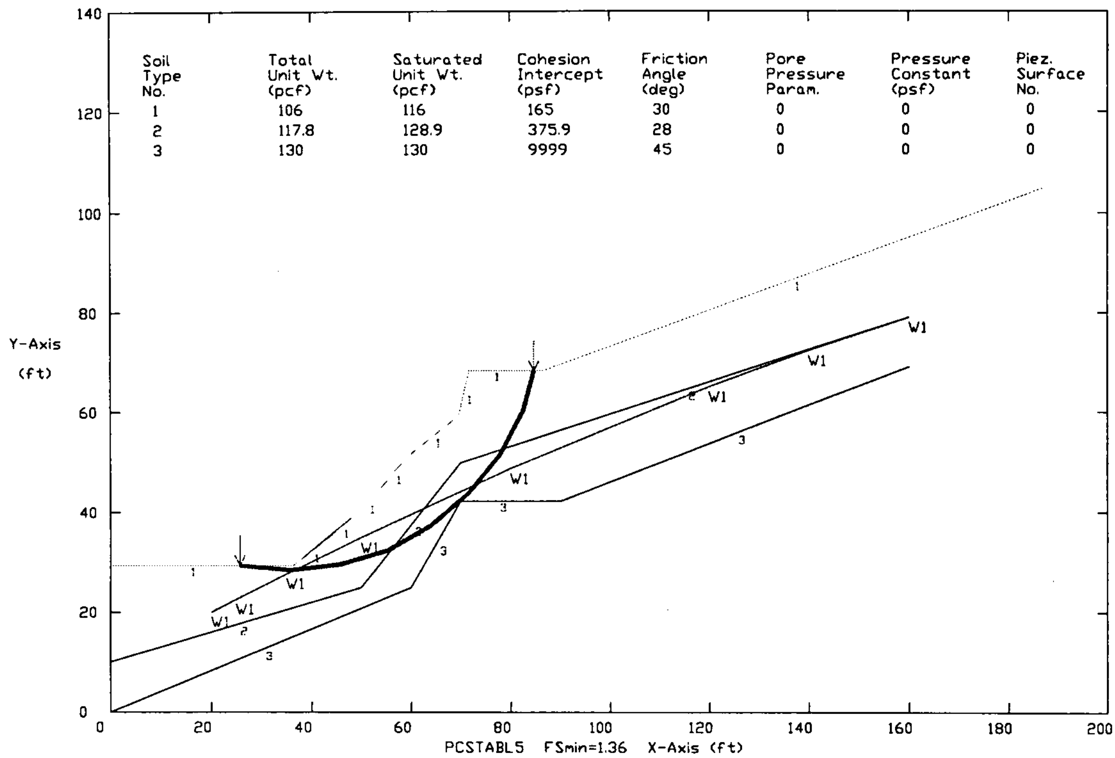

Figure 4-1 Stability Analysis of 1995 slope

slope to about 1H:3.7V (75 degrees).The resulting slope profile consisted of an upper portion 4.0 m (13 ft) high at the natural slope inclination, and a lower portion about 4.0 m (13 ft) high. Figure 4-2 shows the critical failure surface obtained from the STABL analysis with the subsurface profile observed during excavation. The total height of the study slope was limited to 8.0 m (26 ft) because as the slope was excavated, the face was moved back towards the sandstone layer until the toe of the slope rested on the sandstone. This also exposed the MrE soil at the face resulting in a soil profile that is slightly different than that in Figure 4-1. The STABL analysis of the cut slope yielded a factor of safety of 1.16.

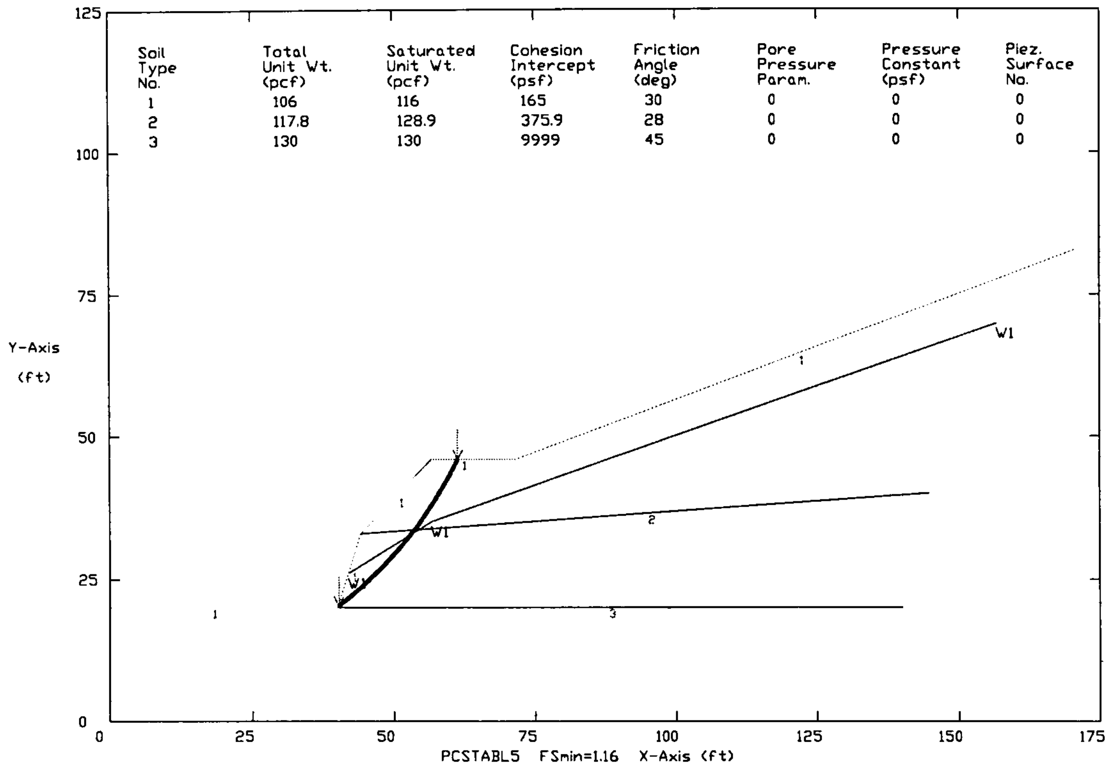

Figure 4-2 Profile of Cut Slope Used in Soil Nailing Design

4.3 Slope Nail Design

The slope was divided into four sections: three sections with different nailing schemes and design factors of safety, and a fourth unreinforced control section. The fourth section was to be steepened to match the slope in Sections 1 through 3 following completion of the nailing in Sections 1,2, and 3.

The design factors of safety for the sections decreased from Section 1 to Section 3. The 8 m (26 ft) high design slope was composed of a reinforced upper slope, and a reinforced lower slope. The 4 m (13 ft) high upper natural slope was the same for all three sections with three rows of nails 3 m (10 ft) long, spaced 1.5 m (5 ft) vertically and spaced horizontally to match the spacings in the lower cut slope. The lower slope was

4 m (13 ft) high with the three nailing schemes as shown in Table 4-1. The first row of nails in each section was 0.3 m (1 ft) below the upper slope. An elevation view of the idealized slope is provided in Figure 4-3 and the material properties used for design are shown in Table 4-2. The study sections were analyzed using five different methods:

Table 4-1 Nailing Scheme for Design Cut Slope

Design Nail Spacings	Section 1		Section 2		Section 3	
	Sh= 1.5 m (5 ft)	Sv= 1.5 m (5 ft)	Sh= 1.8 m (6 ft)	Sv= 1.8 m (5 ft)	Sh= 1.8 m (6 ft)	Sv= 1.8 m (6 ft)
Nail Length, Row 1	5.8 m (19 ft)		4.0 m (13 ft)		4.0 m (13 ft)	
Nail Length, Row 2	5.8 m (19 ft)		4.0 m (13 ft)		2.7 m (9 ft)	
Nail Length, Row 3	5.8 m (19 ft)		4.0 m (13 ft)		2.7 m (9 ft)	

Table 4-2 Summary of Material Properties used for Analysis of Nailed Slope

Property	Dry Unit Weight γ_d	Saturated Unit Weight γ_s	c	ϕ
MrE	18.5 kN/m^3 (117.8 lb/ft^3)	20.2 kN/m^3 (128.9 lb/ft^3	18.0 kPa (375.9 lb/ft^2)	28°
UDE	16.6 kN/m^3 (106 lb/ft^3)	18.2 kN/m^3 (116 lb/ft^3)	8.6 kPa (180 lb/ft^2)	30°

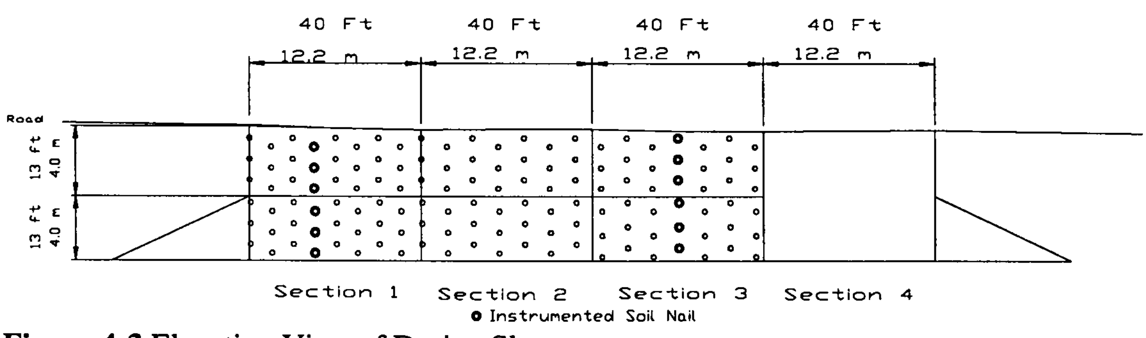

Figure 4-3 Elevation View of Design Slope

SNAIL (Caltrans, 1992), EDINA, (Law Environmental Inc., 1991), NAIL (Geoconsult Inc., 1991), the Kinematic method (Juran et al., 1991) and STABL5 (USDOT, 1985; Carpenter, 1986). The factors of safety for the design slope are listed in Table 4-3.

Table 4-3 Summary of Design Factors of Safety from Stability Analysis

Analysis Method	Program	Section 1	Section 2	Section 3	Section 4 Unreinforced
Stocker and Riedinger (1990)	SNAIL	1.57	1.43	1.39	1.24
Gässler and Gudehus (1981)	EDINA	2.39	1.03	0.83	N/A
Juran et al. (1990)	FHWA Method	1.8	1.25	0.5	N/A
Shen et al. (1981)	NAIL	1.64	1.06	0.91	N/A
Bishop (1955)	STABL5	N/A	N/A	N/A	1.16

Due to the variation of design assumptions in each of the analysis methods, there were some differences in the manner in which the slope was idealized and the manner in which the resulting factors of safety were calculated. SNAIL, EDINA, FHWA, and STABL5 were all able to accommodate the natural slope above the cut slope, but neglected the nails in the upper slope. For the NAIL program, only the cut portion of the slope was analyzed, resulting in slightly elevated factors of safety. In the FHWA, analysis only uniform nail lengths could be analyzed, therefore each section was analyzed assuming a uniform nail length equal to the longest nails included in Table 4-1. The results in Table 4-3 reflect the decrease in the factor of safety from Section 1 to Section 4. The differences between the factors of safety for the analysis methods can be attributed to the various design assumptions for the methods and the manner in which the method is implemented in the computer code.

The above analyses were conducted with an assumed water table based on observations in the field following completion of construction. Water has been observed in Section 1, and the lower three feet of the cut portion remains damp in Section 1. No ground water has been detected in Section 3 or Section 4 but the assumed water table was used to analyze these sections.

4.4 As-built Slope Analysis

Following construction of the slope a typical section was surveyed and the factors of safety re-evaluated to reflect the as-built condition. The survey revealed an as-built slope consisting of an upper natural slope 4.6 m (15 ft) high and a lower cut slope height of 3.4 m (11 ft). From the survey STABL was used to analyze the new slope geometry shown in Figure 4-4 and a new factor of safety equal to 1.17 was calculated. SNAIL was used to calculate as-built factors of safety for the nailed slope. For the as-built analysis it was assumed the first row of nails were placed from 0 - 0.3 m (0 -1 ft) below the top of the cut slope. Section 2 was not evaluated in the as-built analysis. Changes in Section 2 due to construction errors, poor weather, and local failures resulted in a slope geometry and nail pattern that was not comparable with the other sections. Therefore, only Section 1 and Section 3 have reported factors of safety. Since the nails were installed in an irregular pattern, a typical nailing scheme was assumed for the analysis. The assumed nail lengths and spacings are shown in Table 4-4. The resulting factors of safety are compared in Table 4-5, and compared graphically in Figure 4-5.

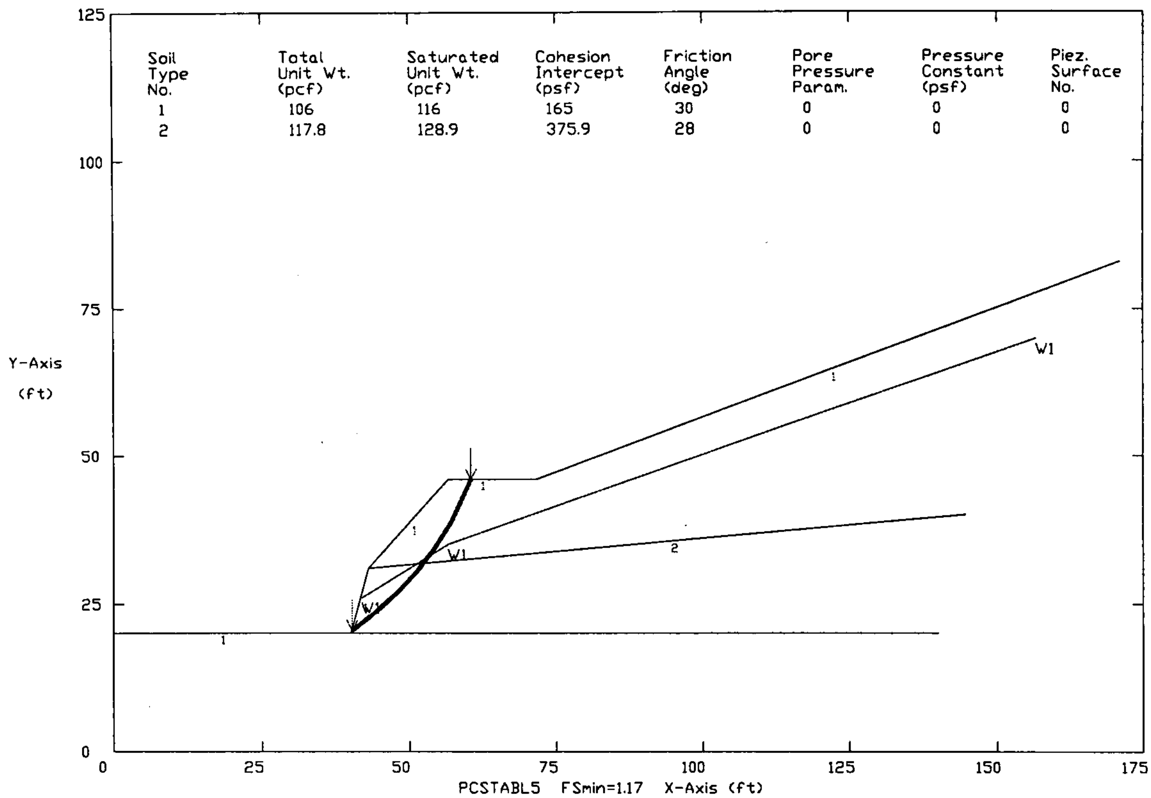

Figure 4-4 STABL Analysis of As-built Slope

Table 4-4 Nailing Scheme for As-built Cut Slope

Design Nail Spacings	Section 1		Section 3	
	Sh= 1.5 m (5 ft)	Sv= 1.5 m (5 ft)	Sh= 1.5 m (5 ft)	Sh= 1.5 m (5 ft)
Nail Length, Row 1	5.8 m (19 ft)		4.0 m (13 ft)	
Nail Length, Row 2	5.8 m (19 ft)		2.7 m (9 ft)	
Nail Length, Row 3	5.8 m (19 ft)		2.7 m (9 ft)	

Table 4-5 Summary of As Built Factors of Safety from Stability Analysis

Analysis Method	Program	Section 1	Section 3	Section 4 Unreinforced
Stocker and Riedinger (1990)	SNAIL	1.86	1.59	1.01
Gässler and Gudehus (1981)	EDINA	3.34	1.54	N/A
Juran et al. (1990)	FHWA Method	4.0	2.5 Constant Lengths	N/A
Shen et al.(1981)	NAIL	1.79	1.13	N/A
Bishop (1955)	STABL5	N/A	N/A	1.03

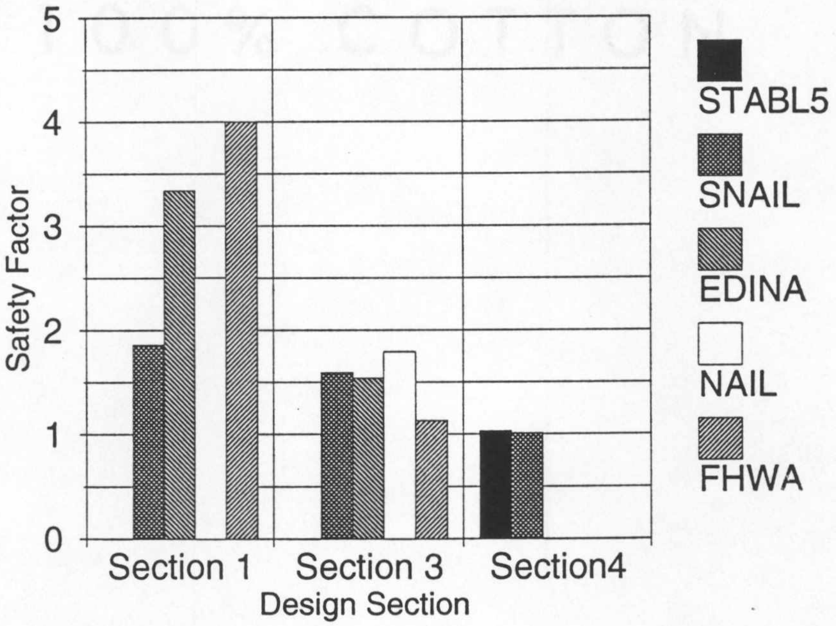

Figure 4-5 As-built Safety Factor Comparison

4.4 Analysis of Control Section 4

Following the completion of Sections 1,2 &3, control Section 4 was cut to study the stability of an unreinforced section. A near vertical slope 4.6 m (15 ft) high was excavated and failed 4 hours after being cut. It was noticed the failure was a very shallow failure and continued to "peel" off in layers over a 10 minute period. The slope eventually stabilized and has not changed significantly since the final small sheet failure. The slope was expected to fail based on observations in March 1996 when an access road below the slope was over excavated resulting in a slope failure. The over excavated slope failed with an unsupported height of 4.0 m (13 ft).

Once the Section 4 slope had stabilized, an optical survey was conducted and the shape of the failure scarp estimated. The slope profile for Section 4 was then reconstructed and the measured failure surface analyzed using STABL5. The resulting safety factor was equal to 1.05. The low factor of safety is close to 1.0 which suggests the slope profile and material properties have been properly estimated. The slope profile as well as the failure surface are shown in Figure 4-6.

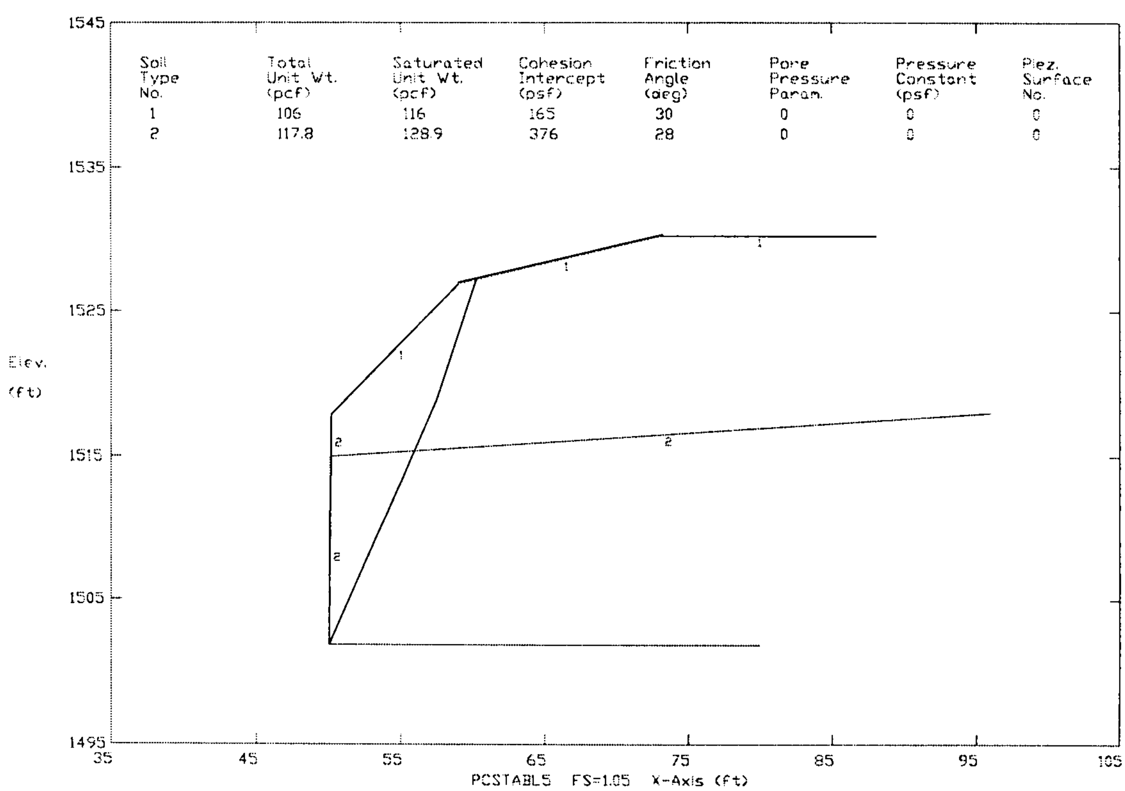

Figure 4-6 Section 4 Profile and Failure Surface

The analysis was repeated allowing the STABL5 search routines to identify the critical failure surface. This yielded a factor of safety equal to 0.98. The failure surface from the analysis is shown in Figure 4-7. After the initial failure of the slope, small failures continued in Section 4 until the slope stabilized at another, more stable inclination. The predicted failure surface in Figure 4-7 resembles the resultant surface observed in Section 4.

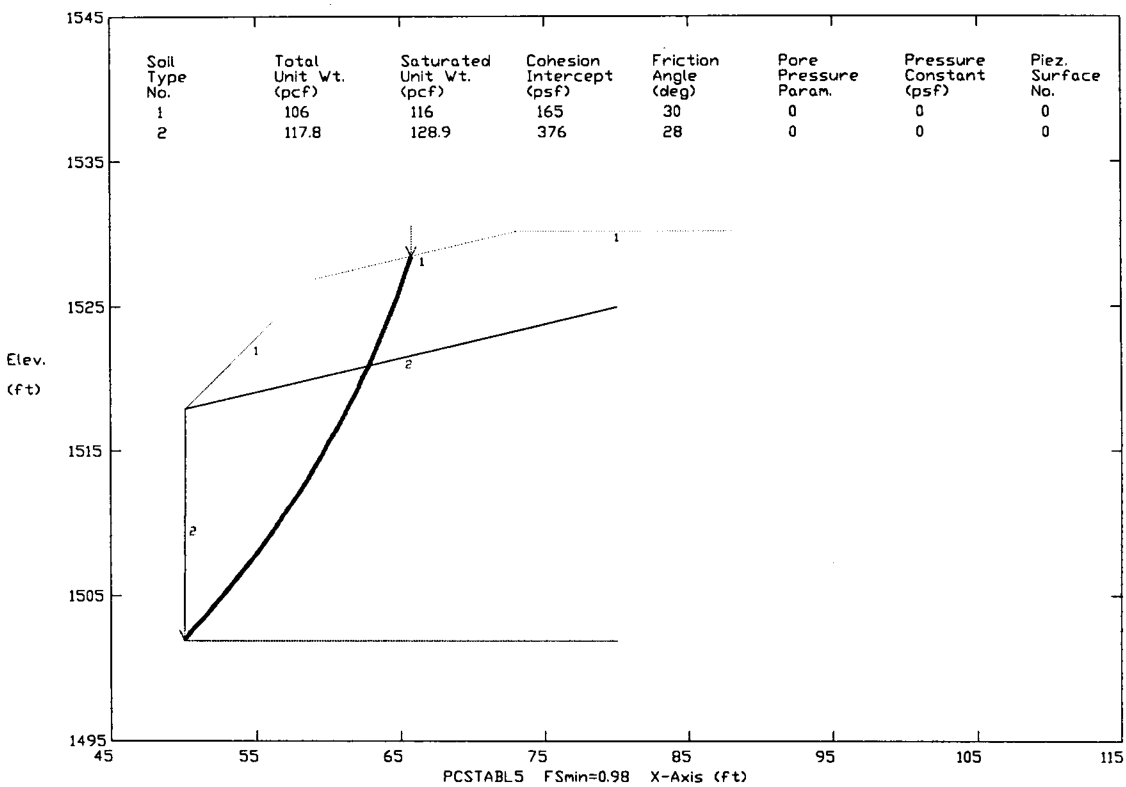

Figure 4-7 Section 4 Profile of STABL5 Random Analysis

4.5 Facing Design and Nail Plate Design

Like many abandoned mine land sites, Buffalo Mountain is very remote and not well suited to the application of shotcrete on the face which is typical soil nailing practice. Therefore, a reasonable facing which can be easily delivered and installed was selected.

Instead of the conventional shotcrete and wire mesh facing, which is somewhat rigid, a geosynthetic facing was installed, and held in place by steel plates attached to the nails. The facing for the slope was composed of three elements as shown in Figure 4-8. A reinforced

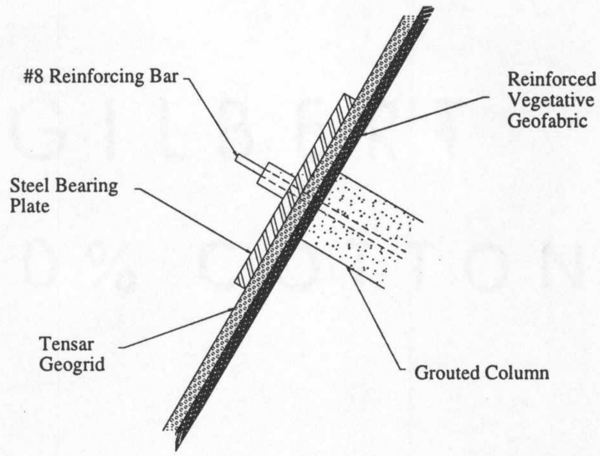

Figure 4-8 Cross Section of Soil Nailed Slope Facing

vegetative geofabric (Erosion Control Systems 1995) was selected for its ability to promote growth of grass. For added strength and reinforcement Tensar BX1100 biaxial geogrid was used over the vegetative mat. Finally, steel plates 0.45 m x 0.45 m (1.5 ft x 1.5 ft) were placed on the nail heads to hold the fabric in place and to insure an intimate contact with the slope. Properties for the erosion control blanket and the geogrid are provided in Table 4-6. It has been suggested (Santha and Santha 1996) that it is very important for the erosion control facing to be placed securely and evenly against the slope face. Loose or uneven material will reduce the effectiveness of the vegetative mat and the erosion prevention of the fabric. Although the rough nature of the mine waste slope made this difficult, most areas of the facing were able to be placed into intimate contact with the slope face.

Table 4-6 Properties of Geosynthetic Materials

Product	Erosion Control Mat	Geogrid
Manufacturer	Erosion Control Systems	Tensar
Product ID	Earthlock	Tensar BX1100 Geogrid
Material	polyester	polypropylene
Weight (ASTM D3776)	8.0 N/m^2	N/A
Thickness (ASTM D1777)	Ribs - 40 mils	Ribs - 0.762 mm (0.03 in)
	Junctions - 60 mils	Junctions - 0.11 mm (2.79 in)
Aperture Size	20 mm x 20 mm (0.79 in x 0.79 in)	25.4 mm x 33.02 mm (1.0 in x 1.3 in)
Wide Width Tensile Strength @ 2%Strain	5 kN/m (warp) (320 lb/ft) 3 kN/m (fill) (180 lb/ft) (ASTM D4595)	19 kN/m (warp) (1260 lb/ft) 11 kN/m (fill) (765 lb/ft) (GRI GG2-87)
Elongation at break	2%	2%
Erosion Control Mat Material	Bio-degradable curled wood excelsior mulch	N/A

To design the bearing plates for the nails, the largest total nail force was found from the SNAIL analysis. This force was assumed to be resisted by the nail friction mobilized along the portion of the nail inside the assumed failure plane. The difference between the total nail force and the force mobilized along the nail was assumed to act on the nail head. This force was used in the bearing plate design based on Terzaghi's bearing capacity equation. Final plate dimensions of 0.45 cm x 0.45 cm (1.5 ft x 1.5 ft) by 3.2 mm (0.125 in) thick were chosen. The calculations are located in Appendix 3.

Chapter 5

Construction and Slope Instrumentation

5.1 Construction

Construction of the soil nailed slope began in November, 1995. Following three months of mobilization, equipment setup, site preparation, development of appropriate equipment, and delays due to poor weather, the first nail was placed March 8, 1996. Since the first nail was placed three months after the initial start date, the date of the first nail placement was adopted as the start of construction. Table 5-1 chronicles major events during the construction and monitoring period. The table also lists the day of construction, as referenced to March 8, 1996, that the event took place.

Table 5-1 Construction Events

Chronology of Construction Events		
Date	Event	Construction Day
3/8/96	First Nail Placed	0
5/1/96	First Lift Excavation (Started)	74
5/21/96	Second Lift Excavation (Started)	94
5/22/96	Third Lift Excavation (Started)	95
5/27/96	First Major Rain Event	100
6/30/96	Second Major Rain Event	134
7/12/96	Construction Completed	146

5.2 Instrumentation of Slope and Nails

To monitor the stresses in the nails and the displacement of the slope, two methods were used. Vertical inclinometers were placed in each of the test sections and in the roadway above the slope to monitor the slope before, during, and after construction. The inclinometers were installed in boreholes advanced to a sandstone unit about 8 m (26 ft) below the roadway elevation. Three inclinometers were used to monitor slope deflections in Section 1,3, and 4. Two more were added later in the roadway between Sections 1 and 2, and between Sections 2 and 3.

To record stresses in the nails, strain gages, model EGP-5-350 manufactured by the Micro Measurements Group (Figure 5-1) were utilized. In the top section or "natural" part of the slope, there are two instrumented columns of nails (Section 1 and Section 3) as shown in Figure 5-2. Both sections have the top three nails instrumented with three gages, each along the length of the nail. The gages are placed at 99 cm (3.2 ft), 180 cm (5.9 ft), and 231 cm (7.6 ft) along the nail relative to the facing end. The instrumentation

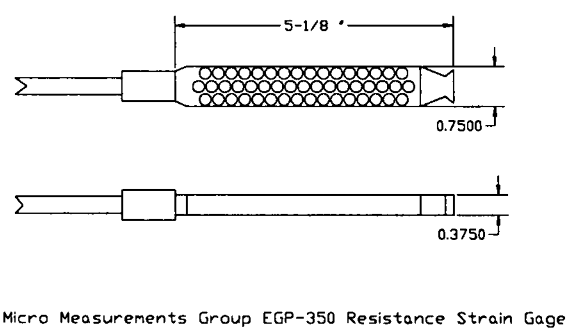

Figure 5-1 Embedment Strain Gage

scheme for the various nail lengths is listed in Table 5-2. A minimum of four resistance gages were used for a single nail. The nails were installed along the neutral axis of the nail to avoid the measurement of bending stress.

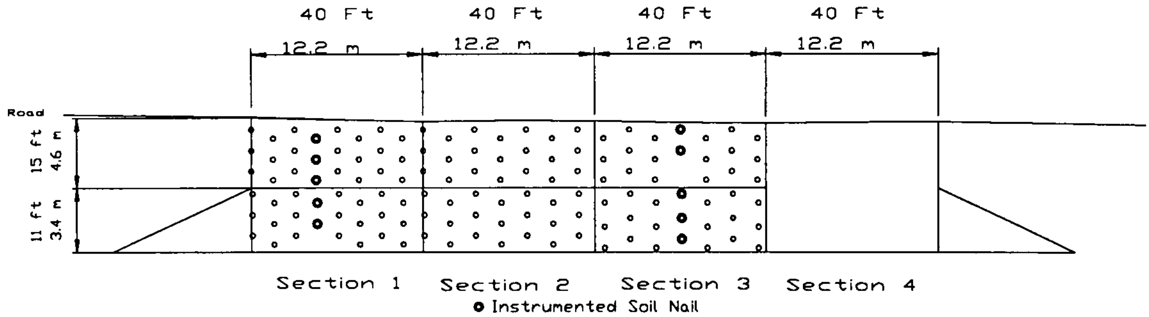

Figure 5-2 Elevation View of Instrumented Slope

Table 5-2 Summary of Strain Gage Placement for Nail Lengths in Lower Cut Slope

Nail Length	Gage 1*	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7
6.1 m (19.5 ft)	45 cm (1.5 ft)	120 cm (4.0 ft)	195 cm (6.4 ft)	270 cm (8.9 ft)	345 cm (11.3 ft)	420 cm (13.8 ft)	495 cm (16.2 ft)
4.6 m (14.5 ft)	45 cm (1.5 ft)	120 cm (4.0 ft)	195 cm (6.4 ft)	270 cm (8.9 ft	345 cm (11.3 ft)	Not Used	Not Used
3.0 m (9.5 ft)	45 cm (1.5 ft)	120 cm (4.0 ft)	195 cm (6.4 ft)	270 cm (8.9 ft)	Not Used	Not Used	Not Used

* Location of all gages are relative to the facing end of the nail and are installed at the neutral axis to eliminate effects due to bending.

The gages were attached directly to the steel rebar prior to filling the nail hole with grout. Assuming that the strain in the grout and nail are equal, and the cross sectional area of the grout is much larger than that of the rebar, the stress in the nail is obtained by multiplying the strain measured in the resistance gage by the grout modulus. Compression tests on 152 mm x 305 mm (6 in x 12 in) grout cylinders yielded an average modulus of 4 x 10^7 kPa (5.8 x 10^6 psi). Data from the tests are provided in Appendix 1.

Chapter 6

Slope Monitoring and Interpretation of Results

6.1 Results of Instrumented Soil Nails

The slope was monitored during construction (April 17, 1996 to July 1, 1996), and has continued to be monitored since. Stresses in the nails began to slowly increase as lifts were excavated and new rows of nails installed. Typical plots of stress versus time for Sections 1 and 3 are shown in Figures 6-1 and 6-2. In Figure 6-1, the time corresponding to some major rain events are shown as well as the dates corresponding to the excavation of the first, second and third lifts of the lower slope. The most recent measurements suggest that nail stresses are still increasing, but the rate has decreased. The results for all nails in Sections 1 and 3 are in Appendix 4.

The initial day for the stress readings in each nail varies. Following installation of the nails, the tensile force reduced as the nails went into compression due to shrinkage of the grout. However, after the shrinkage in the grout ceased, and excavation of the lower cut slope began, the nail tensile stresses began to increase. The reported nail stresses begin on the day the nail stresses begin to increase, and are referenced to the number of days into construction as described in Chapter 5. In Figure 6-3 and Figure 6-4 the nail stresses in Section 1 and Section 3 are shown along the nail for two times during the monitoring process. The graphs of nail stress are superimposed on a schematic of the slope profile. Also shown is the location of the most critical failure surface from the SNAIL analysis. Most analysis methods assume the critical failure surface intersects the

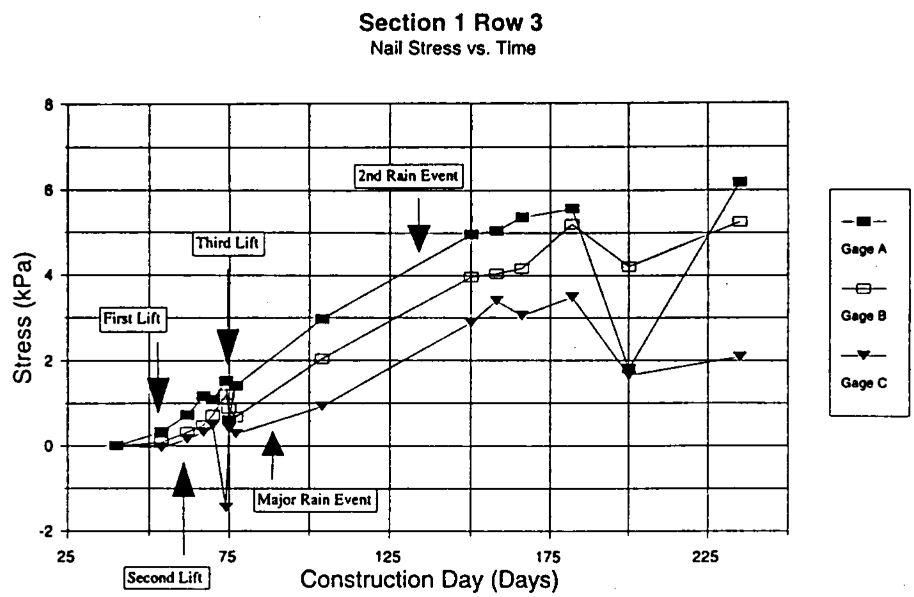

Figure 6-1 Nail Stress vs Time (Section 1 Row 3)

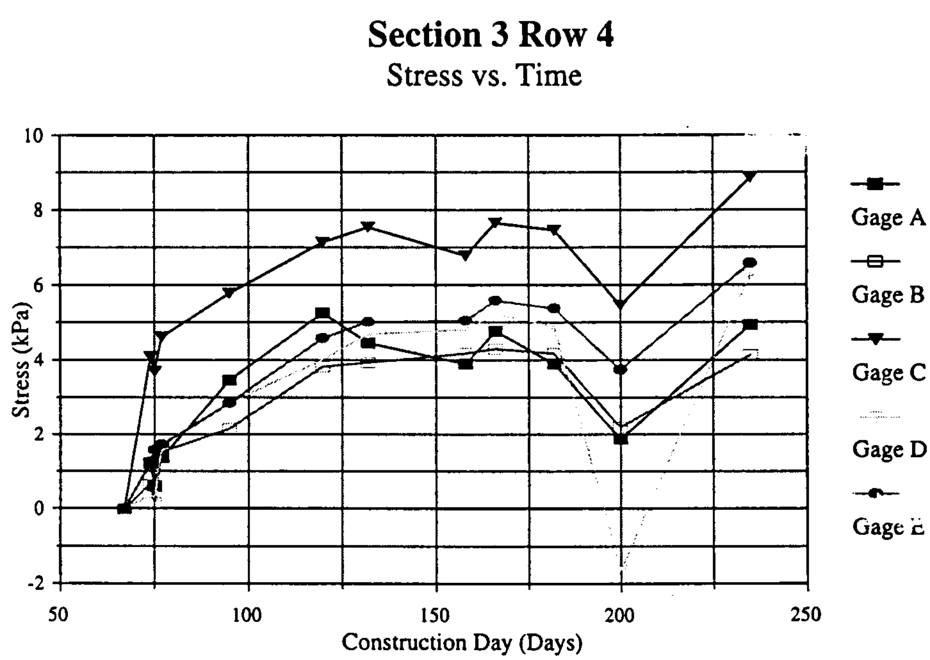

Figure 6-2 Typical Plot of Stress vs Time (Section 3 Row 4)

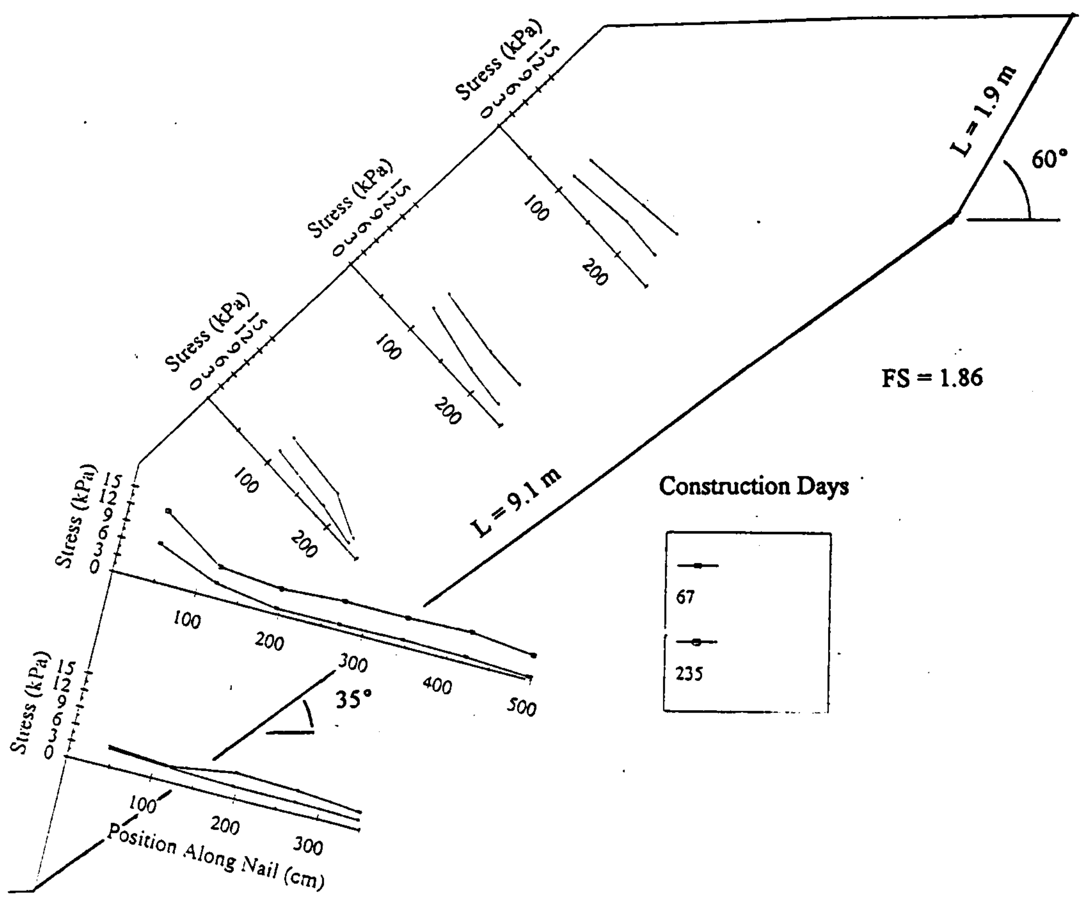

Figure 6-3 Section 1 Nail Stresses

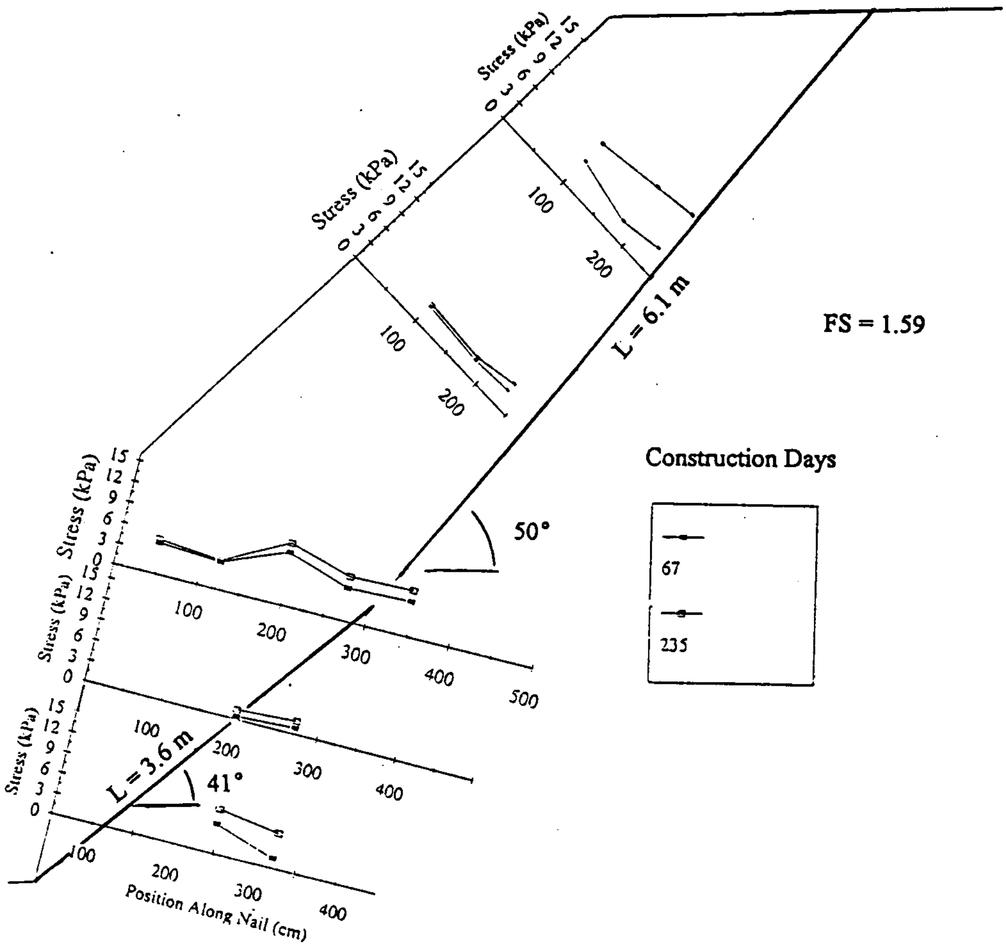

Figure 6-4 Section 3 Nail Stresses

soil nail at the location of maximum tension. The measured nail stresses are relatively uniform and do not indicate the existence of a zone of maximum stress. This is likely due to the high computed factors of safety for the reinforced sections, and reflects the relatively low levels of measured nail stress.

Although Figures 6-1 to 6-4 indicate that the nail stresses have increased over time, the calculated stresses are relatively small compared to the maximum stress the nails are capable of withstanding. The measured nail stresses cannot be compared directly with those from the pullout tests since the tensile stress in the nail depends upon the length over which the shear stresses are mobilized. The tensile stress in the nail is produced by the mobilized skin friction along the length of the nail and is calculated as follows:

$$\sigma_{nail} = \frac{\text{Tensile Force}}{\text{Cross Sectional Area}} = \frac{F_N}{A} = \frac{F_N}{\frac{\pi d^2}{4}} \qquad \textbf{(31)}$$

$F_N = \tau_{ult} \times A_{surf}$

$A_{surf} = \pi dL$ = tensile force in nail

d = diameter of grouted nail

L = length of nail over which skin friction is mobilized

τ_{ult} = ultimate skin friction measured in pullout test

Therefore, σ_{nail} depends upon A_{surf} or the available nail length to mobilize τ_{ult} as:

$$\sigma_{nail} = \frac{\tau_{ult}\, 4\, L}{d} \qquad \textbf{(32)}$$

The nail pullout tests found τ_{ult} for the MrE and UDE soil to be 13.6 kPa (1.97 psi) and 33.7 kPa (4.9 psi). The relationship between σ_{nail} and the length over which τ_{ult} is mobilized is shown for both the MrE and UDE soils in Figure 6-5.

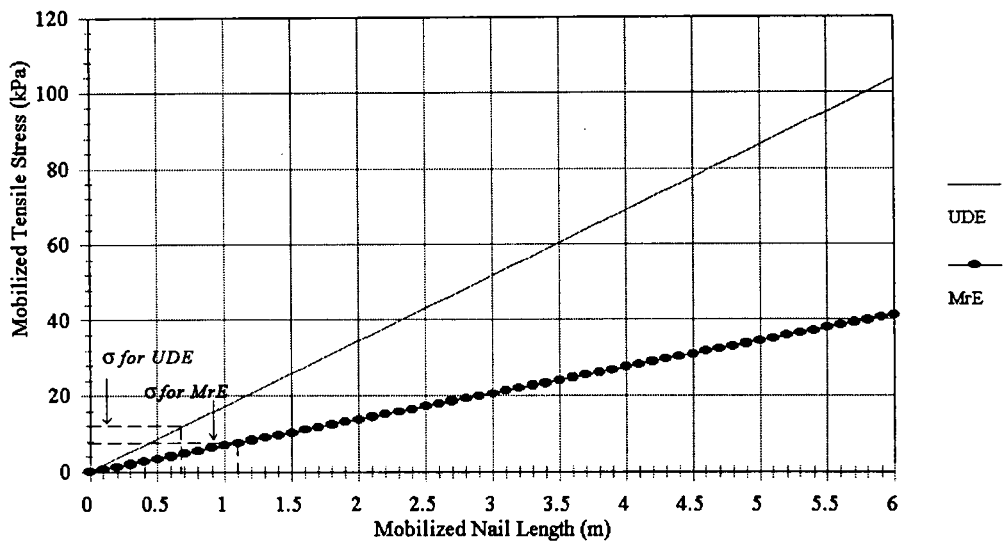

Figure 6-5 Mobilized Nail Stress

For measured tensile stresses of 12 kPa in the UDE soil, Figure 6-5 suggests that a mobilized nail length of about 0.7 m is sufficient to generate the measured stress. Similarly for the MrE soil, for a measured stress of 8 kPa, only about 1.1 m of nail is required. Since lengths significantly greater than these values are available, it can be concluded that the shear stresses along the nails are much less than τ_{ult}.

6.2 Results from Slope Inclinometers

Readings on the inclinometers reflect only very small movements in the slope. A typical plot of the slope inclinometer is shown in Figure 6-6. Deflection of the slope was expected since soil nailing is a passive reinforcement with displacement required to induce tension in the nails. The newer inclinometers located in the road have not recorded any displacements. This is not surprising because the inclinometers were installed after the final excavation and nail installation.

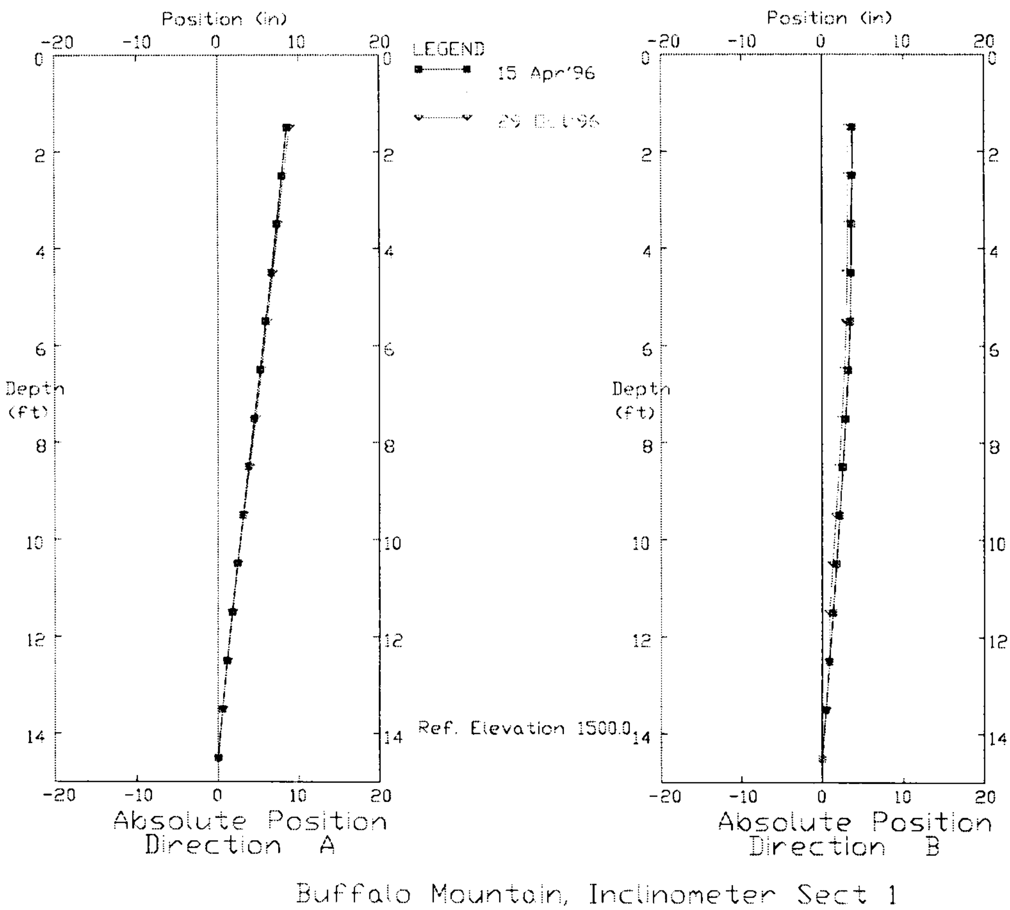

Figure 6-6 Typical Plot of Inclinometer Section 1

6.3 Observations of Slope Facing Performance

Since completion of the slope, the facing material played an important role in the stability of the slope because of the mine waste's susceptibility to erosion. Two major rain events occurred during construction and following both rain storms, the sections of the slope not properly covered with the fabric suffered extensive erosion and slope failure. After repairing those sections and properly placing the erosion control mat and geogrid, the slope has remained stable and unaffected by other rain events. Along the slope, localized bulges were observed in the slope face where small "pop-out" failures occurred due to the facing being too loose following construction. However, the bulges do indicate the facing is picking up load from the soil mass. This supports the need for using a geogrid on the slope. Without the geogrid, the soil mass of the "pop-outs" could have induced tensile failure in the erosion control mat and resulted in localized slope failure. Following construction, the slope was seeded with grass to cover the slope and to prevent the erosion of finer soil particles under the facing. The slope is currently covered with large patches of grass, and grass is expected to cover most of the slope in due time.

6.4 Conditions of Existing Slope

Shortly following the completion of slope construction, an optical survey was conducted to establish an elevation view of the completed slope, and to review the final nail spacings. The data was collected and the slope drawn as shown in Figure 6-7. As can be seen the nailing pattern was irregular, and does not closely resemble the original slope design. Traditional stability analysis methods cannot be applied directly to such a slope. An equivalent nail pattern must be obtained to permit analysis of the existing sections.

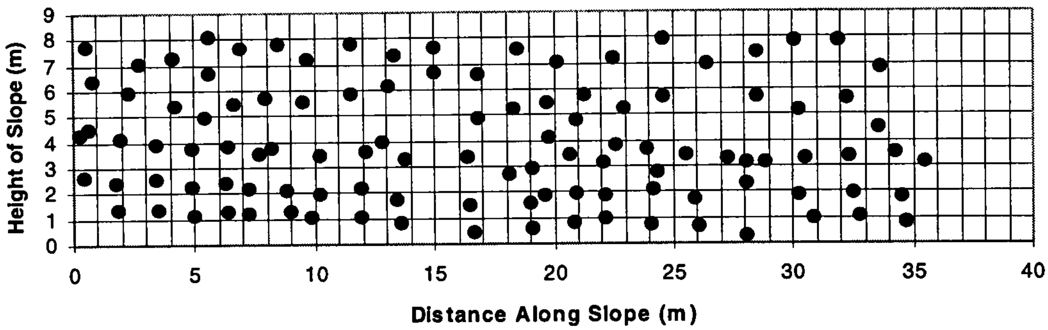

Figure 6-7 Elevation View of Slope

6.4.1 Determination of Equivalent Nailing Pattern

The first step in determining an equivalent nailed slope is to evaluate the nail spacings. To do so the elevation view of the slope was divided into a grid 1.5 m x 1.5 m (5 ft x 5 ft) which was the design spacing in Section 1. The grid was located such that the upper left grid intersection was centered between the first two rows and first two columns of the original design configuration. The intersection is located at the height of 8 m and 2 m along the slope in Figure 6-7. A circle having the diameter of two grid squares 3.0 m (10 ft) was overlain at each grid intersection, and the number of nails, n, inside the circular area counted. A schematic of the grid, circles and counting method is shown in Figure 6-8. The number of nails in the circular area was then divided by the area of the circle to calculate the nail density ρ. This resulted in a number of nails or nail density assigned to each grid point. A contour map of the nail density is shown in Figure 6-9.

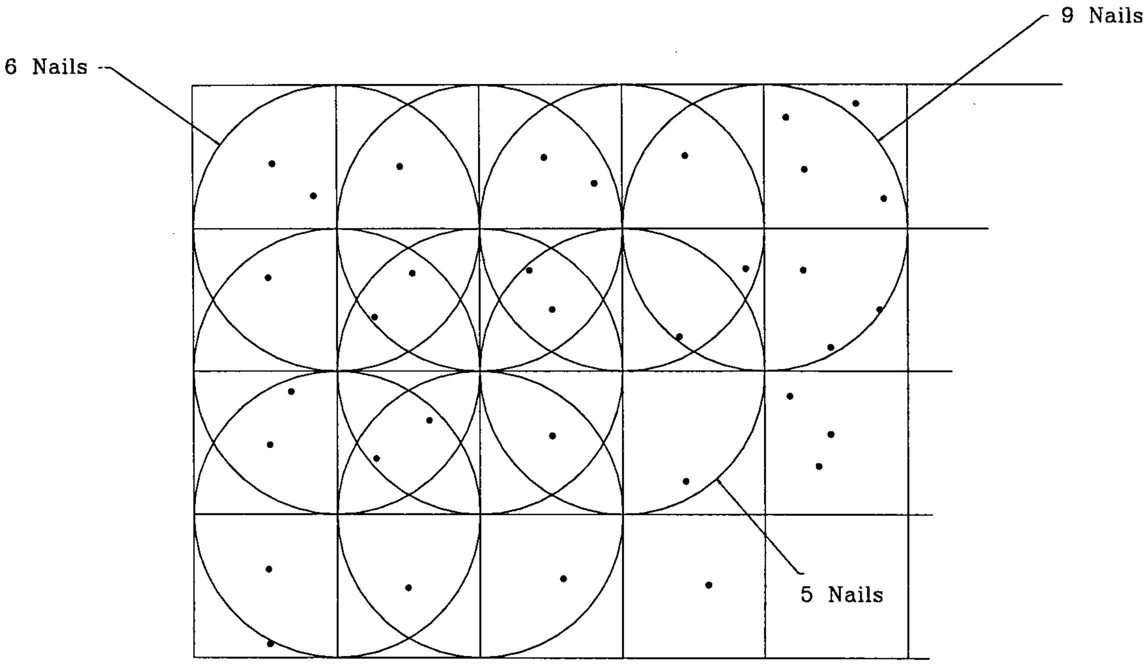

Figure 6-8 Method of Counting Nails

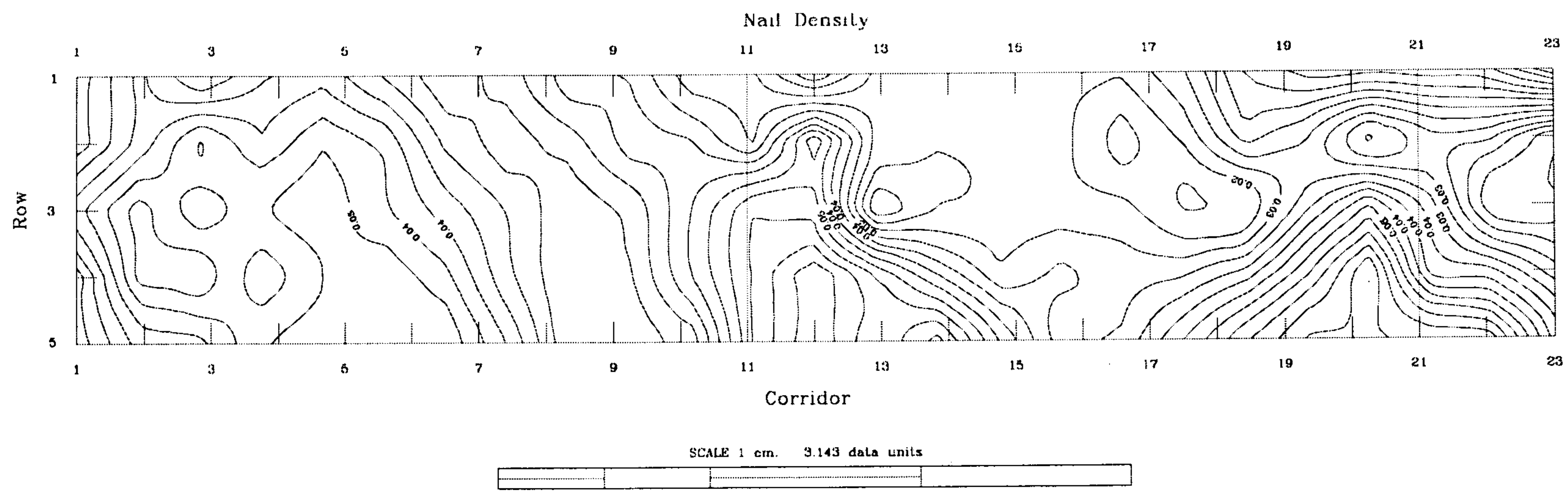

Figure 6-9 Contour Map of Nail Density

From the contours of nail density, the left portion of the slope (Section 1) appears to have a more consistent nailing pattern than Sections 1 and 2. This is also apparent by inspection of the nail locations in Figure 6-7.

From the nail density values, equivalent horizontal and vertical spacings could be calculated. As designed, the slope has an ideal design spacing, S (Figure 6-10) and design number of nails. To calculate an equivalent spacing an equation was derived based on the nail density.

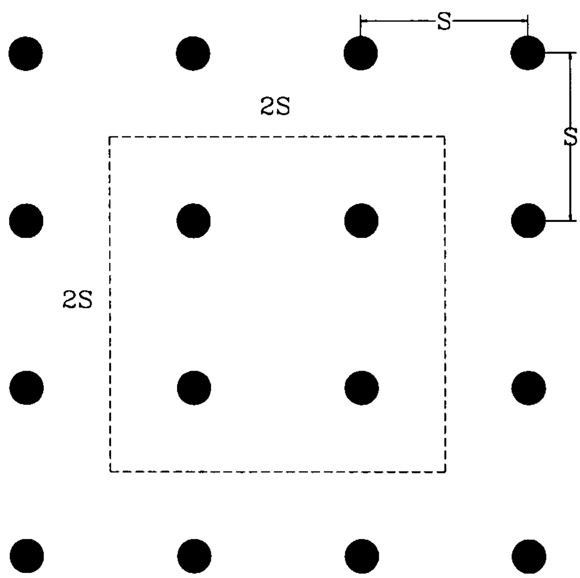

Figure 6-10 Measurement of Nail Spacing

Let S = design nail spacings, with S_h = horizontal and S_v = vertical

and assume $S_h = S_v$

A = area of nailed zone for the idealized nail spacing Figure 6 -10

$$A = 4S^2 \quad \textbf{(33)}$$

then solve for S

$$S = \sqrt{\frac{A}{4}} \tag{34}$$

The ideal slope is based on a constant nail spacing. For a specific design spacing there is a corresponding area A, and a particular number of nails located in that region. For the area defined by a given nail spacing there will be a certain number of nails. In an area defined by one vertical spacing and by one horizontal spacing, there should be one nail. Therefore, for an area corresponding to n spacings, there are n nails. It could be assumed that a general equation for the nail spacing in a square grid would be as follows:

$$S = \sqrt{\frac{A}{n}} = \sqrt{\frac{1}{\rho}} \tag{35}$$

where

S = equivalent nail spacing

n = number of nails

ρ = nail density = n/A

The two examples below verify the accuracy of the equation.

Example one:

From Figure 6-10 $\quad A = 4S^2 \quad n = 4 \quad \rho = 1/S^2$

$$S = \sqrt{S^2} = S$$

Example two:

From Figure 6-11

$$A = 6S^2 \qquad n = 6 \qquad \rho = 1/S^2$$

$$S = \sqrt{S^2} = S$$

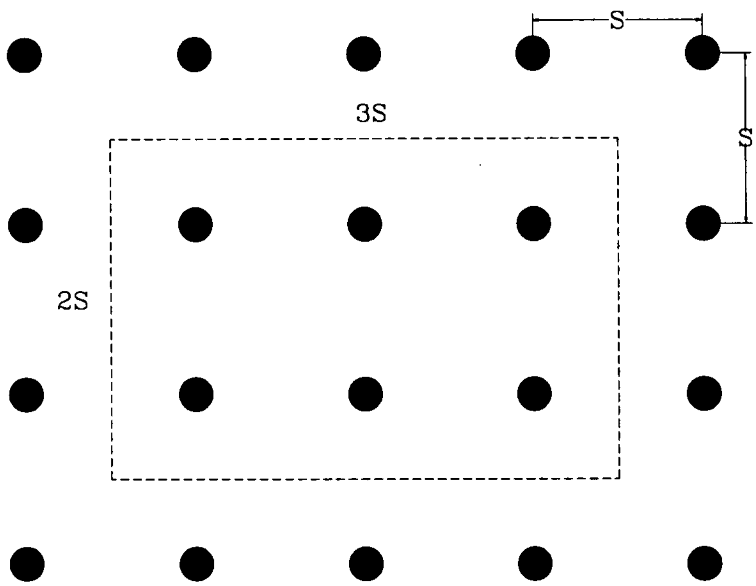

Figure 6-11 Verification of Spacing Equation

As shown in the two examples, the developed equation can be used to calculate the nail spacing based on the area, A and the number of nails, n.

Furthermore, a coefficient can be included to account for unequal vertical spacings (S_v) and horizontal nail spacings (S_h).

When $S_v \neq S_h$

If $\quad S_v = \alpha S_h$

$\alpha = S_v/S_h$ = spacing coefficient (For example, if S_v=5, S_h=6, then α=5/6) then $$A = (2S_h)(2S_v) = (2S_h)(2\alpha S_h) = 4\alpha S_h^2$$

where $$S_h = \sqrt{\frac{A}{4\alpha}} \qquad \textbf{(36)}$$

Generally, the equivalent horizontal spacing is expressed as the following:

$$S_h = \sqrt{\frac{A}{n\alpha}} = \sqrt{\frac{1}{\rho\alpha}} \qquad \textbf{(37)}$$

and the equivalent vertical nail spacing is expressed:

$$S_v = \alpha S_h = \sqrt{\frac{\alpha}{\rho}} \qquad \textbf{(38)}$$

These relationships can be used to determine the equivalent nail spacing for the slope to investigate the effect on computed factors of safety.

6.4.2 Equivalent Spacings and Factors of Safety

From the counted nails and densities, equivalent nail spacings for the slope were calculated. The calculated number of nails n, nail densities ρ, and equivalent spacings S corresponding to each grid intersection are located in Appendix 2.2. Contour plots of the horizontal spacings, and vertical spacings are located in Appendix 4.

The calculated equivalent spacings were assigned to vertical corridors of nails as shown in Figure 6-12. The corridor is made up of two sets of nail halves, with each set having an equivalent horizontal and vertical spacing associated with it. Since only the lower portion of the slope can be analyzed in SNAIL, the mean spacings for the lower nails were

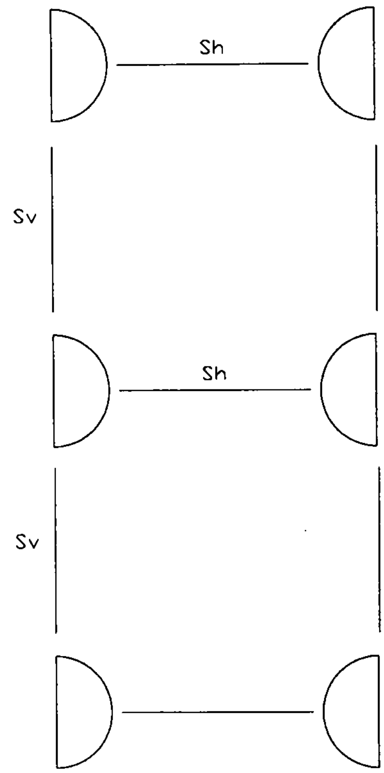

Figure 6-12 Nail Corridor

calculated and input into the SNAIL analysis program. The as-built slope cross section was used in the analysis. The resulting factors of safety for each vertical corridor of nails is plotted in Figure 6-13. This plot reveals how the factors of safety decrease across the slope from Section 1 to Section 3. These values can be compared with the factors of safety calculated from SNAIL for the as-built spacings which were 1.86 (Section 1) and 1.59 (Section 3) as shown in Table 4-5. It is evident in the nailing density contour, some empty, wide open sections are visible around corridors 10 - 16. These areas reflect the lack of nails and lower computed factors of safety in this area. It was in these areas that small failures occurred during the construction period. These areas were repaired and the repairs resulted in the addition of nails to Section 2, which altered the nail density, changed the design nail spacings, and affected the computed factor of safety shown in Figure 6-13. It should be noted

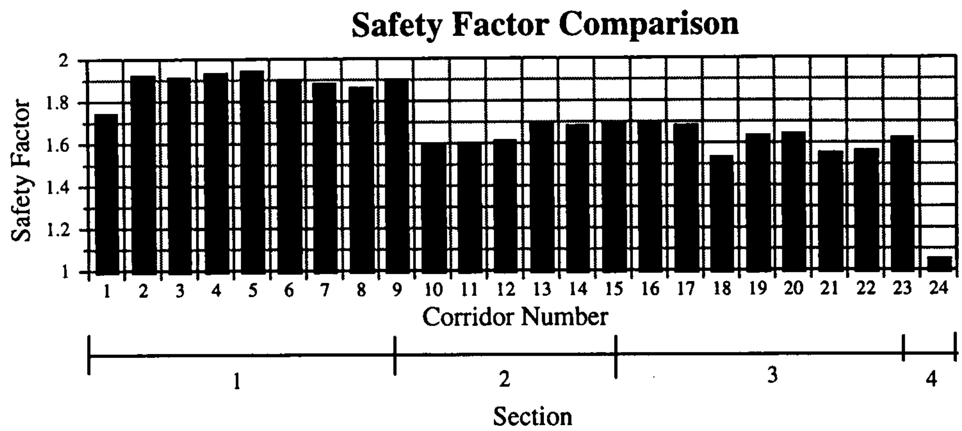

Figure 6-13 Safety Factor Comparison For Corridors

that the actual slope profile in the area of corridors 10-16 or Section 2 is very different than Section 1 and Section 3, and that the computed factors of safety in this section do not correspond to the actual as-built conditions. However the original design goal of having varying factors of safety for each of the design sections was maintained. The proposed method of equivalent nail spacings provides a means of analyzing a nailed slope for which the nails are not installed on a regular pattern.

Chapter 7

Conclusions and Recommendations

7.1 Conclusions

Following completion of construction and monitoring of the soil nailed slope, it is apparent that soil nailing is a feasible alternative for stabilizing mine waste, and dumped or loosely placed fill. As shown in Chapter 4 the nail lengths, nail spacings, slope height, and slope inclination can be easily adjusted to accommodate changes due to irregularities in the slope material such as large rock inclusions, multiple soil layers, and the presence of bedrock. The effects of the irregularities can be analyzed, and a new factor of safety for the as-built slope determined. From the as-built slope analysis in Chapter 4 and the equivalent slope analysis in Chapter 6 it is apparent the slope was over designed. However, the nail stresses continue to increase indicating that equilibrium may have not yet occurred, and conclusions about the state of stress in the nails cannot be made at this point. Five positive conclusions can be drawn from this study.

1. The project slope was significantly steepened to explore more fully the application of nails for stabilizing mine waste slopes. Shorter nails would have been appropriate if the slope was to have been stabilized at the original inclination. By using shorter nails, risk of surface erosion could have been reduced by using smaller nail spacings which would have reduced the exposed area between nails and prevented the small slope failures encountered in Sections 2 and 3.
2. Subsurface conditions were different than initially anticipated. Materials

and soil conditions not detected in the subsurface investigation were encountered during construction and the nail spacings and nail lengths had to be adjusted. This suggests soil nailing is especially suitable since the design can be easily modified during construction.

3. The geofabric and geogrid conformed easily to the irregular surfaces on the slope, and could be arranged around large boulders. The facing material also allowed for a free draining face to reduce pore pressure due to groundwater, and the facing could be easily transported to the site. Good contact between the slope surface and the covering is important. Shotcrete and precast panels would have been difficult and economically unfeasible to use on the project.

4. Due to the remote nature of most mining sites, mobilization of equipment and transportation of large quantities of materials is not economical. Soil nailing can be accomplished with relatively light equipment and minimal quantities of material.

5. The flexibility of the nailing process is not only important in the construction of irregular slopes, but it is also important should repairs need to be made. In the event a failure occurs during construction, the nailing scheme can be adjusted to repair damage to the slope. Spacings can be reduced, nail density increased, and nail lengths altered. The resulting as-built slope's stability can then be evaluated using equivalent mean nail spacings as shown in Chapter 6.

6. Finally, soil nailing can allow for a stabilized slope that lends aesthetic beauty to the surrounding environment. Following placement of the facing material, the slope can be seeded with an appropriate blend of grass. Once established, the grass will allow the slope to blend with the environment. This not the case with tie back walls or cast in place concrete retaining walls.

Although the soil nailing was successful, some simple guidelines and practices could have helped in the success of the project. A more thorough subsurface investigation, better communication with the contractor, and higher quality control would have improved the project. Overall, soil nailing appears to be an applicable and satisfactory method for stabilizing mine waste.

7.2 Guidelines For The Nailing of Mine Waste Slopes

Soil nailing is a process that requires different phases of construction to achieve a quality product. Below are simple guidelines which can be followed to ensure that a quality and stable product is achieved.

1. **Subsurface Investigation.** The subsurface investigation can reveal many important facts about the slope conditions and the properties associated with the slope. The random nature of mine spoil suggests that split spoon samples and Shelby tube samples may not be easily obtained, but auger cuttings may be very useful. If possible, boreholes should extend at least 3.0 m (10 ft) below the toe of the proposed soil nailed wall. One of the advantages of soil nailing in mine waste is that changed subsurface conditions may be easily accommodated. However, a

good subsurface investigation program is important.

a. **Soil Structure Profile**. From the subsurface borings and driller's log during nail installation an approximate subsurface profile can be created to aid in the design and analysis of the soil nailed slope. It is very important to know in which soils the nails will be placed. It is very apparent that mine waste is not homogenous, and the materials on the surface of the slope may not be the same as those in which the nail friction will be mobilized. Material properties may vary greatly between layers, and a poor estimation of the profile could result in a lower safety factor.

b. **Water Table.** Following the completion of the boring, test wells can be installed to detect a water table in the slope. Water can greatly reduce the stability of the slope. If test wells are not possible, an open borehole can be used to estimate the water table.

c. **Soil Samples.** From the drilling process various types of soil samples can be collected for testing. If possible, split spoon and Shelby tube samples can be collected. If the mine waste is comprised of loose, uncompacted material with many large particles, it may be impossible to collect a sample. Therefore, loose cuttings can be collected and remolded for testing. From the samples material properties can be evaluated and determined for the soil nail design. In place testing such as the borehole shear test may also be useful.

2. **Design of Soil Nailed Slope** Following the determination of the soil profile and

soil properties, the slope can be designed. There are two aspects of the design process that are important.

a. **Selection of Design Method**. There are many different design methods available today for soil nailing. In the selection process it is important to know which analysis method is best for the project. Some can accommodate water tables, inclined slopes, varying nail lengths, soil layers, and surcharges. Two simple and readily available design methods are the French and Kinematical Design methods. Both methods have design charts so that calculations can be done without the need of a computer. The Kinematic Design method is a final design, whereas the French design charts are for preliminary design calculations. Of the available computer codes SNAIL which is based on the German method appears to be a reliable and accurate design method.

b. **Selection of Design Parameters.** It is very important to select the correct soil nailing parameters for the wall design. Perhaps one of the most important is the nail spacing. By reducing the nail spacing, the importance of the facing material is reduced. It is best when designing to vary and experiment with the design code, and see what is the best combination of wall inclination, vertical and horizontal spacing, and nail lengths.

3. **Construction Process**. There are five important aspects of the soil nailing process that should be of concern during construction.

a. **Quality Control.** This is the most important part of the soil nailing process. It is very important that the design be implemented properly during the construction. Good communication with the contractor and design engineer should be a top priority. Regular visits should be made, or a full time technician should be part of the construction process. This will ensure the design is properly implemented. Results of poor construction can be structurally and aesthetically devastating to the project. Incorrect nail spacings, lengths, and inclinations can result in an unsafe and unstable slope. Furthermore, if excavation is involved, over excavation can result in local or total slope failure. Keeping a simple analysis program readily available onsite would allow for more sound, and safe adjustments to be made. Nail spacings and lengths may need to be adjusted due to large rock inclusions such as boulders, tree root balls and even bedrock. The equivalent spacing method described in Chapter 6 may prove useful in analyzing design changes in the field. Finally, a procedure should be developed between the contractor and engineer to stipulate the allowable variation from the design.

b. **Slope Protection.** During and following construction it is very important that the slope be protected from the elements. A heavy rainfall on a fresh cut or face can have devastating effects on slope stability. It is good practice to cover the exposed slope with plastic sheets and install the design facing material as soon as possible. If using a geosynthetic material for the facing, make sure there is an intimate contact between the slope and the material. If there is not good contact, then water can seep below and encourage erosion.

7. **Facing.** The role of the slope facing can never be stressed enough. The facing when stabilizing mine waste plays an important role due to the granular nature and the erosion susceptibility of the dumped material. By using the appropriate materials such as a vegetative geofabric, a reinforcing geogrid, and grass seed, the slope can be protected from erosion and damage for the lifetime of the project.

Good relations between the contractor and engineering company aid in the quality and duration of the soil nailing project. In soil nailing, the nail length, inclination, nail spacing, and facing inclination play an important role in the slope stability. It is very important to convey these parameters to the contractor and the quality of their application to ensure a safe and stable product. If quality control on the site is not maintained, small failures, and unnecessary delays can be encountered. However, if constructed and monitored properly a stable and safe slope can be constructed that will last.

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Appendices

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Appendix 1

Hand Calculations

A1.1.1 German Method

German Method Hand Calculations

Material Properties

$\gamma = 106\ lb/ft^3$ $c = 150\ lb/ft^2$ $\phi = 30°$ $S_{hd} = 5$ ft (design horizontal spacing)

$T^{\#}_{mk} = 0.72\ kips/ft^2 = 1.51\ kips/ft$ (Ultimate Friction Force from nail pullout test)

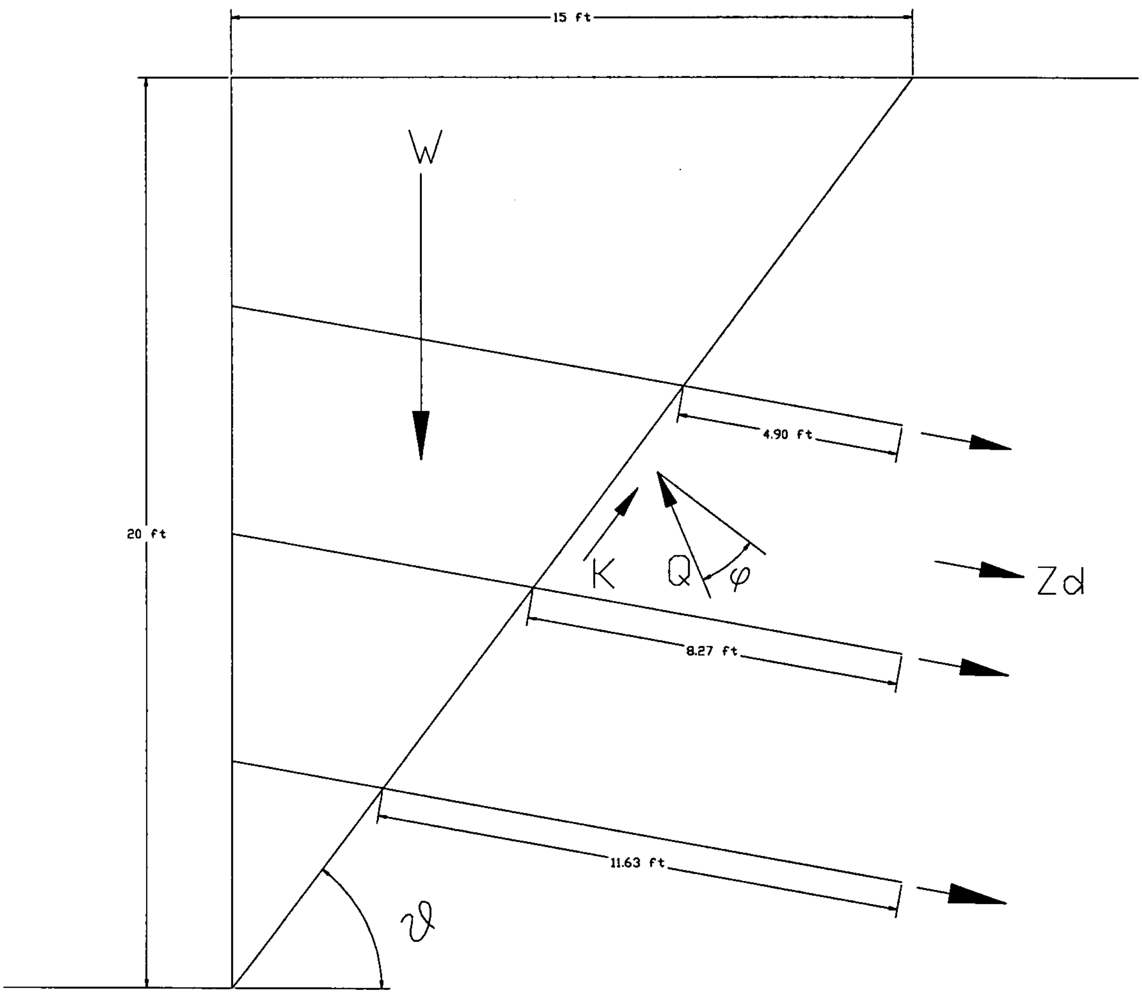


Figure A5-1 Failure Surface from SNAIL

To determine the Factor of safety for the nailed slope, the forces that can be calculated are determined and plotted on a Force Polygon as shown in Figure A5-2

Forces $$W = \frac{1}{2} h \times l \times \gamma = \frac{1}{2}(15ft)(20ft)(106\tfrac{lb}{ft^3})\left(\frac{1kip}{1000lb}\right) = 15.9kip/ft$$

$$K = cL = 150\frac{lb}{ft^2}(25ft)\left(\frac{1kip}{1000lb}\right) = 3.75kip/ft$$

W = force from weight of soil mass

K = cohesive force

c = cohesion

L = length of failure surface

From Figure A5-2 a value for Z_d can be determined $$Z_d = \frac{T_{m,d}}{S_h}\sum l_i$$

Z_d = 3.6 kip/ft

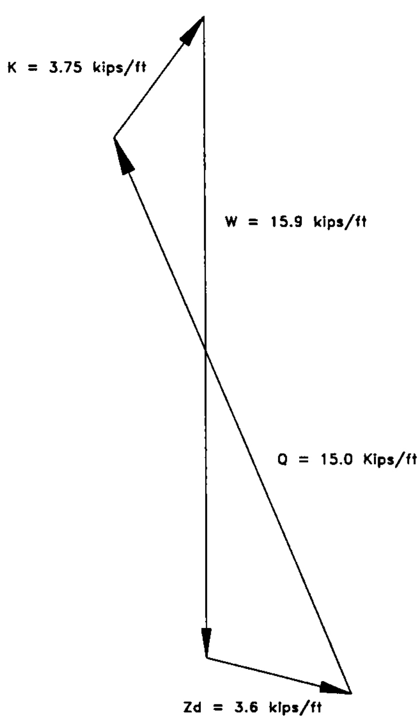


Figure A5-2 Force Polygon

l_i = length of reinforcing beyond failure plain, Figure A5-1

l_i = 4.9 ft+8.27 ft+11.63 ft = 24.8 ft

Solve for $T_{m,d}$, and let S_h = 1.0 ft (= unit width)

$$T_{m,d} = \frac{Z_d}{\sum l_i} = \frac{3.6\frac{kip}{ft}}{24.8ft} = 0.14kip/ft/ft$$

The required spacing, S_{hr} can be calculated as follows:

$$S_{hr} = \frac{T^{\#}_{m,k}(kip/ft)}{T_{m,k,\max}} = \frac{1.51(kip/ft)}{0.14(kip/ft/ft)} = 10.8ft$$

Then solve for FS $$FS = \frac{S_{hr}}{S_{hd}} = \frac{10.8ft}{5.0ft} = 2.2$$

A1.1.2 French Method

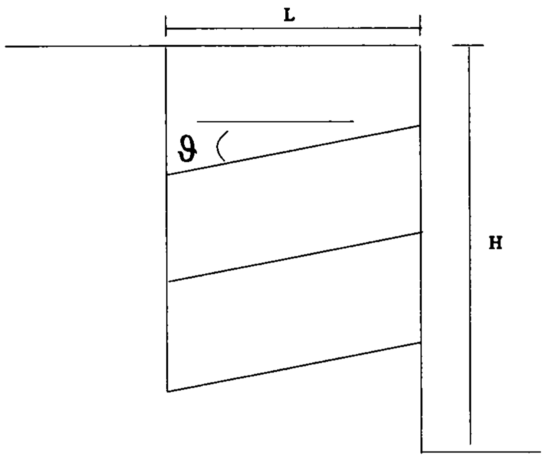
French Method Hand Calculations

Known:

H = 20 ft γ = 106 lb/ft^3 c = 150 lb/ft^2 σ = 5 psi

$S_h = S_v$ = 5 ft $\theta = 20°$ $\phi = 30°$ l = 15 ft

T_{lk} = 1508 lb/ft = limit shear force from pullout test



$$N = \frac{c}{\gamma H} = \frac{150\frac{lb}{ft^2}}{106\frac{lb}{ft^3}(20ft)} = 0.07$$

$$\tan\phi = \tan 30° = 0.58$$

$$L = 15\cos 20° = 14.1ft$$

$$\frac{L}{H} = \frac{14.1ft}{20ft} = 0.7$$

Must interpolate since charts do not account for L / H = 0.7

Since values known for T_{Lk}, must find what the nail density, d plots as on Figure 1, pg 160 (Schlosser 1990). Then plot as if TL unknown, and compare to get FS.

Interpolate between L/H = 0.8 & 0.6

$$d = \frac{T_L}{\gamma S_h S_v L} = \frac{12064lb}{106\frac{lb}{ft^3}(5ft)(5ft)(14.1ft)} = 0.32$$

From Figure 1	L/H = 0.8	L/H = 0.6	L/H = 0.7
	d = 0.25	d = 0.17	$d_{0.7}$ = 0.21

$$T_{LR} = d\gamma S_h S_v L = 0.21(106\tfrac{lb}{ft^3})(5ft)(5ft)(14.1ft) = 7846lb$$

$$FS = \frac{T_{Lk}}{T_{LR}} = \frac{12064lb}{7846lb} = 1.54$$

A1.1.3 Kinematic Method

SOIL NAILED WALLS AND SLOPES - COHESIVE SOILS

by ECD 9·

Ref: Elais, V. and Juran, I (1991) "Soil Nailing for Stabilization of Highway Slopes and Excavations" FHWA REport FHWA-RD-89-198, 221 pp.

Units:

$psf \equiv \frac{lbf}{ft^2}$ $psi \equiv \frac{lbf}{in^2}$ $pcf \equiv \frac{lbf}{ft^3}$ $ksi \equiv \frac{kip}{in^2}$ $tonm \equiv 1000 \cdot kg$ $kip \equiv 1000 \cdot lbf$

Kinematical Design Method

Elais, V. and Juran pp 113

SUMMARY OF RESULTS

$L_{nail} = 15.074 \cdot ft$ If TENSION = 0: OK

$TENSION_{flex} = 0$ if TENSION = 1: Tensile capacity too low

$TENSION_{rigid} = 0$ If BENDING = 0: OK

$BENDING = 0$ if BENDING = 1: Bending capacity too low

SLOPE PROPERTIES / GEOMETRY

$H \equiv 20 \cdot ft$ $\beta \equiv 90 \cdot deg$ Inclination of slope

$\gamma \equiv 106 \cdot pcf$ $\beta_{over} \equiv 0 \cdot deg$ Inclination of slope above nailed section

Desired Factors of Safety:

$FS_{pullout} \equiv 1.35$ FS w/r to pullout of the nail

$SF_m \equiv 1$ FS w/r to plastic bending of the nail

PRELIMINARY CHECK OF SOIL PROPERTIES

Atterberg limits

$w := 33$ Natural water content

$LL := 50$

$PL := 25$ $I_c := \frac{LL - w}{PL}$ $I_c = 0.68$

LL > 50 & PI > 20 suggest susceptibility to creep. Soils with a consistency index, Ic < 0.9 should not be used for permanent nailed structures due to creep considerations.

Strength properties

$\phi \equiv 30 \cdot deg$

$c \equiv 150 \cdot psf$

Nails probably not appropriate if unconfined strength < 100 psf

see pp 33 for evaluation of critical creep load Tc

Expect horizontal movements up to 1.5H/1000

NAIL PROPERTIES

$S_v \equiv 5 \cdot ft$ Vertical spacing

$S_h \equiv 5 \cdot ft$ Horizontal spacing

Steel:

$D_b \equiv 1 \cdot in$ Db = diameter of nail $r_b \equiv \frac{D_b}{2}$ $E_s \equiv 29000 \cdot ksi$ $I_s \equiv \pi \cdot \frac{r_b{}^4}{4}$ $I_s = 0.049 \cdot in^4$

$f_y \equiv 60 \cdot ksi$

Grout:

$D_g \equiv 8 \cdot in$ Dg = diameter of grouted hole $f_{c\_grout} \equiv 3000 \cdot psi$ $A_s \equiv \frac{\pi \cdot D_b{}^2}{4}$ $A_s = 0.785 \cdot in^2$

Soil/Grout Bond:

Fl = Ultimate or limit pullout resistance, or friction limit (must be confirmed by load tests)

$F_l \equiv 720 \cdot psf$ $F_l = 5 \cdot psi$

Estimated values for soil in Table 4, pp 31

Augered soft clay	F1 = 400 - 600 psf
stiff clay	F1 = 800 - 1200 psf
clayey silt (calcareous)	F1 = 800 - 2000 psf
sandy clay	F1 = 4000 - 6000 psf

Soil nail test data $P_{test} := 12000 \cdot lbf$ $l_{a\_test} := 8 \cdot ft$

P = ultimate nail capacity
A = perimeter area of grouted hole
la = length of grouted nail
Ftest = calculated pullout resistance

$$F_{test} := \frac{P_{test}}{D_g \cdot \pi \cdot l_{a\_test}} \qquad F_{test} = 716.197 \cdot psf \qquad F_{test} = 4.974 \cdot psi$$

KINEMATICAL PARAMETERS

Stability Number: $N_s \equiv \frac{c}{\gamma H}$ $\qquad N_s = 0.071 \qquad c = 150 \cdot psf \qquad c = 0.732 \cdot \frac{tonm}{m^2} \cdot (g)$

Horizontal Subgrade Reaction: (from Figure 16, pp 61) $\qquad$ convert shear strength parameters into SI units

INPUT (from Figure 16, pp 61): $k_h \equiv 3000 \cdot \frac{tonm}{m^3} \cdot g \qquad k_h = 108.384 \cdot \frac{lbf}{in^3}$

Transfer Length: $L_0 \equiv \left(\frac{4 \cdot E_s \cdot I_s}{k_h \cdot D_b} \right)^{0.25} \qquad L_0 = 15.14 \cdot in$

Bending Stiffness Parameter, N

$$N \equiv \frac{k_h \cdot D_b \cdot L_0^{\,2}}{\gamma H \cdot S_h \cdot S_v} \qquad N = 0.469$$

If N = 0, nail is perfectly flexible & carries only tension
If N < 1, nail is relatively flexible
N usually between 0.1 and 1.5
Design charts provided for N = 0, N = 0.67, and N = 1.33
Interpolate.

INPUT Select dimensionless parameters from Figure 37 through 67 (no groundwater)

For N = 0:	For N = 0.67:	For N = 1.33:
$TN_1 \equiv 0.15$	$TN_2 \equiv 0.125$	$TN_3 \equiv 0.11$
$TS_1 \equiv 0$	$TS_2 \equiv 0.08$	$TS_3 \equiv 0.115$
$SH_1 \equiv 0.44$	$SH_2 \equiv 0.45$	$SH_3 \equiv 0.45$

Interpolate a $\qquad$ Interpolate b

$$TN_{sa} \equiv \frac{0 - N}{0.67} \cdot (TN_1 - TN_2) + TN_1 \qquad TN_{sb} \equiv \frac{0.67 - N}{0.67} \cdot (TN_2 - TN_3) + TN_2$$

$$TS_{sa} \equiv \frac{0 - N}{0.67} \cdot (TS_1 - TS_2) + TS_1 \qquad TS_{sb} \equiv \frac{0.67 - N}{0.67} \cdot (TS_2 - TS_3) + TS_2$$

$$SH_{sa} \equiv \frac{0 - N}{0.67} \cdot (SH_1 - SH_2) + SH_1 \qquad SH_{sb} \equiv \frac{0.67 - N}{0.67} \cdot (SH_2 - SH_3) + SH_2$$

INPUT Define SHs, TSs, & TNs based on approriate interpolation (a or b)

$TN_{sa} = 0.133$	$TN_{sb} = 0.13$	Define:	$TN_s \equiv 0.163$
$TS_{sa} = 0.056$	$TS_{sb} = 0.069$		$TS_s \equiv 0.063$
$SH_{sa} = 0.447$	$SH_{sb} = 0.45$		$SH_s \equiv .367$

RESULTS: NAIL PULLOUT CRITERION

Equation 30: $\mu \equiv \dfrac{F_l \cdot D_g}{\gamma S_h \cdot S_v}$ $\mu = 0.181$ $F_l = 5 \cdot psi$

Equation 29: $LoverH \equiv SH_s + FS_{pullout} \cdot \dfrac{TN_s}{\pi \cdot \mu}$ $LoverH = 0.754$ $L_{nail} \equiv LoverH \cdot H$

$L_{nail} = 15.074 \cdot ft$

RESULTS: NAIL FAILURE CRITERION

Flexible Nails (N = 0) TNactual > TNs

$$TN_{actual} \equiv \frac{f_y \cdot A_s}{\gamma H \cdot S_v \cdot S_h} \qquad TN_{actual} = 0.889$$

$$TENSION_{flex} \equiv if\left(TN_{actual} > TN_s, 0, 1\right) \qquad TENSION_{flex} = 0$$

If TENSION = 0: OK
if TENSION = 1: Tensile capacity too low

Rigid Nails (N > 0) Mp > TNactual >Keq

$$K_{eq} \equiv \left(TN_s^{\,2} + 4 \cdot TS_s^{\,2}\right)^{0.5} \qquad K_{eq} = 0.206 \qquad TENSION_{rigid} = 0$$

If TENSION = 0: OK
if TENSION = 1: Tensile capacity too low

$$TENSION_{rigid} \equiv if\left(TN_{actual} > K_{eq}, 0, 1\right)$$

Breakage/Excessive Bending Failure Criterion

$$M_{plastic} \equiv f_y \cdot 0.4244 \cdot \pi \cdot \left(\frac{D_b}{2}\right)^3 + \frac{0.4244}{2} \cdot \left(\frac{D_g^{\,3}}{2} - \frac{D_b^{\,3}}{2}\right) \cdot f_{c\_grout} \qquad M_{plastic} = 14.388 \cdot ft \cdot kip$$

$$TS_{actual} \equiv \frac{\dfrac{M_{plastic}}{L_0}}{\gamma H \cdot S_v \cdot S_h} \qquad TS_{actual} = 0.215$$

$$BENDING \equiv if\left(TS_{actual} > 0.32 \cdot SF_m \cdot TS_s, 0, 1\right) \qquad BENDING = 0$$

If BENDING = 0: OK
if BENDING = 1: Bending capacity too low

A1.2 Computer Input / Output Files for Hand Calculations

A1.2.1 Edina

Analysis of nailed wall DDD

Calculation according to the method of Gaessler and Gudehus.

The compressive strength of natural soils usually meets the requirements of modern excavation techniques while their tensile and shear strengths remain unsatisfactory.

It is the principle of Soil Nailing to reinforce in-situ soil with steel rods, thus achieving a composite body whose tensile and shear strengths exceed those of unreinforced soil. Soil Nailing is supposed to encourage the surrounding soil to support itself, so that costly retaining structures and foundations can be avoided.

Soil nailing is applied in the following fields:
a) Containing of landslides in slopes
b) Reinforcing the surface of a slope to prevent clod-like movements
c) Retaining steep to vertical cuts

Conventional excavation methods use concrete walls or steel pilings to retain near vertical cuts but make no use of the soil`s ability to support its own weight. Soil Nailing reinforces the soil by placing steel rods in boreholes, which are consequently encased in cement. The surface of a cut is sealed with shotcrete, thus preventing erosion. A monolithic body is formed; horizontal forces resulting from earth pressure and surface load can be absorbed. Excavation is executed in layers of 1 to 1.5 meters (3 to 5 feet) each, with nailing following as fast as possible. This procedure represents the main difference between Soil Nailing and the method of "Reinforced Earth", where a monolithic body is formed by placing subsequent layers of earth behind prefabricated elements.

The calculations of EDINA are based on studies by Stocker, Gaessler and Gudehus of the Technical University of Karlsruhe, who, in cooperation with the contractor Karl Bauer KG, Schrobenhausen, conducted model and in-situ tests to establish criteria for the calculation and design of Soil-Nailing structures.

For adequate analysis of a nailed wall both internal and external safety factors have to be determined. External safety analysis can be performed with the active Coulomb pressure, the earth pressure at rest or a percentage of either. Cohesion can be taken into account or neglected. Gaessler and Gudehus maintain, that a reduction in earth pressure by 30 to 40% of the active Coulomb value is indicated by experimental evidence. Earth pressure is calculated according to Coulomb`s theory.

The influence of the inclination of the terrain is taken into account for the uppermost layer of soil; the angle between the vertical rear section of the monolith and the earth pressure is equal to the internal angle of friction.

Internal safety is determined according to the kinematical method (Gudehus, 1972 ; Goldschneider and Gudehus, 1974). A system of slip surfaces is assumed. Its governing parameter, the angle of the main slip surface, is given a wide variety of values. Active (dead weight, surface load, earth pressure) and retaining (cohesion, soil reaction) forces are analyzed in a force polygon, resulting in a required nail force for each slip surface. The quotient of the actual nail force and the required nail force is the internal safety factor of the slip surface. The slip surface with the minimal safety factor is then chosen to represent the overall stability of the wall. Please be aware of the limits of this theory! Wall inclinations of be low 70 degrees against the horizontal and horizontal nail-distances of more than 2m (6 ft.) are beyond the scope of both the theory and the algorithm.

LAW ENVIRONMENTAL, INC.	Project: DDD
114 Townpark Drive	Engineer: clyde
Kennesaw, Georgia 30144-5599	Date: 11-18-1996
Tel: (404) 499-6800	Time: 11:34:19
Nailed wall analysis	Page: 3

1. SPECIFIC SOIL DATA

Number of soil layers: 1

Layer 1
Designation: dff
Upper edge of layer: 0.00 FT below surface
Angle of friction phi: 30.00 degrees
Unit weight: 106.00 LBS/FT^3
Cohesion: 150.00 LBS/FT^2

2. DISTRIBUTION OF NAILS

Height of wall: 20.00 FT
Inclination of wall: 0.00 degrees
Inclination of terrain: 0.00 degrees
Horizontal distance of nails: 5.00 FT
Inclination of nails: 10.00 degrees
Number of nails: 3

Nail 1 :
Depth below ground line: 5.00 FT
Length of nail: 15.00 FT
Friction force per FT: 1500.00 LBS

Nail 2 :
Depth below ground line: 10.00 FT
Length of nail: 15.00 FT
Friction force per FT: 1500.00 LBS

Nail 3 :
Depth below ground line: 15.00 FT
Length of nail: 15.00 FT
Friction force per FT: 1500.00 LBS

3. SURFACE LOAD

Distributed load 0.00 LBS/FT^2

4. STABILITY

4.1 Internal safety factor

A Gaessler slip mechanism with a characteristic angle of 55.00 degrees is formed

The slip mechanism can be described by the following values: (Data per FT of wall)

Wedge 1:14842.24 LBS   Load on wedge 1: 0.00 LBS
Wedge 2: 0.00 LBS   Load on wedge 2: 0.00 LBS

Cohesion between wedge 1 and wedge 2: 0.00 LBS
Cohesion between wedge 1 and soil: 3662.15 LBS
Cohesion between wedge 2 and soil: 0.00 LBS

Action of nails:

Nail	Depth(FT)	Act. Length(FT)	Force(LBS)
1	5.00	5.51	8262.25
2	10.00	8.67	13008.17
3	15.00	11.84	17754.09

Total required nail force: 3851.36 LBS/FT
Total actual nail force: 7804.90 LBS/FT

The inner safety factor can thus be determined as:

F.S.= 2.03

4.2 External safety factor

Safety against sliding on a horizontal surface 8.75

Safety against tilting around the toe 42.44

LAW ENVIRONMENTAL, INC.

A1.2.2 Nail

LAW ENVIRONMENTAL, INC.	Project: THESIS
114 Townpark Drive	Engineer: Ray
Kennesaw, Georgia 30144-5599	Date:10-23-1996
Tel: (404) 499-6800	Time:12:46:21
Nailed wall analysis - Modified Shen	Page: 1

1. WALL GEOMETRY:

```
ELEVATION AT TOP OF WALL (FEET) =       20.00
ELEVATION AT BOTTOM OF WALL (FEET) =     0.00
HEIGHT OF WALL IN FEET =                20.00
SURCHARGE LOADING IN PSF =               0.00
```

2. SOIL PROFILE:

LAYER NO.	PHI DEGREE	UNIT WEIGHT LBS/FT^3	K	COHEION LBS/FT^2	DEPTH FEET
1	30.00	106.00	0.50	150.00	20.00

3. NAIL GEOMETRY:

NAIL ROW #	NAIL DEPTH (FT.)	NAIL SLOPE (DEGREE)	NAIL SPACING (FT.)	NAIL LENGTH (FT.)	NAIL DIA (IN.)	STEEL AREA (IN^2)	STEEL FY (KSI)	ALLOWABLE NAIL FORCE (LBS/FT)
1	5.00	10.00	5.00	15.00	8.00	0.79	60.0	1508.0
2	10.00	10.00	5.00	15.00	8.00	0.79	60.0	1508.0

3 15.00 10.00 5.00 15.00 8.00 0.79
60.0 1508.0

LAW ENVIRONMENTAL, INC.	Project: THESIS
114 Townpark Drive	Engineer: Ray
Kennesaw, Georgia 30144-5599	Date:10-23-1996
Tel: (404) 499-6800	Time:12:46:22
Nailed wall analysis - Modified Shen	Page: 2

4. CRITICAL FAILURE SURFACE:

TRIAL SURFACE	X (FT.)
92	18.60

TIE #	DEVELOPEMENT LENGTH (FT.)	CALCULATED BOND F.(LB.)	INPUT BOND F.(LB.)	TENSILE STRENGTH(LB.)	DESIGN FORCE(LB.)
1	0.12	134.50	179.26	47100.00	179.26
2	3.10	5095.68	4680.86	47100.00	5095.68
3	6.98	14959.59	10524.31	47100.00	14959.59

TOTAL PULLOUT RESISTANCE (LB.)= 20234.53
TOTAL PULLOUT RESISTANCE PER FOOT OF WALL (LB./FT.)= 4046.91

5. CALCAULATION DETAILS:

W1F= 24639.60 LB./FT.
W2F= 1648.40 LB./FT.
N1F= 1524.23 LB./FT.
S1F= 880.02 LB./FT.
N2F= 17143.00 LB./FT.
S2F= 16521.46 LB./FT.
N3F= 2519.01 LB./FT.
S3F= 1539.72 LB./FT.

TOTAL DRIVING FORCE = 16521.46 + 1539.72 = 18061.18 LB./FT.

RESISTIVE FORCES:

FROM COHESION = 4287.33 LB./FT.
FROM FRICTION = 11351.87 LB./FT.
FROM NAIL ANCHORS = 4383.52 LB./FT.

TOTAL RESISTIVE FORCES = 20022.73 LB./FT.

6. FACTOR OF SAFETY:

FACTOR OF SAFETY = TOTAL RESISTIVE FORCES / TOTAL DRIVING FORCES

FACTOR OF SAFETY = 20022.73 / 18061.18 = 1.11

A1.2.3 Snail Ver. 2.11

```
"TITLE"
"High Wall 20 ft"
"FLAGS"
0,0,0,0,0
"GEOMETRY"
20,10,0,60,0,0,0,0,0
"SURCHARGE"
0,0,0,0,0,0,0,0
"REINFORCEMENTLAYERS"
3,5,20,60,8
"SOIL"
1
118,30,180,5
"EARTHQUAKE & Horizontal FORCE."
0,0,0
"SEARCHLIMIT"
0,150
"NODELIMITS"
1,0,10,0
"REINFORCEMENT"
15,10,5,5,1
```

Appendix 2

Section Soil Structures

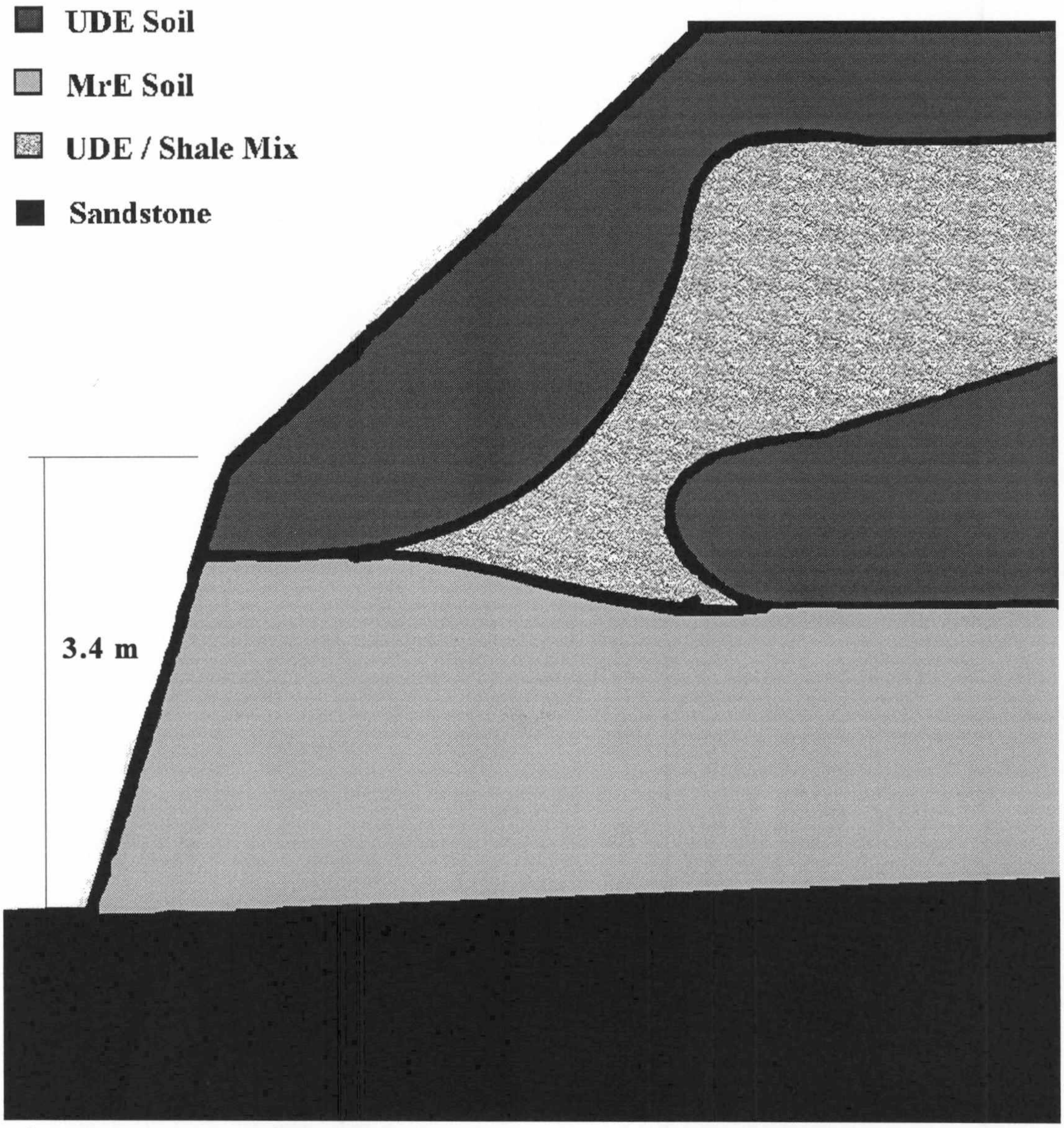


Figure A3-1 Soil Structure Section 2

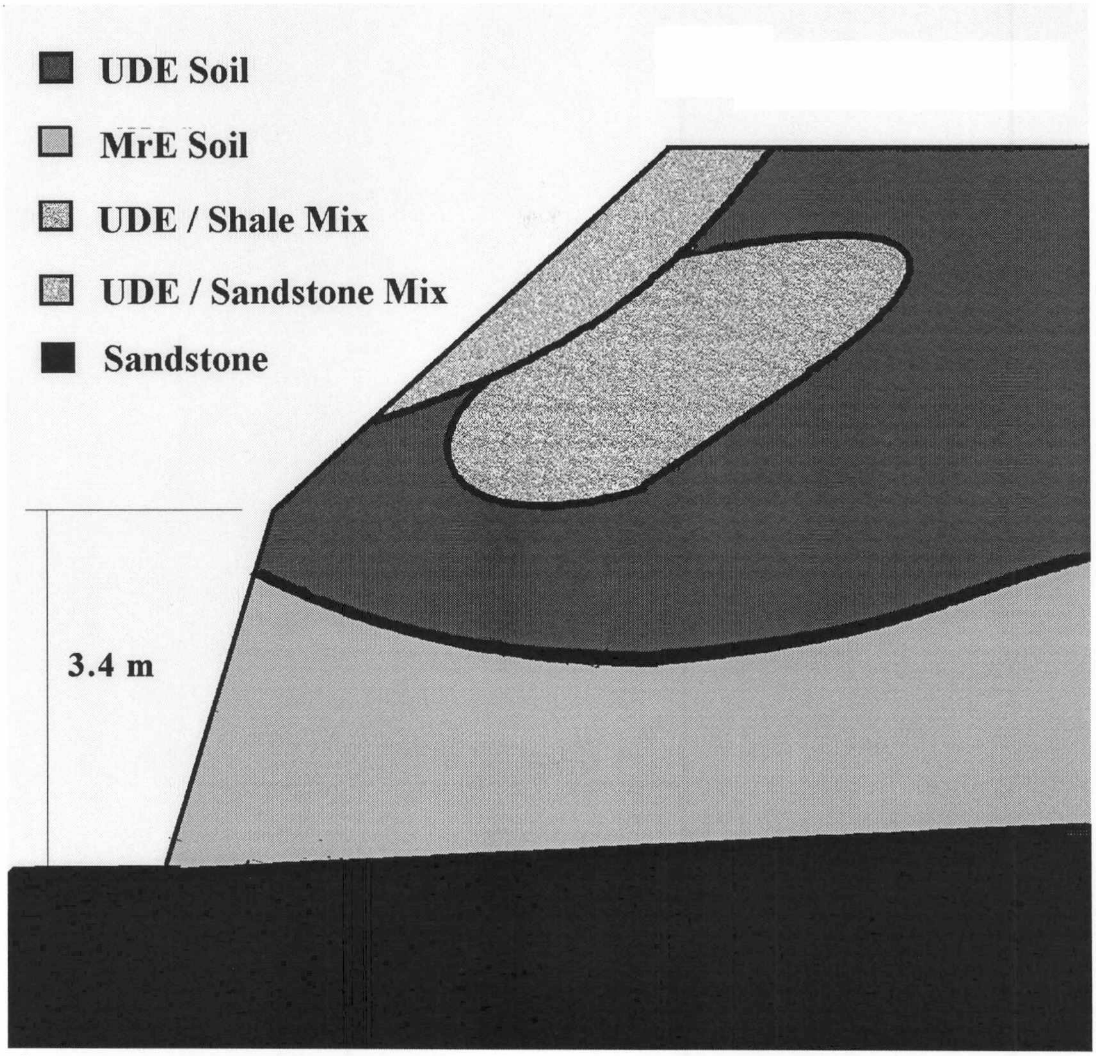


Figure A3-2 Soil Structure Section 3

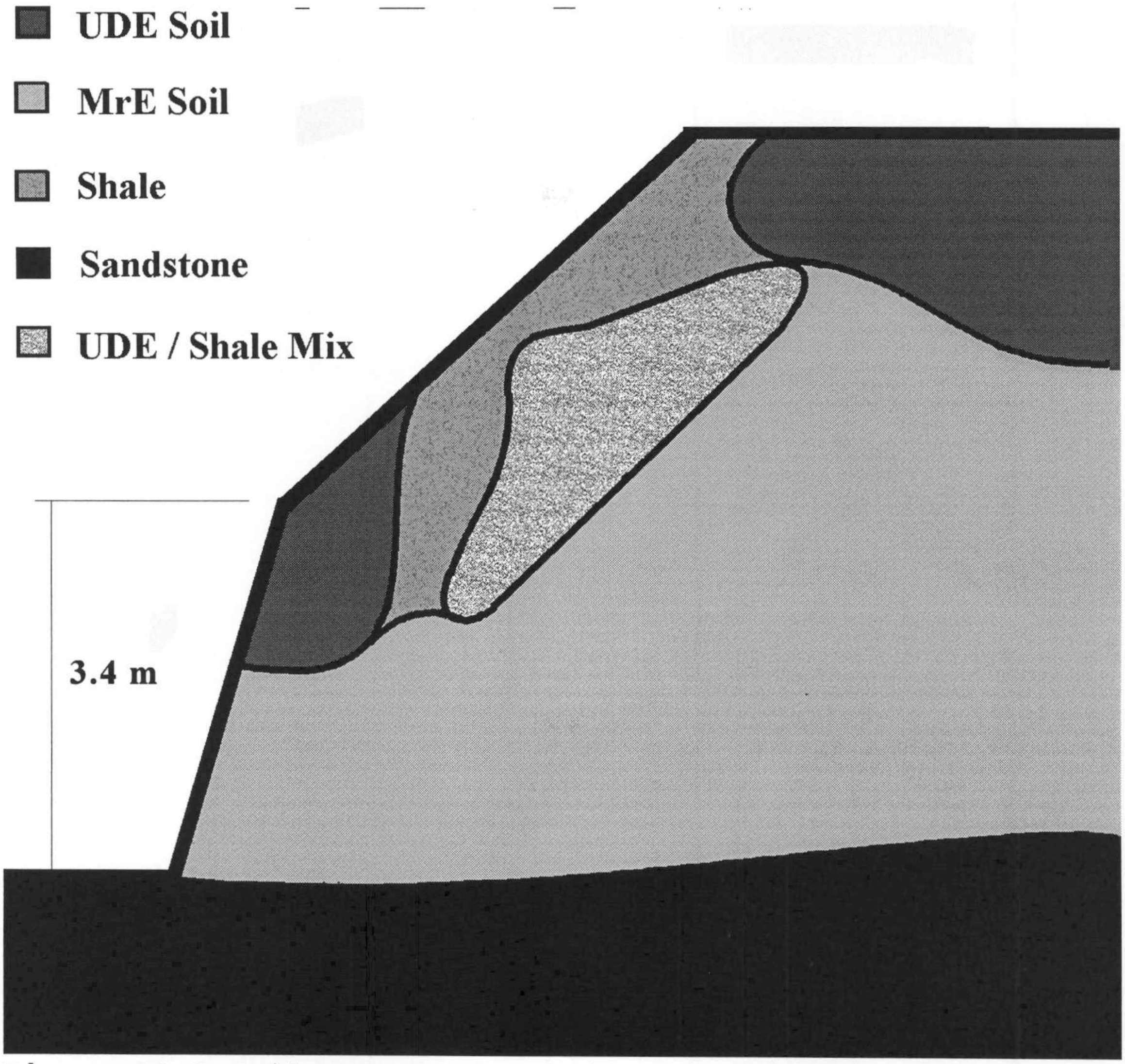
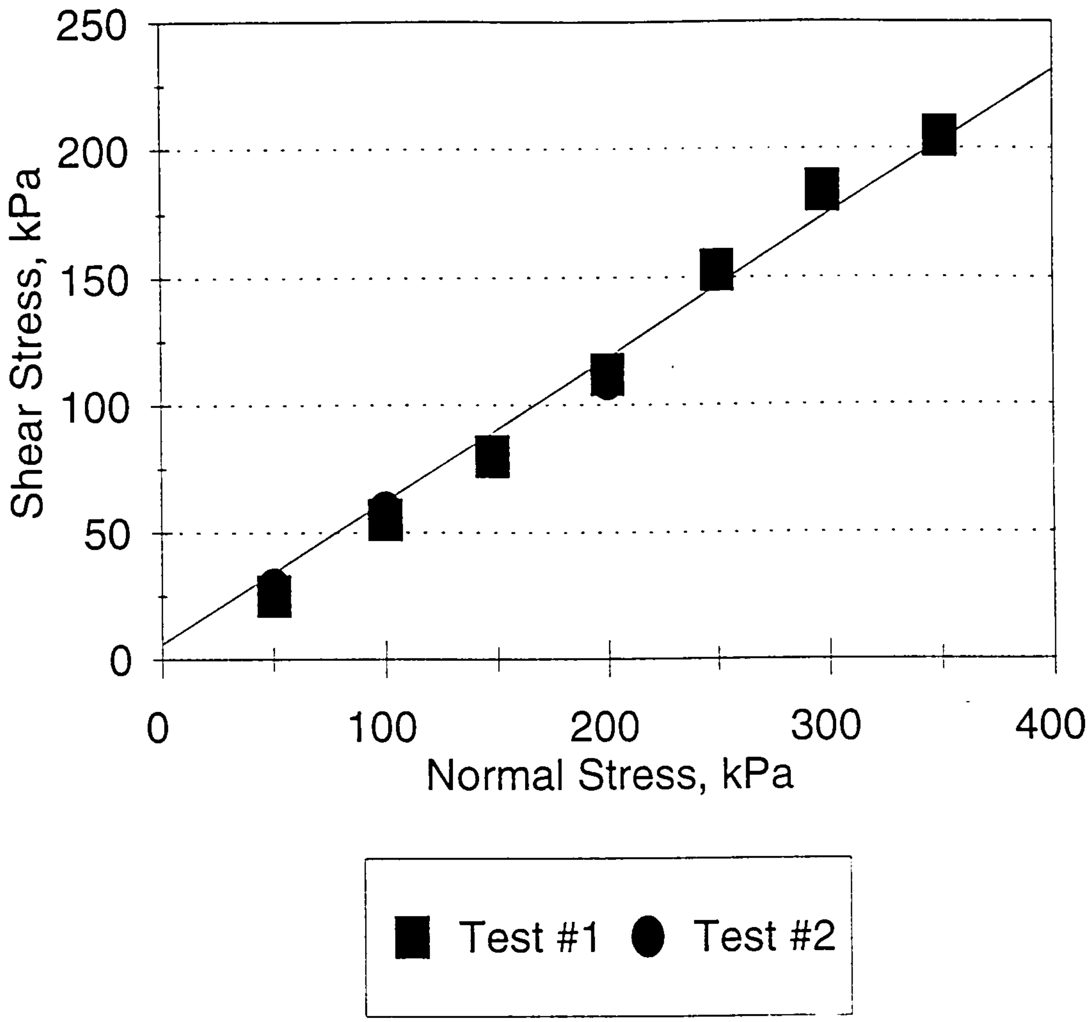


Figure A3-3 Soil Structure Section 4

Appendix 3

Material Properties

A3.1 Borehole Shear Tests



Two borehole shear tests were conducted. One on 7/6/95 and another on 8/27/95. From the two tests a best fit line was drawn through the data points, and the friction angle of the soil calculated.

$$\phi = 30°$$

A3.2 Sieve Results

Published Soil Properties (Moneymaker 1978)

Soil Type	Depth inches	Fragments > 3 in	USCS	Percent Passing Sieve #				LL %	PI %
				4	10	40	200		
MrE	0-5	0-10	ML, CL, SM, SC	75-100	70-95	50-90	30-80	20-35	2-10
	5-26	0-15	GM, SM, ML, CL	70-90	55-85	50-80	40-75	20-35	2-10
	26-36	0-15	GM, SM, ML, GM-GC	20-80	10-65	10-65	10-60	20-35	2-10
JSE	0-10	5-20	SM, SC, ML, CL	75-90	70-90	50-80	30-65	20-35	2-10
	10-58	5-20	SM, SC, ML, CL	75-90	70-90	50-80	30-70	15-35	2-15
	58-66	5-25	GM, SM, ML, GM-GC	55-75	50-75	35-70	20-60	20-35	2-10

Laboratory Tests from Auger Cuttings

Soil Type	Depth Feet	USCS	Percent Passing Sieve #			
			4	10	40	200
UDE Boring 1	5-15	ML	62.4	49.8	34.4	27.0
	15-25	ML	84.0	70.8	49.4	36.2
UDE Boring 2	5-15	ML	59.2	48.1	34.0	28
MrE Boring on Slope	0 - 2	ML	51.0	44.5	35.8	32.3

A3.3 Grout Tests

Dia.:5.98		Name:RT 3/13/96		Dia.:5.99		Name:RT 4/18/96		Dia.:6.00	
Max. Stress:5411.91		Max.Force:162,000		Max. Stress:5748.71		Max.Force:142,000		Max. Stress:5022.22	
Strain	Stress(psi)	Dis.(.0005")	Force(1000lb	Strain	Stress(psi)	Dis.(.0005")	Force(1000lb	Strain	Stress(psi)
0	0	0	0	0	0	0	0	0	0
0	178.023256	0.6	5	30	177.42935	0.1	5	5	176.838412
0	356.046512	1.25	10	62.5	354.858701	0.15	10	7.5	353.676824
0	534.069767	1.9	15	95	532.288051		15	0	530.515236
20	712.093023	2.45	20	122.5	709.717402	0.5	20	25	707.353649
40	890.116279	3.1	25	155	887.146752	1	25	50	884.192061
80	1068.13953	3.6	30	180	1064.5761	1.5	30	75	1061.03047
110	1246.16279	4.25	35	212.5	1242.00545	2	35	100	1237.86888
180	1424.18605	5	40	250	1419.4348	2.5	40	125	1414.7073
225	1602.2093	5.6	45	280	1596.86415	2.6	45	130	1591.54571
255	1780.23256	6.3	50	315	1774.2935	3.5	50	175	1768.38412
285	1958.25581	6.95	55	347.5	1951.72285	4.2	55	210	1945.22253
315	2136.27907	7.55	60	377.5	2129.15221	4.7	60	235	2122.06095
345	2314.30233	8.3	65	415	2306.58156	5.25	65	262.5	2298.89936
380	2492.32558	9	70	450	2484.01091	5.7	70	285	2475.73777
0	2670.34884	9.1	75	455	2661.44026	5.75	75	287.5	2652.57618
0	2848.37209		80	0	2838.86961	7.1	80	355	2829.41459
400	3026.39535	10	85	500	3016.29896	7.2	85	360	3006.25301
430	3204.4186	10.6	90	530	3193.72831	8	90	400	3183.09142
470	3382.44186	11.5	95	575	3371.15766	9.2	95	460	3359.92983
505	3560.46512	12.4	100	620	3548.58701	10	100	500	3536.76824
540	3738.48837	13.15	105	657.5	3726.01636	10.5	105	525	3713.60665
580	3916.51163	14.1	110	705	3903.44571	10.7	110	535	3890.44507
0	5411.90698		162		5748.71095		142		5022.2109

Dia.:5.98		Name:RT 3/13/96		Dia.:5.99		Name:RT 4/1 6		Dia.:6.00	
Max. Stress:5411.91		Max.Force:162,000		Max. Stress:5748.71		Max.Force:142,000		Max. Stress:5022.22	
Strain	Stress(psi)	Dis.(.0005")	Force(1000lb	Strain	Stress(psi)	Dis.(.0005")	Force(1000lb	Strain	Stress(psi)
0	0	0	0	0	0	0	0	0	0
0	178.023256	0.6	5	30	177.42935	0.1	5	5	176.838412
0	356.046512	1.25	10	62.5	354.858701	0.15	10	7.5	353.676824
0	534.069767	1.9	15	95	532.288051		15	0	530.515236
20	712.093023	2.45	20	122.5	709.717402	0.5	20	25	707.353649
40	890.116279	3.1	25	155	887.146752	1	25	50	884.192061
80	1068.13953	3.6	30	180	1064.5761	1.5	30	75	1061.03047
110	1246.16279	4.25	35	212.5	1242.00545	2	35	100	1237.86888
180	1424.18605	5	40	250	1419.4348	2.5	40	125	1414.7073
225	1602.2093	5.6	45	280	1596.86415	2.6	45	130	1591.54571
255	1780.23256	6.3	50	315	1774.2935	3.5	50	175	1768.38412
285	1958.25581	6.95	55	347.5	1951.72285	4.2	55	210	1945.22253
315	2136.27907	7.55	60	377.5	2129.15221	4.7	60	235	2122.06095
345	2314.30233	8.3	65	415	2306.58156	5.25	65	262.5	2298.89936
380	2492.32558	9	70	450	2484.01091	5.7	70	285	2475.73777
0	2670.34884	9.1	75	455	2661.44026	5.75	75	287.5	2652.57618
0	2848.37209		80	0	2838.86961	7.1	80	355	2829.41459
400	3026.39535	10	85	500	3016.29896	7.2	85	360	3006.25301
430	3204.4186	10.6	90	530	3193.72831	8	90	400	3183.09142
470	3382.44186	11.5	95	575	3371.15766	9.2	95	460	3359.92983
505	3560.46512	12.4	100	620	3548.58701	10	100	500	3536.76824
540	3738.48837	13.15	105	657.5	3726.01636	10.5	105	525	3713.60665
580	3916.51163	14.1	10	705	3903.44571	10.7	110	535	3890.44507
0	5411.90698		62		5748.71095		142		5022.2109

Name:UT33 Dia.:6.00
Max.Force:126,000 Max. Stress:4456.33

Dis.(.0005")	Force(1000lb	Strain	Stress(psi)
0	0	0	0
0.15	5	7.5	176.838412
0.25	10	12.5	353.676824
0.35	15	17.5	530.515236
0.45	20	22.5	707.353649
0.55	25	27.5	884.192061
0.65	30	32.5	1061.03047
0.75	35	37.5	1237.86888
0.9	40	45	1414.7073
1	45	50	1591.54571
1.1	50	55	1768.38412
1.25	55	62.5	1945.22253
1.6	60	80	2122.06095
2	65	100	2298.89936
2.3	70	115	2475.73777
2.75	75	137.5	2652.57618
3.2	80	160	2829.41459
3.7	85	185	3006.25301
5	90	250	3183.09142
5.1	95	255	3359.92983
5.25	100	262.5	3536.76824
5.8	105	290	3713.60665
6.5	110	325	3890.44507
	126		4456.32799

Name:RT 3/13/96 Dia.:5.98
Max.Force:65,000 Max. Stress:

Dis.(.0005")	Force(1000lb	Strain	Stress(psi)
0	0	0	0
0.8	5	40	178.023256
1	10	50	356.046512
1.25	15	62.5	534.069767
1.5	20	75	712.093023
1.75	25	87.5	890.116279
2.5	30	125	1068.13953
3.3	35	165	1246.16279
4.2	40	210	1424.18605
5.2	45	260	1602.2093
5.4	50	270	1780.23256
6.4	55	320	1958.25581
7.4	60	370	2136.27907
9.4	65	470	2314.30233

Name:RT 4/18/96
Max.Force:152,000

Dis.(.0005")	Force(1000lb
0	0
	5
	10
	15
0.4	20
0.8	25
1.6	30
2.2	35
3.6	40
4.5	45
5.1	50
5.7	55
6.3	60
6.9	65
7.6	70
	75
	80
8	85
8.6	90
9.4	95
10.1	100
10.8	105
11.6	110
	152

Name:RT 3/18/96 Dia.:6.00
Max.Force:157,000 Max. Stress:5552.73

Dis.(.0005")	Force(1000lb	Strain	Stress(psi)
0	0	0	0
	5	0	176.838412
	10	0	353.676824
0.4	15	20	530.515236
1.05	20	52.5	707.353649
1.75	25	87.5	884.192061
3.1	30	155	1061.03047
3.7	35	185	1237.86888
3.9	40	195	1414.7073
4.4	45	220	1591.54571
5.1	50	255	1768.38412
6.6	55	330	1945.22253
7.4	60	370	2122.06095
8.2	65	410	2298.89936
8.6	70	430	2475.73777
9.3	75	465	2652.57618
9.7	80	485	2829.41459
11.5	85	575	3006.25301
12.4	90	620	3183.09142
13.3	95	665	3359.92983
13.6	100	680	3536.76824
15	105	750	3713.60665
15.5	110	775	3890.44507
	157		5552.72614

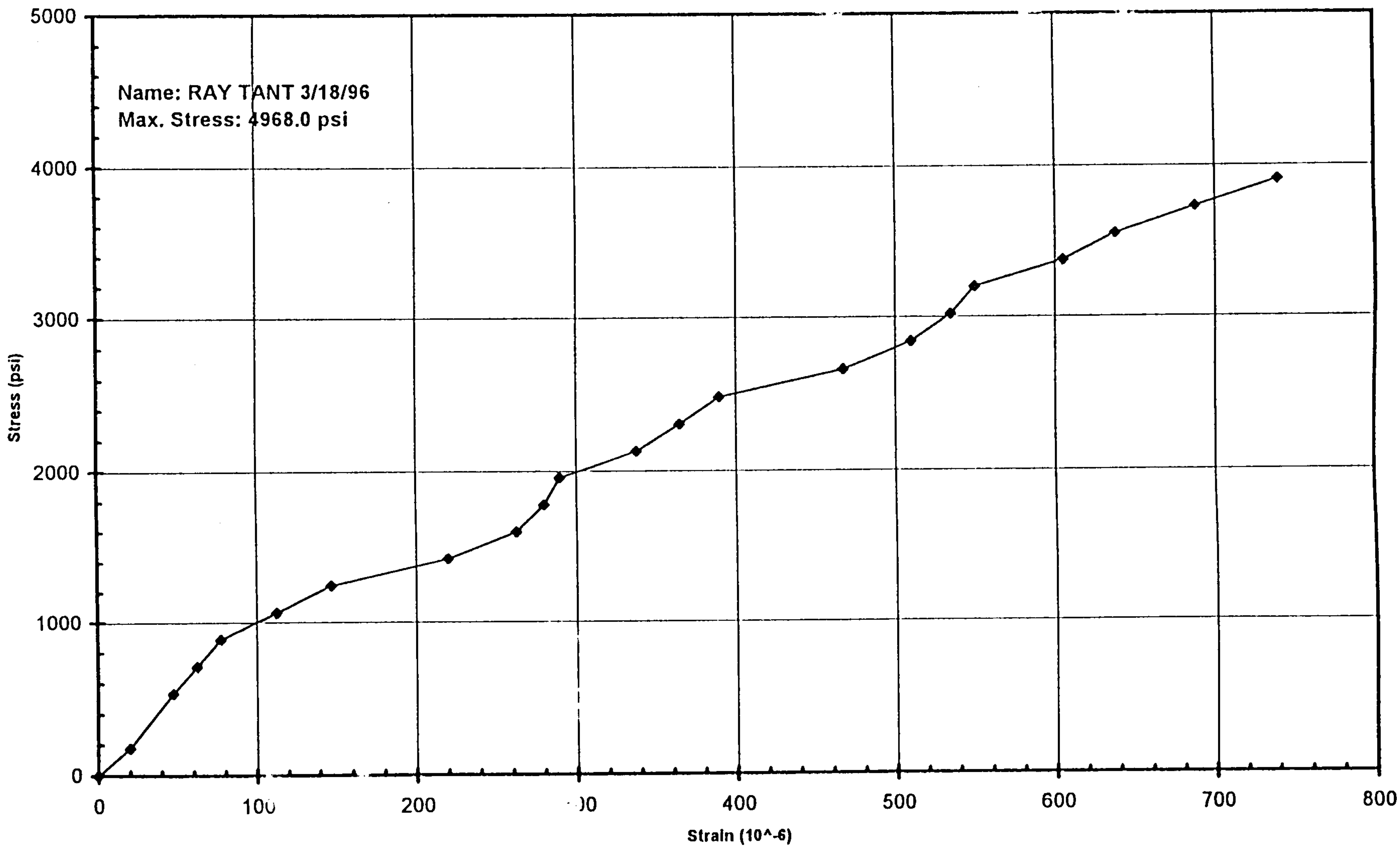
Name:RT 3/18/96 Dia.:5.99
Max.Force:140,000 Max. Stress:4968.02

Dis.(.0005")	Force(1000lb	Strain	Stress(psi)
0	0	0	0
0.4	5	20	177.42935
	10	0	354.858701
0.95	15	47.5	532.288051
1.25	20	62.5	709.717402
1.55	25	77.5	887.146752
2.25	30	112.5	1064.5761
2.95	35	147.5	1242.00545
4.4	40	220	1419.4348
5.25	45	262.5	1596.86415
5.6	50	280	1774.2935
5.8	55	290	1951.72285
6.75	60	337.5	2129.15221
7.3	65	365	2306.58156
7.8	70	390	2484.01091
9.35	75	467.5	2661.44026
10.2	80	510	2838.86961
10.7	85	535	3016.29896
11	90	550	3193.72831
12.1	95	605	3371.15766
12.75	100	637.5	3548.58701
13.75	105	687.5	3726.01636
14.8	110	740	3903.44571
	140		4968.02181

Sheet1

		5° Strain	40% Ut Stress	40 Ut. St Strain			ϕ Secant Modulus psi	E M psf
UT32	55	890.116	1602.209	120	712.093	65	1.0955E+07	1.5776E+09
UT13	55	1780.232	1958.2558	70	178.0238	15	1.1868E+07	1.7090E+09
UT23	57.5	707.35	2122.06	185	1414.71	127.5	1.1096E+07	1.5978E+09
UT33	50	1591.5457	2122.06	80	530.5143	30	1.7684E+07	2.5465E+09
RT 3/13/96-1	50	356.0465	1246.1629	165	890.1164	115	7.7401E+06	1.1146E+09
RT4/18/96-1	40	890.116	2136.279	315	1246.163	275	4.5315E+06	6.5254E+08
RT3/13/96-2	62.5	354.859	2306.581	415	1951.722	352.5	5.5368E+06	7.9730E+08
RT4/18/96-2	50	884.192	1945.222	210	1061.03	160	6.6314E+06	9.5493E+08
RT3/18/96-1	52.5	707.354	2298.9	410	1591.546	357.5	4.4519E+06	6.4107E+08
RT3/18/96-2	47.5	532.288	1951.723	290	1419.435	242.5	5.8533E+06	8.4288E+08
								5.0033E+09
								8.38E+08 — Average

CYLINDER MODULUS TEST R[illegible]ULT



A3.4 Equivalent Slope Data

Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Row 1	2	3	2	3	3	2	2	3	3	3	3	2	2	1	1	2	2	1	2	2	2	2	1
Row 2	2	3	4	3	4	3	2	1	2	2	3	3	2	4	4	3	1	1	1	2	2	2	2
Row 3	3	4	3	4	4	3	3	3	3	2	2	3	4	4	6	3	3	2	4	3	2	3	2
Row 4	2	4	4	3	4	4	3	4	5	2	2	3	4	5	5	5	5	3	4	5	3	2	3
Row 5	2	3	3	4	4	4	4	4	4	2	2	2	4	5	4	4	5	2	2	2	2	2	3

Nail Density
nails/ft^2

Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Row 1	0.025	0.038	0.025	0.038	0.038	0.025	0.025	0.038	0.038	0.038	0.038	0.025	0.025	0.013	0.013	0.025	0.025	0.013	0.025	0.025	0.025	0.025	0.013
Row 2	0.025	0.038	0.051	0.038	0.051	0.038	0.025	0.013	0.025	0.025	0.038	0.038	0.025	0.051	0.051	0.038	0.013	0.013	0.013	0.025	0.025	0.025	0.025
Row 3	0.038	0.051	0.038	0.051	0.051	0.038	0.038	0.038	0.038	0.025	0.025	0.038	0.051	0.051	0.076	0.038	0.038	0.025	0.051	0.038	0.025	0.038	0.025
Row 4	0.025	0.051	0.051	0.038	0.051	0.051	0.038	0.051	0.064	0.025	0.025	0.038	0.051	0.064	0.064	0.064	0.064	0.038	0.051	0.064	0.038	0.025	0.038
Row 5	0.025	0.038	0.038	0.051	0.051	0.051	0.051	0.051	0.051	0.025	0.025	0.025	0.051	0.064	0.051	0.051	0.064	0.025	0.025	0.025	0.025	0.025	0.038

Equivalent Spacings

	Section 1 (1 - 9)									Section 2 (9 - 16)								Section 3 (16 - 23)					
Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Row 1	6.26	5.12	6.26	5.12	5.12	6.26	6.26	5.12	5.12	5.60	5.60	6.86	6.86	9.71	9.71	6.86	6.26	8.86	6.26	6.26	6.26	6.26	8.86
Row 2	6.26	5.12	4.43	5.12	4.43	5.12	6.26	8.86	6.26	6.86	5.60	5.60	6.86	4.85	4.85	5.60	8.86	8.86	8.86	6.26	6.26	6.26	6.26
Row 3	5.12	4.43	5.12	4.43	4.43	5.12	5.12	5.12	5.12	6.86	6.86	5.60	4.85	4.85	3.96	5.60	5.12	6.26	4.43	5.12	6.26	5.12	6.26
Row 4	6.26	4.43	4.43	5.12	4.43	4.43	5.12	4.43	3.96	6.86	6.86	5.60	4.85	4.34	4.34	4.34	3.96	5.12	4.43	3.96	5.12	6.26	5.12
Row 5	6.26	5.12	5.12	4.43	4.43	4.43	4.43	4.43	4.43	6.86	6.86	6.86	4.85	4.34	4.85	4.85	3.96	6.26	6.26	6.26	6.26	6.26	5.12
Column Avg. Row 1 - 3	5.88	4.89	5.27	4.89	4.66	5.50	5.88	6.36	5.50	6.44	6.02	6.02	6.19	6.47	6.17	6.02	6.75	8.00	6.52	5.88	6.26	5.88	7.13
Row 4 - 5	6.26	4.77	4.77	4.77	4.43	4.43	4.77	4.43	4.20	6.86	6.86	6.23	4.85	4.34	4.60	4.60	3.96	5.69	5.35	5.11	5.69	6.26	5.12
Row 1 - 5	6.04	4.84	5.07	4.84	4.57	5.07	5.44	5.59	4.98	6.61	6.36	6.11	5.66	5.62	5.54	5.45	5.63	7.07	6.05	5.57	6.04	6.04	6.32

Vertical Spacings

	10	11	12	13	14	15	16	17
	4.67	4.67	5.72	5.72	8.09	8.09	5.72	5.22
	5.72	4.67	4.67	5.72	4.04	4.04	4.67	7.38
	5.72	5.72	4.67	4.04	4.04	3.30	4.67	4.26
	5.72	5.72	4.67	4.04	3.62	3.62	3.62	3.30
	5.72	5.72	5.72	4.04	3.62	4.04	4.04	3.30
Row Avg Row 1 - 3	5.37	5.02	5.02	5.16	5.39	5.14	5.02	5.62
Row 4 - 5	5.72	5.72	5.19	4.04	3.62	3.83	3.83	3.30
Row 1 - 5	5.51	5.30	5.09	4.71	4.68	4.62	4.54	4.69

Column	1	2	3	[illegible]	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
F.S.	1.74	1.92	1.91	1 [illegible]	1.94	1.9	1.88	1.86	1.9	1.59	1.6	1.61	1.69	1.68	1.69	1.69	1.68	1.53	1.63	1.64	1.55	1.56	1.62	1.05

A3.5 Bearing Plate Design

Bearing Plate Design Calculations

Known values:

Guesses:

$B := 1 \cdot ft$

$Q := 700 \cdot lb$ $\qquad \phi := 30$ $\qquad c := 150 \cdot \frac{lb}{ft^2}$

$D_f := 0 \cdot ft$ $\qquad \gamma := 106 \cdot \frac{lb}{ft^3}$

From Table 3.2 pg 131(Das 1990)

$N_c := 30.14$ $\qquad N_q := 18.4$ $\qquad N_\gamma := 22.4$ $\qquad q := \frac{1}{2} \cdot \gamma D_f$ $\qquad q = 0 \cdot lb \cdot ft^{-2}$

From Table 3.5 pg 134 (Das 1990) Shape, Depth, and Inclination Factors

Shape Factors $\qquad$ For L=B, L/B=1

$$F_{cs} := 1 + 0.2 \cdot \left(\tan\left(45 + \frac{\phi}{2}\right)\right)^2 \qquad F_{qs} := 1 + 0.1 \cdot \left(\tan\left(45 + \frac{\phi}{2}\right)\right)^2$$

$F_{\gamma s} := F_{qs}$ $\qquad F_{cs} = 1.02$ $\qquad F_{qs} = 1.01$ $\qquad F_{\gamma s} = 1.01$

Depth Factors

For Df = 0, all factors equal 1

$F_{cd} := 1$ $\qquad F_{qd} := 1$ $\qquad F_{\gamma d} := F_{qd}$

$F_{\gamma d} = 1$ $\qquad F_{cd} = 1$ $\qquad F_{qd} = 1$

Inclination Factors $\qquad$ Since no inclined force all factors equal 1

$F_{ci} := 1$ $\qquad F_{qi} := 1$ $\qquad F_{\gamma i} := 1$

$F_{ci} = 1$ $\qquad F_{qi} = 1$ $\qquad F_{\gamma i} = 1$

Equation 3.16 pg 129 (Das 1990)

$$q_u := c \cdot N_c \cdot F_{cs} \cdot F_{cd} \cdot F_{ci} + q \cdot N_q \cdot F_{qs} \cdot F_{qd} \cdot F_{qi} + \frac{1}{2} \cdot B \cdot \gamma \cdot N_\gamma \cdot F_{\gamma s} \cdot F_{\gamma d} \cdot F_{\gamma i}$$

Substitute Q/B^2 for q_u then solve for B

Now Solve for B

$B := 1 \cdot ft$ $\qquad Q := 4721 \cdot lb$ $\qquad Convert := 1 \cdot ft$

Given

$$\left(130 \cdot \frac{lb}{ft^2} \cdot 30.14 \cdot 1.02 + \frac{1}{2} \cdot B \cdot 22.4 \cdot 106 \cdot \frac{lb}{ft^3} \cdot 1.01\right) \cdot B^2 = Q$$

$Plate := Find(B)$

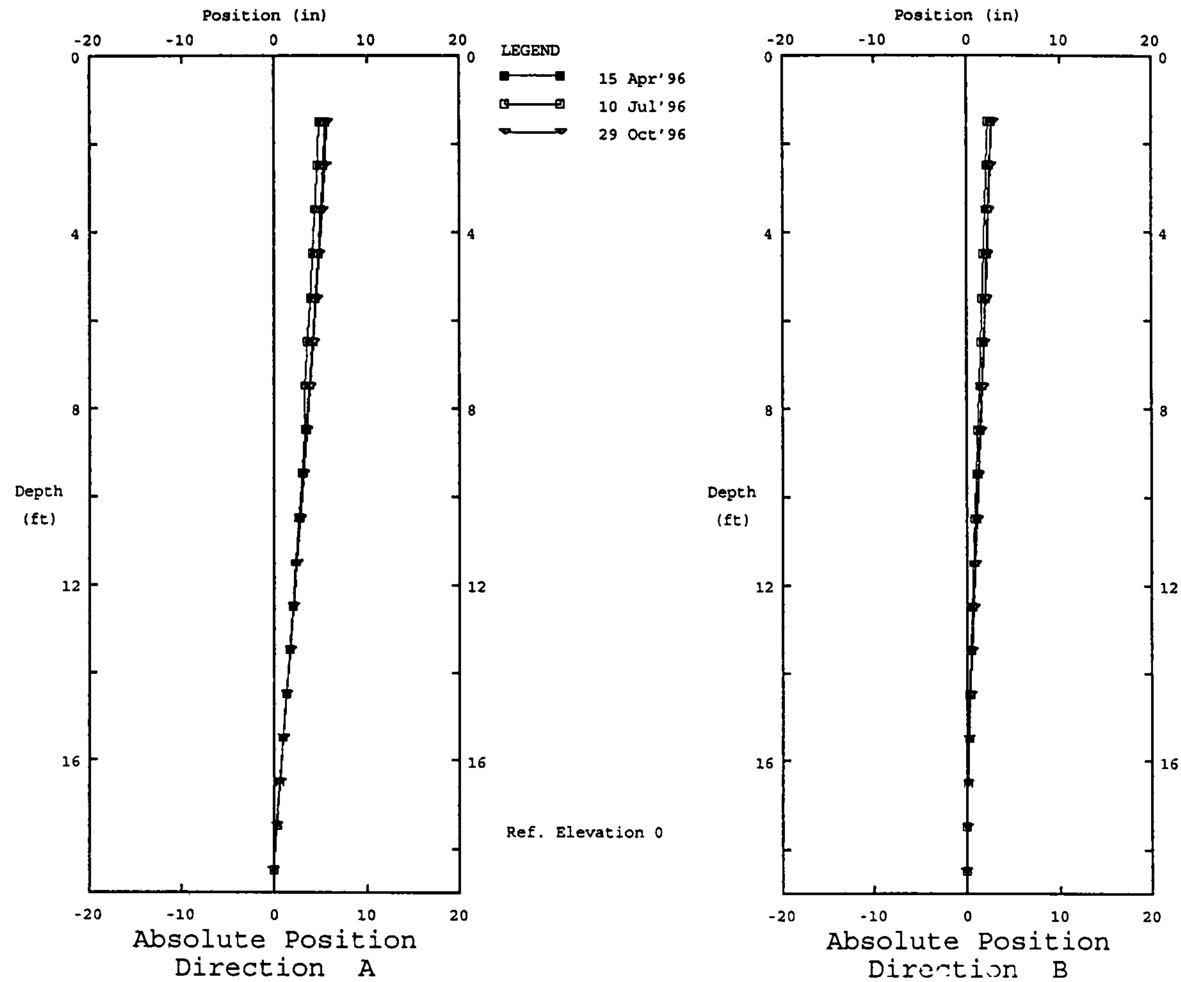
$Plate = 0.958 \cdot ft$ $\qquad Size := \frac{Plate \cdot 12}{Convert}$ $\qquad Size = 11.495$

Appendix 4

Charts & Graphs

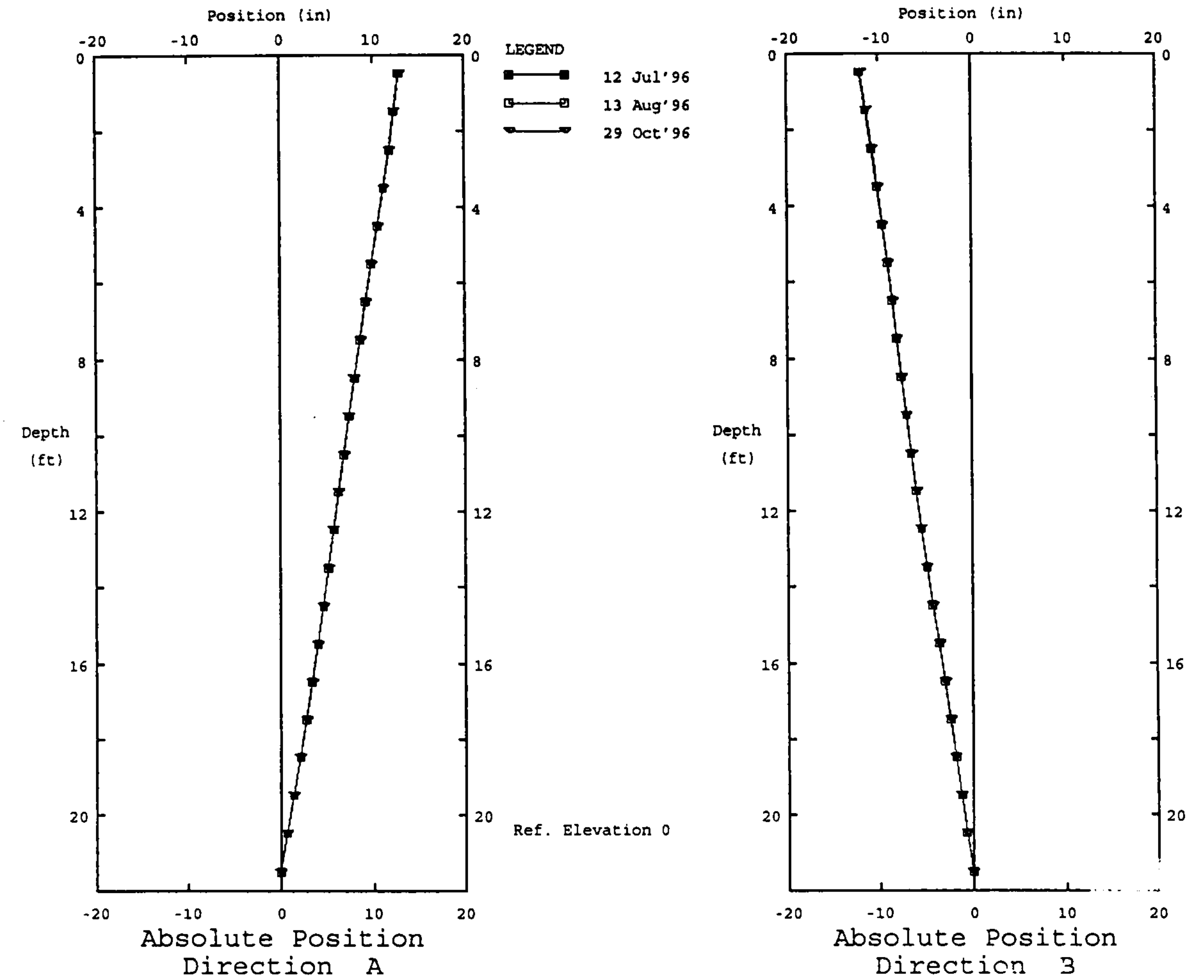
A4.1 Inclinometers Plots

University of Tennessee - Knoxville, TN



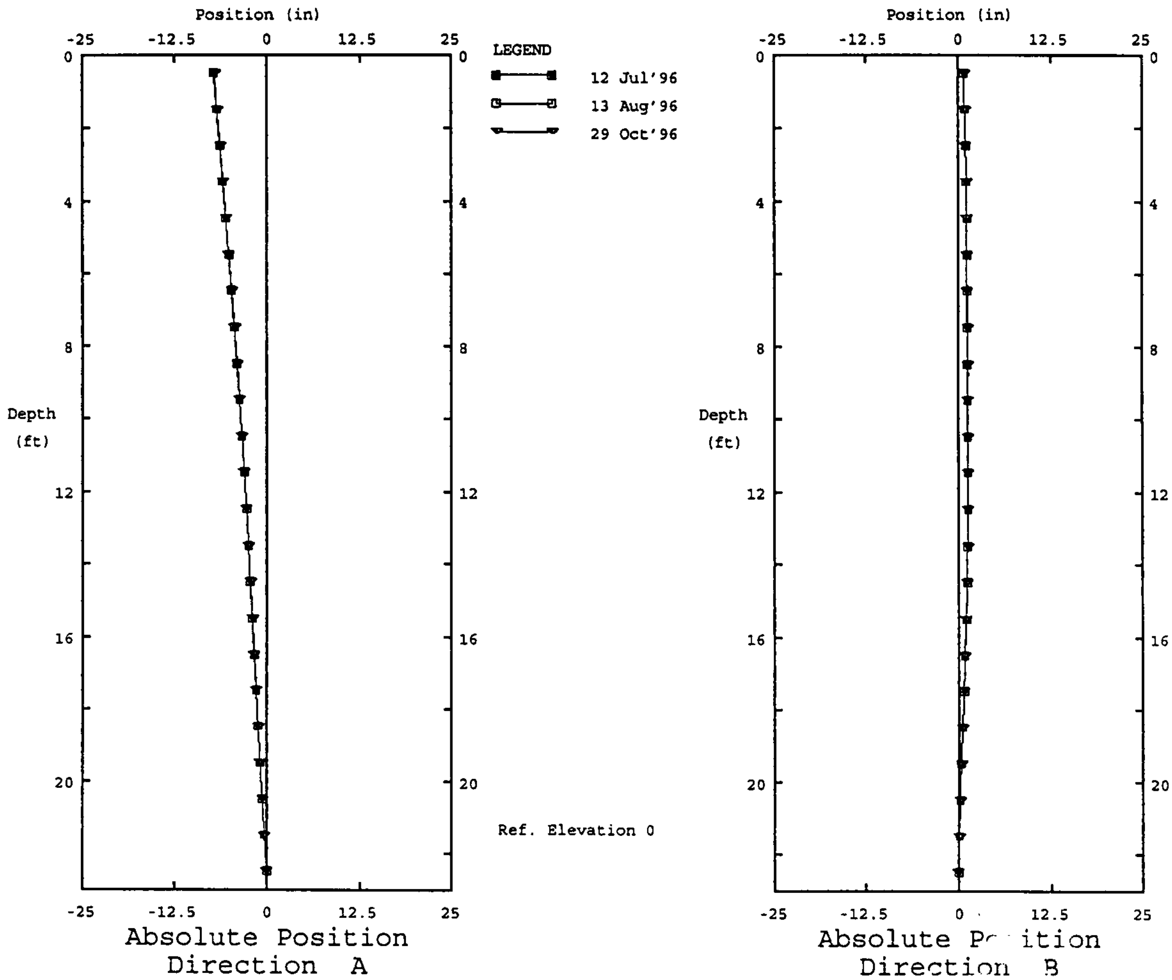
Buffalo Mounatin, Inclinometer Sect 3
University of Tennessee

University of Tennessee - Knoxville, TN



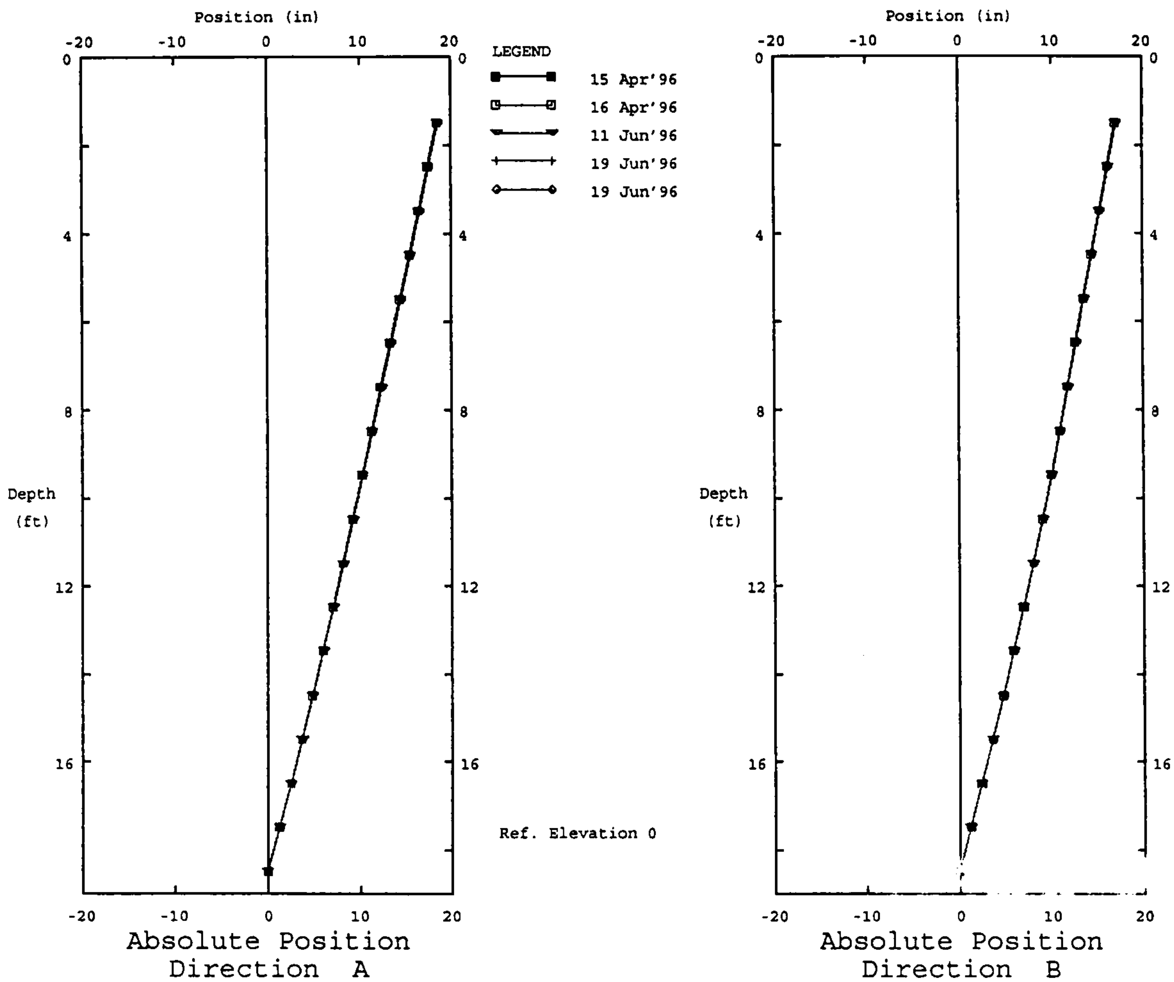
Buffalo Mountain, Inclinometer Road 2/3
University of Tennessee

University of Tennessee - Knoxville, TN



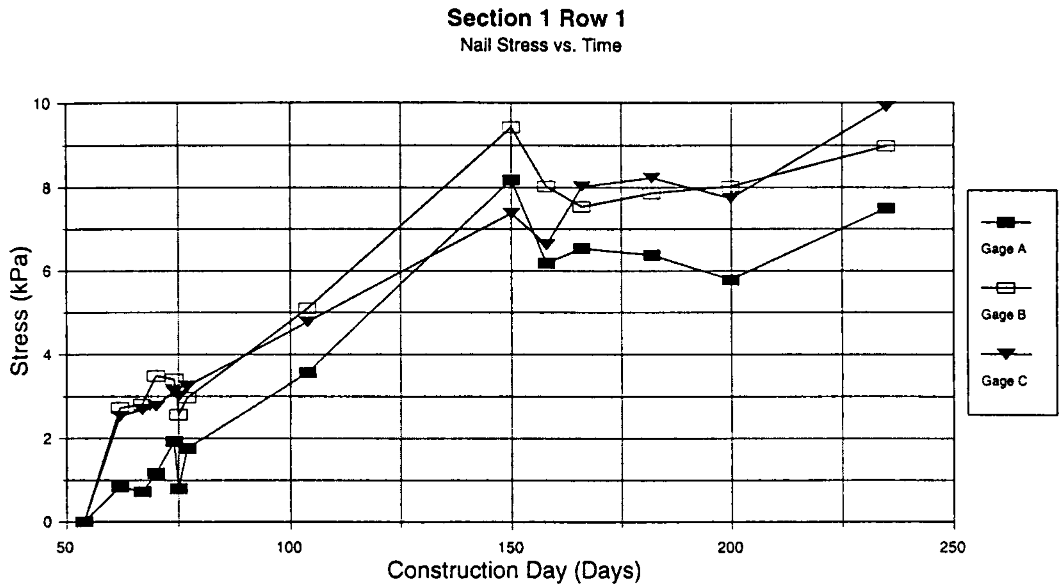
Buffalo Mountain, Inclinometer Road 1/2
University of Tennessee

University of Tennessee - Knoxville, TN

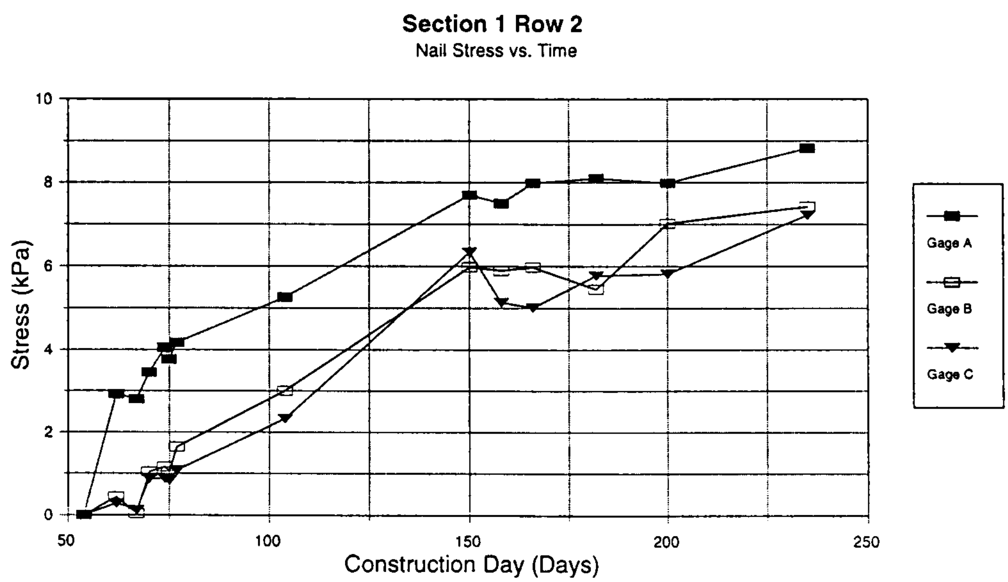


Buffalo Mountain, Inclinometer Section4
University of Tennessee

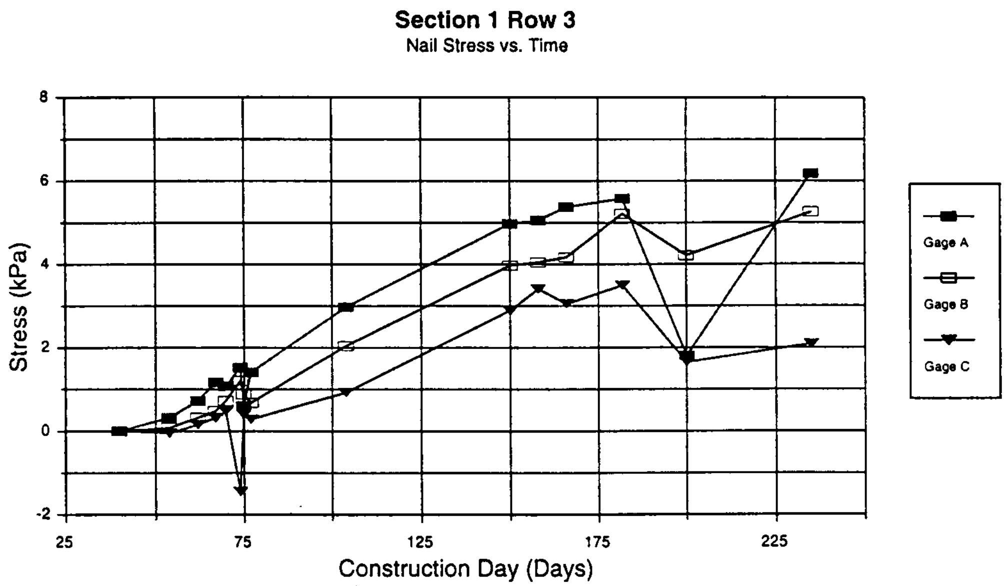
A4.2 Stress Plots

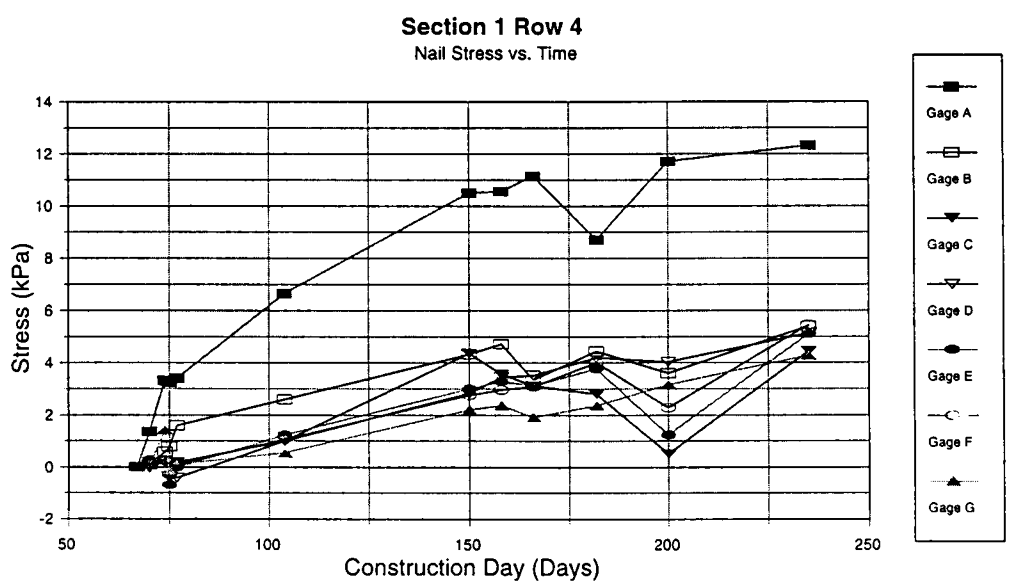


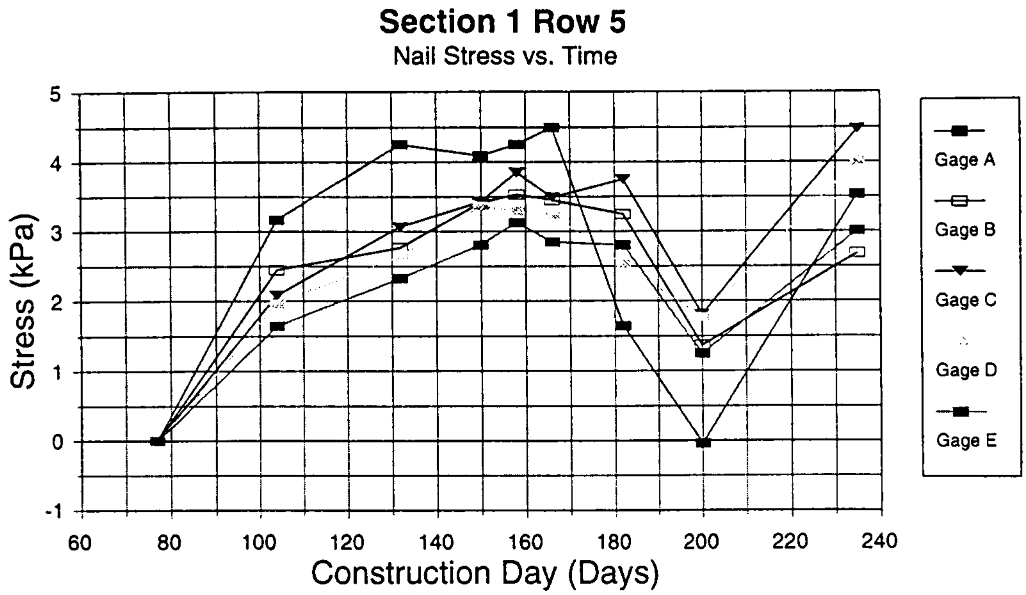
Section 1 Row 1
Nail Stress vs. Time
Stress (kPa)
Construction Day (Days)
Gage A
Gage B
Gage C



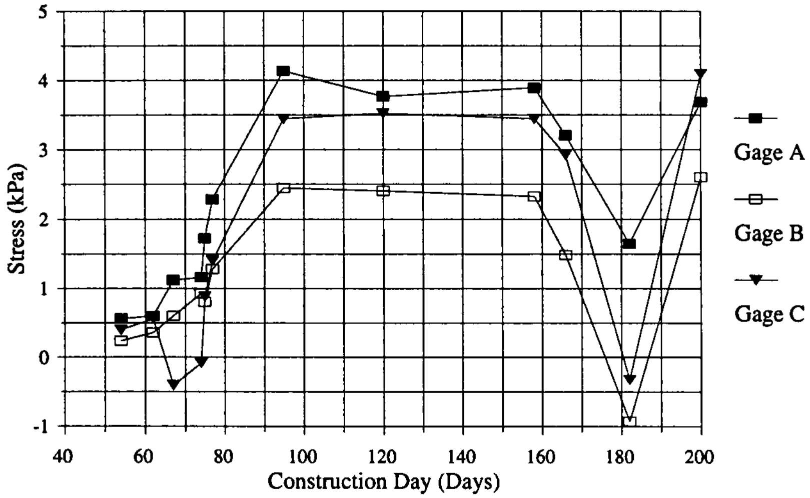
Section 1 Row 2
Nail Stress vs. Time
Stress (kPa)
Construction Day (Days)
Gage A
Gage B
Gage C


Section 1 Row 3
Nail Stress vs. Time
Stress (kPa)
Construction Day (Days)
8
6
4
2
0
-2
25
75
125
175
225
Gage A
Gage B
Gage C

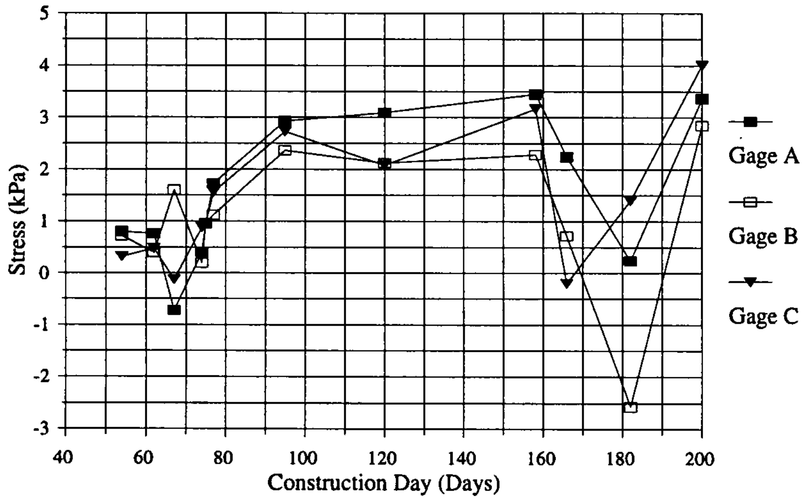

Section 1 Row 4
Nail Stress vs. Time
Stress (kPa)
Construction Day (Days)
14
12
10
8
6
4
2
0
-2
50
100
150
200
250
Gage A
Gage B
Gage C
Gage D
Gage E
Gage F
Gage G


Section 1 Row 5
Nail Stress vs. Time
Stress (kPa)
5
4
3
2
1
0
-1
60
80
100
120
140
160
180
200
220
240
Construction Day (Days)
Gage A
Gage B
Gage C
Gage D
Gage E

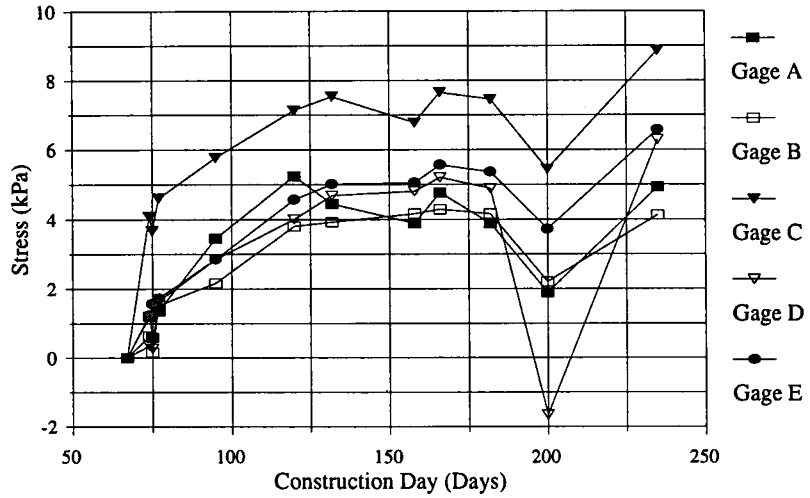
Section 3 Row 1
Stress vs. Time



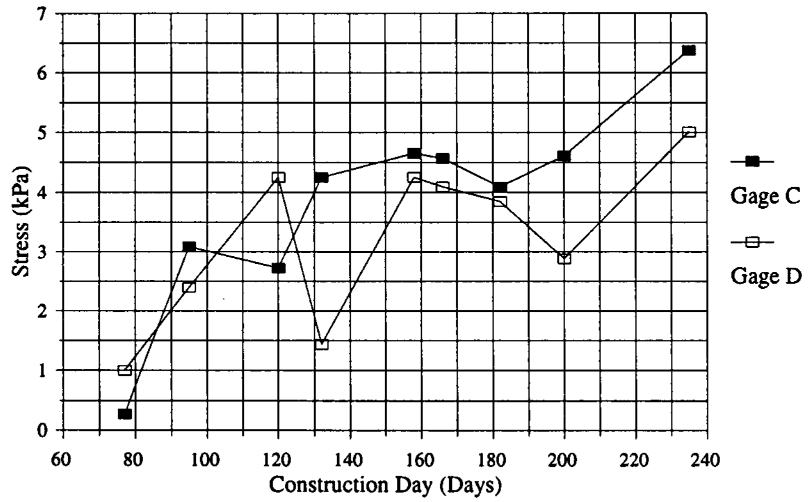
Section 3 Row 2
Stress vs. Time



Section 3 Row 4
Stress vs. Time

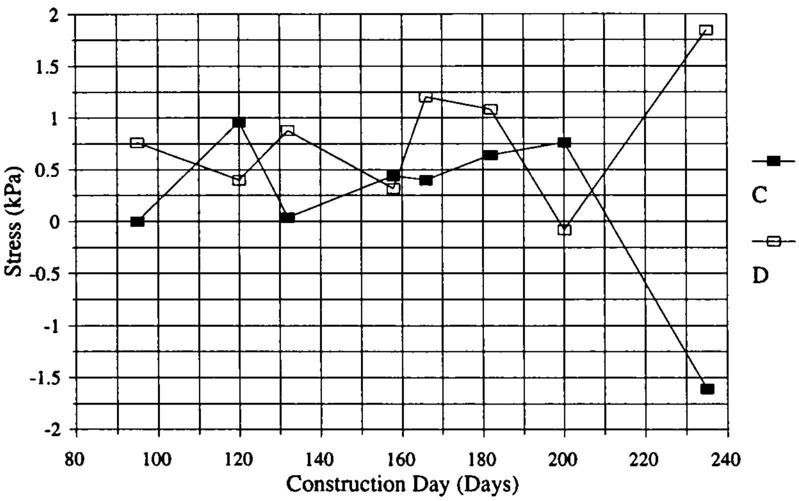


Section 3 Row 5
Stress vs. Time

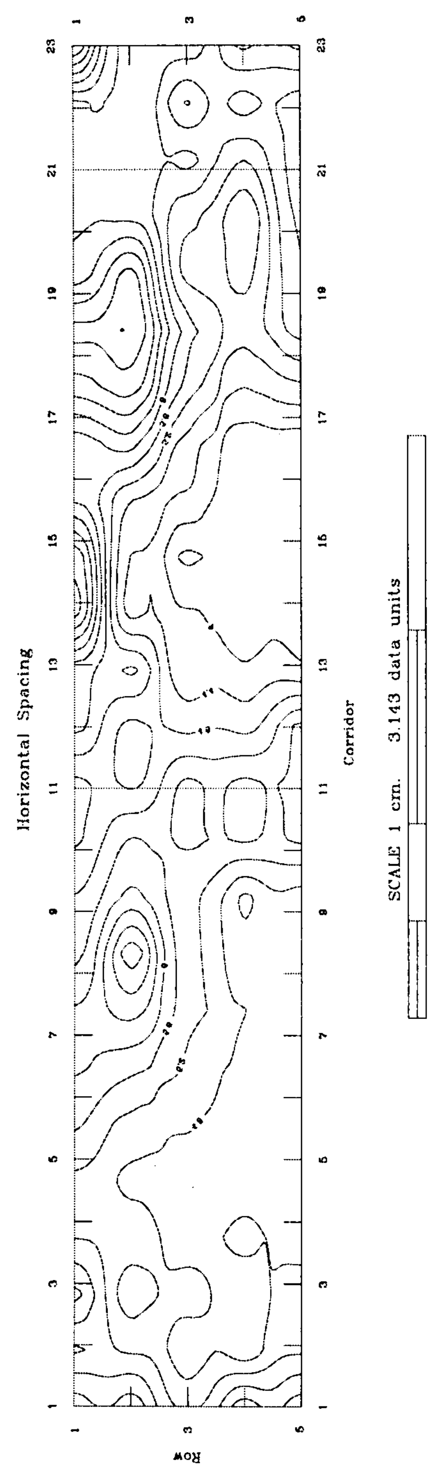


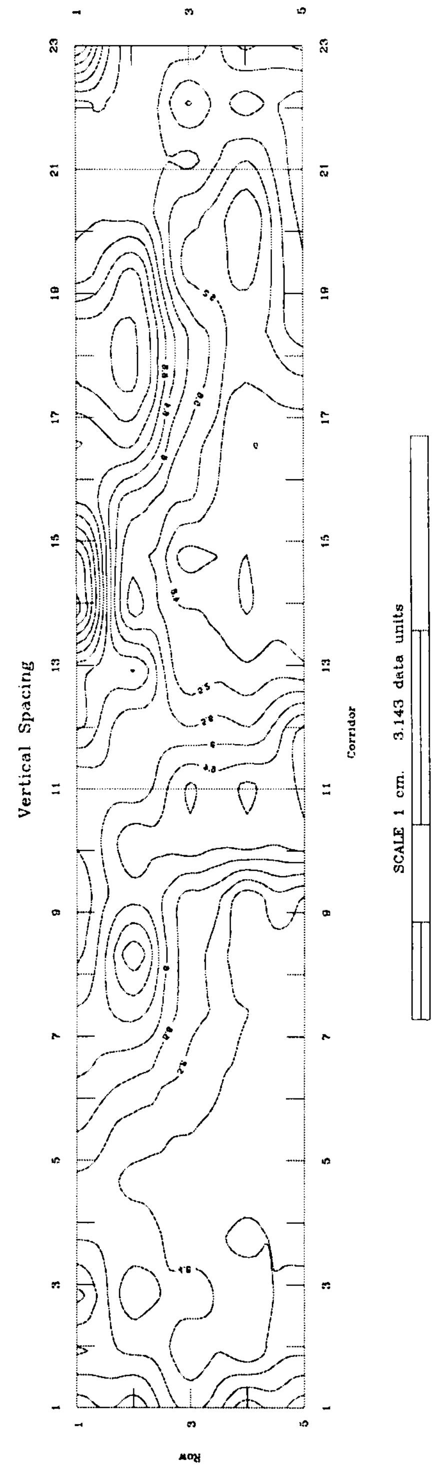
Section 3 Row 6

Stress vs. Time



A4.3 Contour Plots


Horizontal Spacing
Corridor
Row
SCALE 1 cm. 3.143 data units


Vertical Spacing
Corridor
Row
SCALE 1 cm. 3.143 data units

Appendix 5

Computer Input Files Used in Design and Analysis

A5.1 STABL5 Files

```
PROFIL       C:EXIS1995.DAT  PCSTABL Version 6
Existing Slope 1995 Buffalo Mtn.
16 9
0. 29.5 36.5 29.5 1
36.5 29.5 43.06 34.68 1
43.06 34.68 48.3 39.26 1
48.3 39.26 53.65 44.46 1
53.65 44.46 58.98 50.87 1
58.98 50.87 69.59 59.32 1
69.59 59.32 71.6 68.3 1
71.6 68.3 86.6 68.3 1
86.6 68.3 186.6 104.7 1
0. 10. 50. 25. 2
50. 25. 70. 50. 2
70. 50. 160. 79. 2
0. 0. 60. 25. 3
60. 25. 70. 42.3 3
70. 42.3 90. 42.3 3
90. 42.3 160. 69. 3
SOIL  spoil   Clay    Sanstone
3
106. 116. 165. 30. 0. 0. 0
117.8 128.9 375.9 28. 0. 0. 0
130. 130. 9999. 45. 0. 0. 0
WATER
1 62.4
8
20. 20.
25. 22.58
35. 27.66
50. 35.
80. 48.71
120. 65.
140. 72.28
160. 79.
CIRCL2-Bishop circular, search.
40 40
20. 36. 72. 100. 0. 10. 0. 0.
```

```
PROFIL        C:CUTSLOP  PCSTABL Version 6
13 ft High Wall Analysis w/ Mre Soil
7 5
0. 20. 40. 20. 1
40. 20. 44. 33. 1
44. 33. 56.6 46. 1
56.6 46. 71.6 46. 1
71.6 46. 171. 83. 1
44. 33. 144.7 40. 2
40. 20. 140. 20. 3
SOIL  spoil   Clay    Sandstn
3
106. 116. 165. 30. 0. 0. 0
117.8 128.9 375.9 28. 0. 0. 0
130. 130. 9999. 45. 0. 0. 0
WATER
1 62.4
3
42. 26.
56.75 35.
156.75 70.
LIMITS
1 1
40. 20. 140. 20.
CIRCL2-Bishop circular, search.
10 10
40.1 40.2 56.6 71.6 0. 5. 0. 0.
```

```
PROFIL        C:NEW1975.DAT  PCSTABL Version 6
1975 Slope Pofile Buffalo Mtn.        Prior to slope failure
7 4
0. 29.5 39.1 29.5 1
39.1 29.5 70. 68.3 1
70. 68.3 90. 68.3 1
90. 68.3 186.6 104.7 1
0. 10. 50. 25. 2
50. 25. 70. 50. 2
70. 50. 160. 79. 2
SOIL  spoil   Clay
2
106. 116. 165. 30. 0. 0. 0
117.8 128.9 375.9 28. 0. 0. 0
WATER
1 62.4
5
39.1 29.5
70. 48.71
120. 65.
140. 72.28
160. 79.
CIRCL2-Bishop circular, search.
40 40
30. 39.1 70. 90. 0. 5. 0. 0.
```

```
PROFIL       C:SEC4.RAY  PCSTABL Version 6 /O(35, 1495)
Section 4 Scarp, from survey
6 5
15. 6.9 15.1 19.9 2
15.1 19.9 15.2 22.9 2
15.2 22.9 24. 31.9 1
24. 31.9 38.1 35.2 1
38.1 35.2 53.1 35.2 1
15.2 20. 61. 23. 2
SOIL  UDE     MrE
2
106. 116. 165. 30. 0. 0. 0
117.8 128.9 376. 28. 0. 0. 0
LIMITS
1 1
15. 6.9 45. 6.9
SURFAC
3
15. 6.9
22.5 23.9
25.22 32.19
EXECUT
```

```
PROFIL        C:SEC4TEST  PCSTABL Version 6 /O(45, 1495)
Section 4, random analysis
5 4
5. 6.9 5.1 22.9 2
5.1 22.9 14. 31.9 1
14. 31.9 28.1 35.2 1
28.1 35.2 43.1 35.2 1
5.1 22.9 35. 30. 2
SOIL  UDE      MrE
2
106. 116. 165. 30. 0. 0. 0
117.8 128.9 376. 28. 0. 0. 0
LIMITS
1 1
5. 6.9 35. 6.9
CIRCL2-Bishop circular, search.
40 40
5. 8. 13. 35. 0. 1. 0. 0.
```

A5.2 SNAIL Ver. 2.11 Files

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"TITLE"
"Section 1 As Built"
"FLAGS"
1,1,0,0,0
"GEOMETRY"
11,20,46,14,0,15,20,50,0
"SURCHARGE"
0,0,0,0,0,0,0,0
"REINFORCEMENTLAYERS"
3,5,20,60,8
"SOIL"
2
106,30,165,5,117,28,376,1.97
4.18,11.5,40,11.5
"EARTHQUAKE & Horizontal FORCE."
0,0,0
"SEARCHLIMIT"
0,50
"WATERTABLE"
-15,0,20,10,75,20
"NODELIMITS"
1,0,10,0
"REINFORCEMENT"
19,15,1,5,1
```

```
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"Section 3 Design"
"FLAGS"
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"GEOMETRY"
13,20,46,14,0,15,20,50,0
"SURCHARGE"
0,0,0,0,0,0,0,0
"REINFORCEMENTLAYERS"
3,5,20,60,8
"SOIL"
2
106,30,165,5,118,28,376,2
4.18,11.5,50,11.5
"EARTHQUAKE & Horizontal FORCE."
0,0,0
"SEARCHLIMIT"
0,50
"NODELIMITS"
1,0,10,0
"REINFORCEMENT"
14,15,1,1,2
8,15,5,1,2
8,15,5,1,2
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"TITLE"
"Section 2 Design"
"FLAGS"
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"GEOMETRY"
13,20,46,13,0,15,20,50,0
"SURCHARGE"
0,0,0,0,0,0,0,0
"REINFORCEMENTLAYERS"
3,5,15,60,8
"SOIL"
2
106,30,165,5,118,28,376,2
3.28,9,50,10
"EARTHQUAKE & Horizontal FORCE."
0,0,0
"SEARCHLIMIT"
0,40
"WATERTABLE"
-15,0,20,10,70,20
"NODELIMITS"
1,0,10,0
"REINFORCEMENT"
14,15,1,5,1
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"Section 1 Design"
"FLAGS"
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"GEOMETRY"
13,20,46,14,0,15,20,50,0
"SURCHARGE"
0,0,0,0,0,0,0,0
"REINFORCEMENTLAYERS"
3,5,20,60,8
"SOIL"
2
106,30,165,5,117,28,376,1.97
4.18,11.5,40,11.5
"EARTHQUAKE & Horizontal FORCE."
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0,50
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"REINFORCEMENT"
19,15,1,5,1
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"REINFORCEMENTLAYERS"
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"SOIL"
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3.17,18,50,25
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1,0,10,0
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"SURCHARGE"
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"REINFORCEMENTLAYERS"
3,5,20,60,8
"SOIL"
2
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0,50
"NODELIMITS"
1,0,10,0
"REINFORCEMENT"
14,15,1,1,2
8,15,5,1,2
8,15,5,1,2
```

VITA

Ray Tant was born at St. Thomas Hospital in Nashville, Tennessee around 1:00 a.m. September 3, 1969. He attended schools in the Davidson County parochial school system where he was a student at St. Ann's elementary school, and graduated from Father Ryan High School in May, 1987. He attended Belmont College in Nashville, Tennessee where he studied Political Science and English from 1987 to 1990. In the fall of 1990 he transferred to the University of Tennessee to study Civil Engineering, and completed his Bachelor of Science in Civil Engineering in August, 1994. While an undergraduate he served as the President of the student chapter of The American Society of Civil Engineers (ASCE), and was initiated into the civil engineering honor society, Chi Epsilon. Following graduation he immediately began work on his Master's degree in August 1994 which investigated the use of soil nailing for the stabilization of mine waste slopes. While a graduate student he served as the Chairman for the 1996 ASCE Southeastern Regional Student Conference, the Marshall of Chi Epsilon, and was a member of the Undergraduate Enhancement Committee (UEC). He completed his work in December 1996, and received his Master's degree in May, 1997.