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Slope Stability Analysis of a Lake Michigan Coastal Bluff

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SLOPE STABILITY ANALYSIS OF A LAKE MICHIGAN COASTAL BLUFF

by

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To my mother Bernadine, who has always been an inspiration to me in overcoming adversity and achieving hard fought goals. I love you very much. Your example has helped keep me going.

To my husband Mark, who helped me take measurements along the Lake Michigan shore even in the coldest of winter months. You are my rock. Your love for me is constant, eternal and unquestioned. There is absolutely no one else on earth that I would rather have at my side as we journey through life. I am proud to share this achievement with you. I love you.

Gregory Carlton Young

SLOPE STABILITY ANALYSIS OF A LAKE MICHIGAN COASTAL BLUFF

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Western Michigan University, 2004

Bluff failure along the Lake Michigan coast can be produced primarily by increased pore pressure from perched water tables above lacustrine clay deposits. To test this theory, a site in Allegan County, Michigan with alternating layers of sand, clay and glacial till was chosen for bluff failure monitoring. Historically, the site experiences sporadic massive failures as opposed to neighboring sites that show more regular and uniform displacements. Four pole and cable monitoring lines were measured bi-weekly from December 2001 to October 2003. Lake levels were low and no erosion of material at the base of the slope occurred. Slumps above shallow shear planes were observed during the winter and spring seasons. Limit equilibrium analyses, replicating both pore pressure fluctuations and wave cutting, suggests that increased pore pressure is the dominant factor associated with the movement. The periodic massive failures appear to be controlled largely by a combination of intermittent, voluminous groundwater infiltration and by the large volume of weak, unconsolidated sand in the lower portion of the stratigraphic section.

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CHAPTER 1

INTRODUCTION

Many property owners along the Great Lakes coasts are continuously challenged to prevent loss of their land by slope failures caused by erosion. Slope failures worldwide result in loses of life, real estate and structures every year. The process begins with potentially unstable rock, debris or earth. Failure mechanisms pertain to the type of movement that will occur. Trigger mechanisms indicate the principal factor that brought, or will bring, failure to a slope, either by decreasing the forces resisting failure or increasing the forces driving failure.

Failure mechanisms include falls, topples, slides (both rotational and translational), lateral spreads and flows. When two or more failure mechanisms are involved, the slope movement is said to be complex. Failure mechanisms are further divided into the type of material that is involved, either rock, debris or earth (Varnes, 1978) (Table 1).

Trigger mechanisms are often complex and may involve several factors such as changes in groundwater levels, rainfall amounts prior to failure, melting snow, freeze/thaw, changes in pore fluid pressure, reduction in the strength of

Type of Movement	Type of Material		
	Bedrock	**Engineering Soils** **Coarse**	**Fine**
Falls	Rock Fall	Debris Fall	Earth Fall
Topples	Rock Topple	Debris Topple	Earth Topple
Slides Rotational	Rock Slump	Debris Slump	Earth Slump
Translational (few units)	Rock Block Slide	Debris Block Slide	Earth Block Slide
Translational (many units)	Rock Slide	Debris Slide	Earth Slide
Lateral Spreads	Rock Spread	Debris Spread	Earth Spread
Flows	Rock Flow (deep creep)	Debris Flow (soil creep)	Earth Flow
Complex	Two or more principal types of movement		

Table 1

SLOPE FAILURE CLASSIFICATIONS

(Varnes, 1978)

the soil, loading, changes in slope geometry, artificial vibration and earthquakes (Wieczorek, 1987; Crowell et al., 1991; Azzoni et al., 1992; Julian and Anthony, 1996; Mazzoccola and Hudson, 1996; Pellegrini and Surian, 1996; Wieczorek and Jager, 1996).

Identification of slopes prone to displacement, and the failure and trigger mechanisms associated with them, is paramount to deciding how best to alleviate the problem. Slope risk assessment is often complicated by not being able to determine accurately the trigger mechanism that will cause the slope to fail. Often it is even difficult to determine the trigger mechanism responsible for movement after a slope has failed (Wieczorek and Jager, 1996).

The prevailing geologic thought has been that slope movements along the shore of Lake Michigan were primarily the result of wave erosion at the base of the slope (Edil and Vallejo, 1976; Mickelson et al., 1991). However, a Lake Michigan coastal bluff that contains perched groundwater above lacustrine clay deposits has continued to recede even with low lake levels and minimal toe erosion resulting from wave action. An analysis of the movement and correlation of high groundwater levels points to the high levels of perched groundwater decreasing soil cohesion within the slope and has been proposed as the primary cause of failure in this area (Chase et al., 2001b). The bluff, on the southeast coast of Lake Michigan, is identified as being prone to failure. A geologic investigation was made to determine the stratigraphy of the site, displacement was monitored, movement causes and effects were analyzed and possible solutions are advanced.

CHAPTER 2

OBJECTIVES

Slope failure along the eastern shore of Lake Michigan is an ongoing problem. At a site between Saugatuck and South Haven in Allegan County, Casco Township, Michigan, (Figure 1) slope failures are nearly continuous events with relatively minor movements that accumulate into major displacements. From 1997 to 2001, water levels on Lake Michigan have been receding and have been below average since 1999 (Figure 2). When lake levels were high, numerous failures occurred along the southeastern shore of Lake Michigan and were almost universally attributed to erosion at the base of the slope as a result of wave action. At that time, property owners along the shoreline were concerned about the erosion and many types of wave deflection systems were installed. Since approximately 1997, Lake Michigan water levels have lowered and there has been minimal toe erosion resulting from wave action. Locally, however, some areas have continued to show slope movement. This recent movement is attributed to perched water tables above lacustrine clay deposits and coincides with increased pore water pressure in the winter months due to water trapped in the slope from frozen surface materials, and in the spring months due to high groundwater levels from snowmelt and heavy precipitation (Chase et al., 2001b).

Figure 1

LOCATION OF STUDY SITE

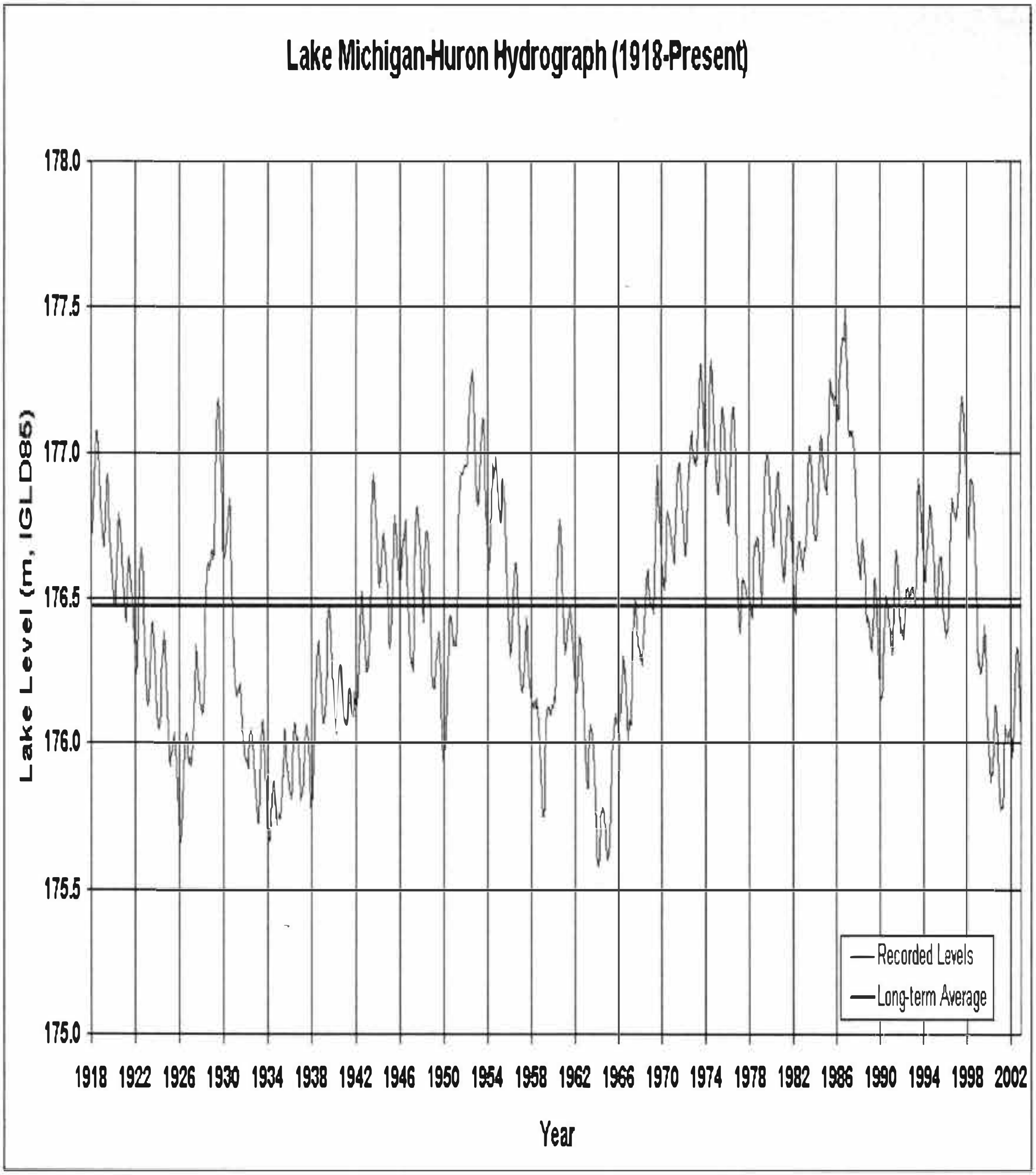


Figure 2

HISTORICAL LAKE MICHIGAN WATER LEVELS

(Data from NOAA)

Without an understanding of the causes of the failures and the trigger mechanisms at various sites with differing stratigraphic units, mitigation techniques that seem to work at one location may not be appropriate at other areas. Wave erosion at the toe of the slope is frequently thought to be the cause of failures along the bluffs. However, even with wave cut erosion at the base of the bluffs, some areas are particularly resistant to failure while others seem to fail even with low lake levels and minimal erosion at the toe of the slope due to wave activity (Chase et al., 2001b). A study area that showed evidence of failure about one kilometer north of an existing site known as Miami Park South (Figure 3) was chosen for bluff failure monitoring.

The site is one of three chosen jointly for study and remediation measures by the U.S. Army Corps of Engineers and Western Michigan University. The site has a history of movement that will be documented here and is one of the most active sections in a sixteen kilometer coastal reach between Saugatuck and South Haven (Montgomery, 1998). The study site is particularly prone to minor non-continuous, shallow movements as evidenced by the lack of vegetation. However, the slopes visibly appear unstable and massive failures are anticipated if pore water pressures reach a threshold point. Exposed stratigraphy at the site makes correlation of stratigraphy and movement possible. In addition, a major failure at a similar area approximately five hundred meters south of Miami Park North in 1997 makes this site particularly interesting and likely vulnerable to a similarly massive failure (Chase et al., 1999).

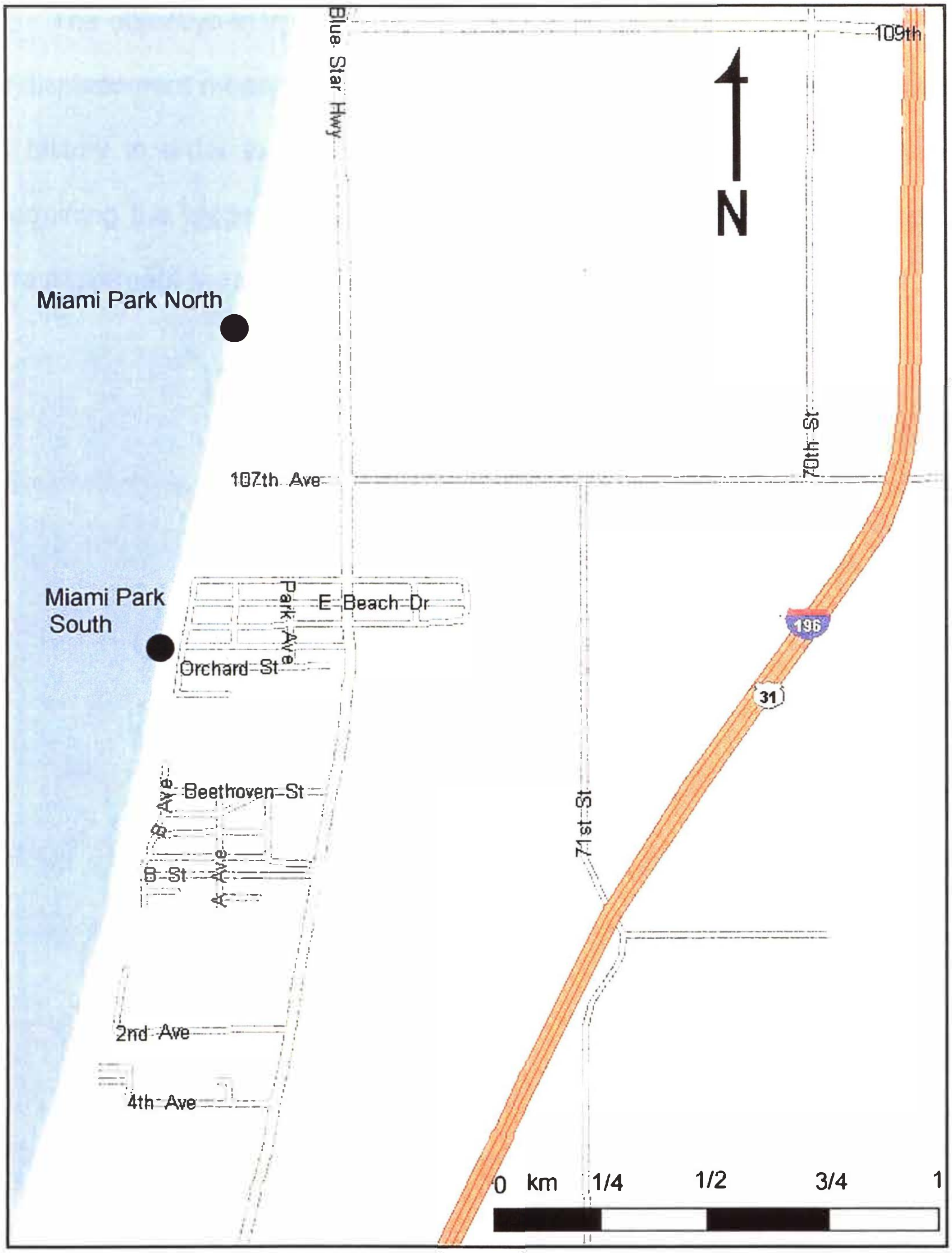


Figure 3

MAP OF STUDY AREA

The objective is to accumulate a foundation of data including stratigraphic and displacement measurements and to demonstrate the movement mechanism and history in order to suggest methods of preventing or slowing movement. Determining the triggering mechanisms working within the slope and predicting future movement areas are objectives as well.

CHAPTER 3

GEOGRAPHY

The study site is a coastal bluff along the eastern shore of Lake Michigan at 738 and 742 Blue Star Highway, Casco Township, Allegan County, Michigan (Figure 3). Historically, numerous glacial advances and retreats of the Lake Michigan Lobe during Late Wisconsin glaciation formed the bluff structure. Recently, shallow slope movements due to surface erosion resulting from runoff and draining seeps within the bluff have contributed to the present bluff topography. The maximum bluff height at this location is approximately twenty meters from current lake levels at the south end of the site and gradually decreases toward the north. The average inclination of the slope is approximately 35°. Four survey lines were erected in the fall of 2001 and were monitored for movement approximately twice a month from December 2001 to October 2003.

The site is devoid of vegetation and, according to local residents, has been without significant vegetation for over twenty years. The northern most section of the study area is the exception however and it is here that various forms of vegetation, predominately smaller scrub trees, are rooted. The climate is typical of southwest Michigan with cold winters, wet springs, humid summers and cool autumn seasons (Table 2).

Month	Avg. High	Avg. Low	Mean	Avg. Precip.	Record High	Record Low
Jan	31°F	20°F	26°F	2.09 in.	67°F (1950)	-14°F (1994)
Feb	35°F	23°F	29°F	1.59 in.	71°F (1999)	-17°F (1934)
Mar	44°F	30°F	37°F	2.08 in.	81°F (1981)	-8°F (1969)
Apr	55°F	39°F	47°F	3.25 in.	87°F (1986)	13°F (1982)
May	66°F	49°F	57°F	3.10 in.	93°F (1934)	24°F (1978)
Jun	74°F	59°F	67°F	3.15 in.	100°F (1953)	35°F (1945)
Jul	78°F	64°F	71°F	3.48 in.	99°F (1934)	39°F (1979)
Aug	78°F	64°F	71°F	3.56 in.	99°F (1934)	37°F (1927)
Sep	72°F	56°F	64°F	4.08 in.	97°F (1939)	26°F (1942)
Oct	61°F	46°F	54°F	2.78 in.	86°F (1951)	17°F (1976)
Nov	48°F	36°F	42°F	3.27 in.	78°F (1961)	-10°F (1950)
Dec	36°F	26°F	31°F	2.75 in.	68°F (1982)	-12°F (1935

Table 2

MICHIGAN TEMPERATURE AND PRECIPITATION

(Data from NOAA)

CHAPTER 4

RELATED WORK

Studies of the Great Lakes shorelines and their recession have produced a variety of theories from a number of authors. Papers from Edil and Mickelson (Edil and Vallejo, 1976; Mickelson et al, 1991) have advanced the theory that bluff failure along the Lake Michigan shore in Wisconsin is primarily the result of wave erosion at the toe of the slope. While they acknowledge that failure occurs when lake levels are low and minimal erosion due to wave action occurs at these times, they advanced the theory that the toe erosion is the primary cause of bluff failure and that without it the bluffs would eventually become stable. In the report by Edil and Vallejo (1976), coastal bluff failure is theorized to be the result of "wave action at the bluff toe and the degradation of the bluff face by solifluction and surface runoff." Bosscher (1998), along with Edil and Mickelson, published "Evaluation of Risks of Slope Instability along a Coastal Reach." Findings largely concur with earlier reports by Edil and Mickelson in that wave action was the primary cause of failures that produced rotational slip, translational slides, sheetwash and solifluction although they do acknowledge that a groundwater problem also exists.

Mass movements that occurred in 1914, 1971 and 1995 in northern Michigan at the Sleeping Bear Dunes National Lakeshore are described by

Barnhardt, Jaffe and Kayen (1999). All three failures occurred at the same location during unseasonably warm winter weather. The 1995 failure deposited approximately one million cubic meters of debris into Lake Michigan, which extended from three to four kilometers into the lake. Four hypotheses are advanced to explain the failures: 1) wave erosion of toe material, 2) loading and subsequent failure of the offshore slope, 3) increases in pore fluid pressure attributed to the warm weather and 4) stratigraphy.

Montgomery (1998) studied the role of groundwater in shoreline recession along a sixteen kilometer stretch of Lake Michigan that includes the present study area. Examination of over two hundred public well records enabled him to map the groundwater hydrology and correlate that with failures in areas that contained lacustrine clay aquitards in their stratigraphic profiles.

Extensive research near the study area examined here has been done by Chase et al. (2001a; 2001b). In regards to the relationship between perched water tables and toe erosion due to wave action from high lake levels or major storms, they advance the theory that increased pore water pressure in the winter, due to bluff face freezing, and in the spring, due to release of water behind frozen bluff faces, high precipitation, and snow melt are the primary driving forces behind the failures and that erosion of the failed toe material at the base is secondary.

CHAPTER 5

GEOLOGY

Bedrock beneath the study area consists of Coldwater Shale deposited during the Mississippian Period (Leverett and Taylor, 1915). Pleistocene glaciation, and more specifically, advances and retreats of the Lake Michigan glacial lobe into the Michigan Basin, as well as the stages of formation of Lake Michigan, are the important geological events that comprise the relevant stratigraphy and depositional patterns for this site (Figure 4).

A review of Hansel et al. (1985) and Colman et al. (1994) yields the following synopsis. The farthest southern advance of the Lake Michigan lobe was into central Illinois and Indiana to the south. As the Lake Michigan lobe retreated, glacial Lake Chicago was formed in the area now occupied by the southern area of Lake Michigan (~14,000 years BP). The retreating Michigan lobe glacial melt water filled the basin and Lake Algonquin was eventually formed (~11,500 years BP), which roughly encompassed the basins that now contain Lakes Michigan, Huron and Superior. As the ice retreated north of Lake Algonquin, the North Bay outlet leading from Ontario, Canada to the Atlantic Ocean was opened. This allowed water to drain from Lake Algonquin to a stage known as Lake Chippewa in the Michigan Basin (~9500 years BP). The lowered

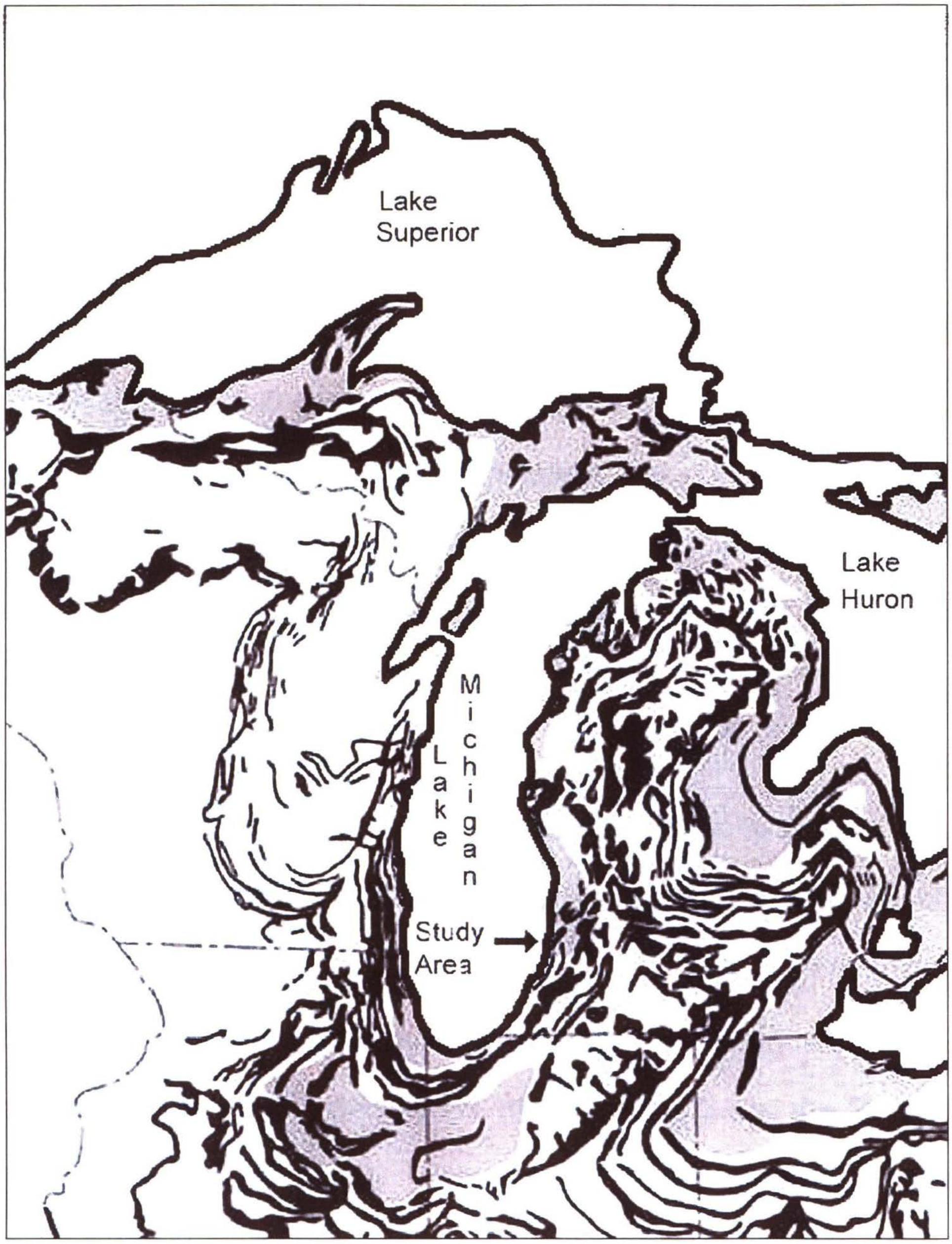


Figure 4

STUDY SITE IN RELATION TO REGIONAL MORAINES water levels increased the gradients of the streams feeding the lake and removed water and sediment from inland areas. Crustal rebound in the North Bay region eventually closed the North Bay outlet and allowed water levels to rise to form Lake Nippissing in the Lake Michigan, Huron and Superior basins. At this stage, they are together known as the Nippissing Great Lakes (~5000 years BP). The final stage before the present Great Lakes was the Algoma Great Lakes stage (~2500 years BP). The Algoma stage is the last stage before the present Great Lakes system and represents a period where the Nippissing Great Lakes, which had been lowering their water levels, stabilized. This allowed some strong shoreline features to evolve; features that are of interest in the current slope stability study.

Due to the glacial advances and retreats, stratigraphy in the study area is heterogeneous. The geological stratigraphic units that compose the shoreline bluff in this location are predominantly glacial tills, sand, lacustrine silt/clay or a combination of the three. The three regional glacial diamicton units, from bottom up, are known as the Glenn Shores Till, the Ganges Till and the Saugatuck Till (Monaghan and Larson, 1986). Interspersed between these till units are deposits of light brown fine to medium grained sand and varved gray lacustrine silt/clay. Only one diamicton layer has been observed at the study location. This is assumed to be the Saugatuck Till based on its location at the upper portion of the bluff. The Glenn Shores Till is presumed to be located below lake level and the Ganges Till is probably below lake level as well or slightly above it in places. Sand and clay deposits between the Ganges Till and the Saugatuck Till may have been the result of rising and falling lake levels or former stream channels.

CHAPTER 6

METHODOLOGY

Field Methods

Field measurements using a pole and cable monitoring system developed by Chase et al. (2001a) were taken on a bi-monthly basis and input into a spreadsheet also developed by Chase et al. (2001a) to record movement history at the site. This system enabled measurements that a more costly extensometer and inclinometer installation would have provided. Four pole and cable monitoring lines were installed along the study area down the face of the bluff.

The locations of the lines were chosen within the property boundaries of 738 and 742 Blue Star Highway, Casco Township, Allegan County, Michigan where permission to monitor the slope was obtained. The northern most line, Line A, was established at the northern edge of the property line at 738 Blue Star Highway and was expected to be a reference line with minimal movement because relatively dense vegetative growth was along the line and evidence of past movement was not apparent (Figure 5). The remaining three lines were installed on the bluff at 742 Blue Star Highway. Line B was established at a

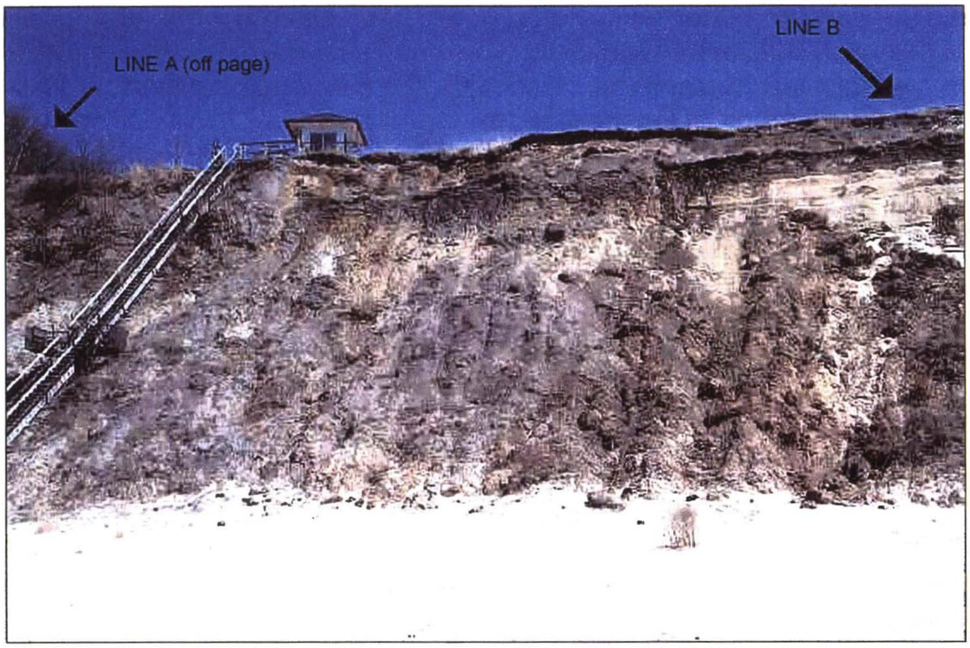


Figure 5

PHOTOGRAPH OF LINES A AND B

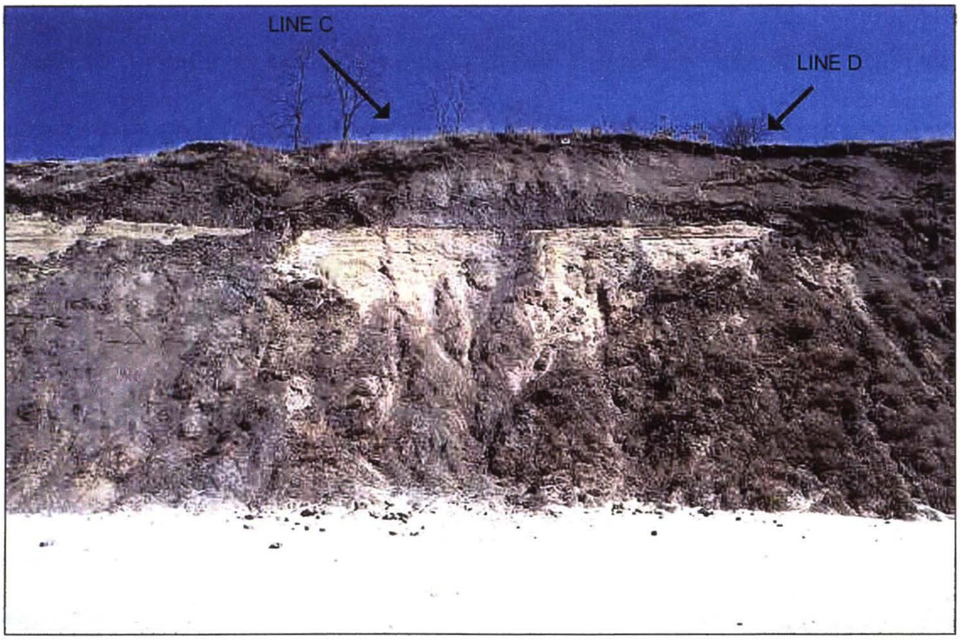


Figure 6

PHOTOGRAPH OF LINES C AND D

location that showed evidence of recent slump movement (Figure 5). Line C was installed at a location chosen for its exposed stratigraphy as well as the near vertical profile near the top of the bluff at this location (Figure 6). The location of line D is near the southern property boundary of 742 Blue Star Highway in an area that had evidence of recent overland flow (Figure 6).

The measurement system developed by Chase et al. (2001a) that was installed consisted of common barbed wire, eight-foot long poles. The poles were pounded vertically into the bluff to a depth of about four feet at regular intervals from the top to the bottom of the bluff. For each line, an unattached reference pole was placed landward from the top of the bluff. This enabled an independent measurement to be made of movements of the initial pole at the top of the bluff to the pole set back away from the bluff face and showed if any movement of the entire system occurred. Plastic coated steel cables were taped at 1.5 inch intervals. The cables were strung through steel eyebolts attached near the top of the poles. The entire cable system was kept tight by a common concrete building block attached to the end of the cable at the base of the bluff (Figure 7). The position of the poles along the cable and cable plunge angles were measured along with the plunge angles of the poles themselves both in an east west orientation and separately in a north south orientation. The distance the pole projected above the surface was also measured (Figure 7).

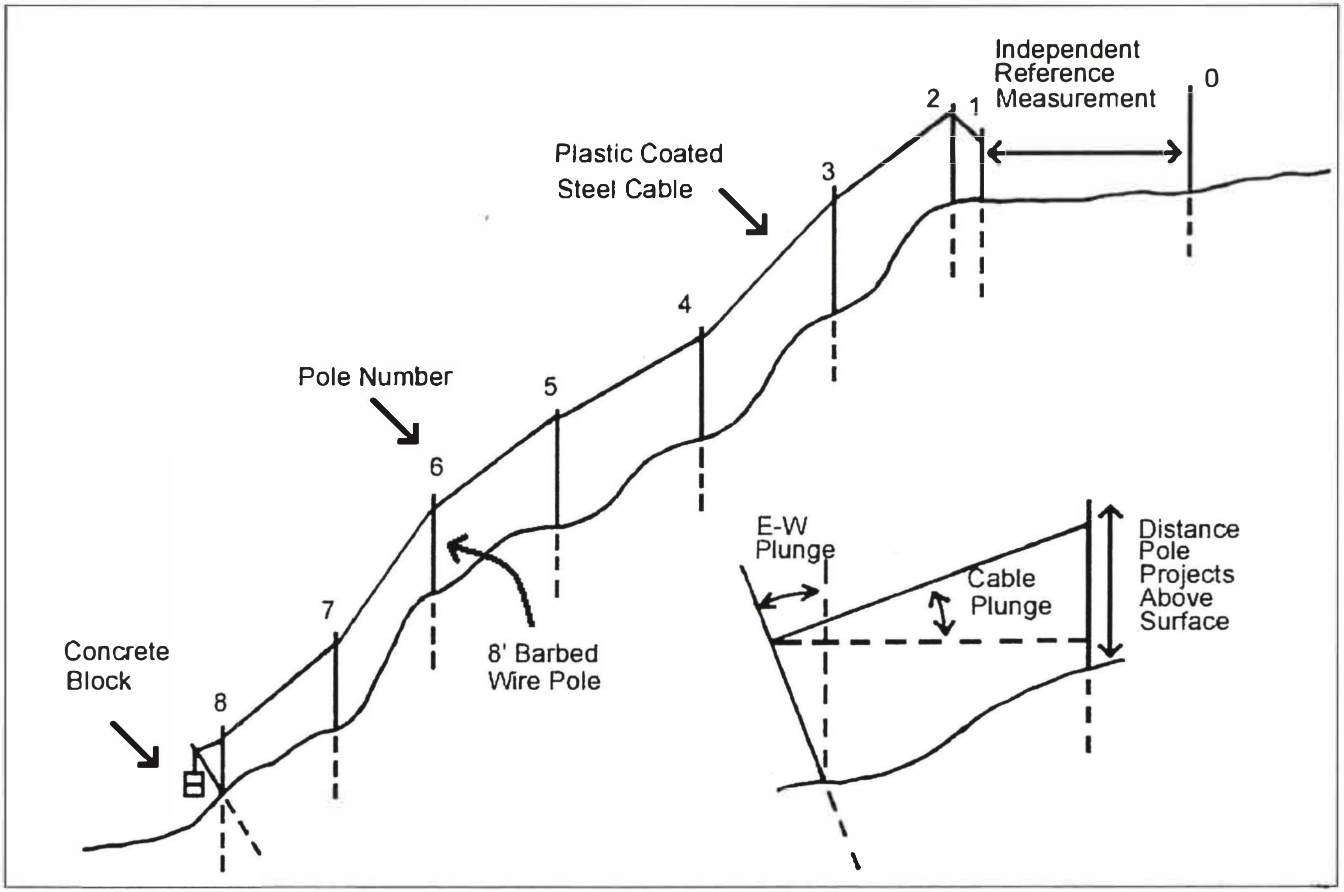


Figure 7

DIAGRAM OF POLE AND CABLE SYSTEM

Laboratory Methods

Slopes are typically evaluated for failure potential by determining the Factor of Safety using a limit equilibrium analysis. The Factor of Safety is a ratio of the sum of forces resisting failure divided by the sum of forces driving failure. When the Factor of Safety is greater than one, the slope is stable. Failure is imminent when the Factor of Safety is equal to one and Factors of Safety less than one indicate that the slope is in the process of failing or metastable, as occurs in slow moving failures such as creep (Anderson and Howes, 1985).

Limit equilibrium analysis involves calculation of the Factor of Safety of a slope along an anticipated failure surface. Various methods of limit equilibrium analysis exist and make different assumptions of soil properties and/or the forces acting within the slope. This is necessary because the analysis involves more unknowns than equations. Common among the majority of limit equilibrium analysis methods is the division of the slope into vertical slices. The number of slices chosen is large enough to satisfy two constraints. First, that the base of the slice involves only one of the various soils that compose the slope. Second, that the slices are narrow enough so that the bases of the slices can reasonably be idealized as straight lines. Potential for failure is then computed for each slice. Under most conditions for a given slope, some slices promote failure and some slices resist failure. The forces are summed up for each slice and a final Factor of Safety for the slope, along an anticipated failure surface, is computed

(Duncan, 1996). The assumptions and formulae for slope stability that were used for this project are discussed in the Appendix.

The GALENA software program determined the Factor of Safety (Clover Technology, 1999) and an analysis of movement causes and effects was produced. Limit equilibrium calculation is a tedious process and use of computer software is ideal. The Factor of Safety for a range of possible slip surfaces can be computed in a very short time after the soil properties, slope geometry and stratigraphy are entered into the program. GALENA allows the user to use one of three limit equilibrium analysis methods: the Bishop Simplified Method (Bishop, 1955), the Spencer-Wright Method (Spencer, 1967) or the Sarma Method (Sarma, 1973). Further, each of these methods can be used in either a single type of analysis (reverse modeling) or a multiple type of analysis (forward modeling). In reverse modeling, one failure surface is input by the user and GALENA calculates the Factor of Safety for that failure surface. In forward modeling, the user selects an initial failure surface and based on a range of possibilities from the initial failure surface, GALENA calculates the failure surface with the lowest Factor of Safety.

The software requires inputs that include surface geometry, stratigraphy, groundwater properties and soil properties including cohesion, angle of internal friction, plasticity index, unit weight and pore pressure ratio. Surface geometry was obtained from analysis of data obtained during the monitoring phase of the project. Stratigraphy was obtained from a combination of trenching at the site and a bluff-top rotosonic soil boring at the site during the installation of a monitoring well that is part of the next phase of the project at this location. An idealized log of the soil boring is shown in Figure 8. Groundwater locations were obtained from observations made at the site during data collection and the soil boring data. Soil properties were obtained from Montgomery (1998) (Table 3). The soils evaluated by Montgomery are similar to the soils found at Miami Park North but were collected at other nearby locations. Therefore, the geotechnical properties used in the GALENA models are reasonable but tests with soil samples collected on-site would increase the accuracy of the models.

GALENA analyses were performed using the Bishop Simplified Method (Appendix) in both the forward and reverse modeling modes. The forward modeling mode allowed GALENA to find the failure geometry with the lowest Factor of Safety. In the reverse modeling mode GALENA calculated the Factor of Safety for a given slope failure surface. The Bishop Simplified method is generally used for circular failure surfaces (Clover Technology, 1999). The GALENA analyses represented the Miami Park North primary stratigraphy with two perched aquifers above the main water table and three levels of wave erosion.

In the forward modeling mode, the main water table was evaluated at four different levels; low normal (five feet below lake level), normal (at lake level), high

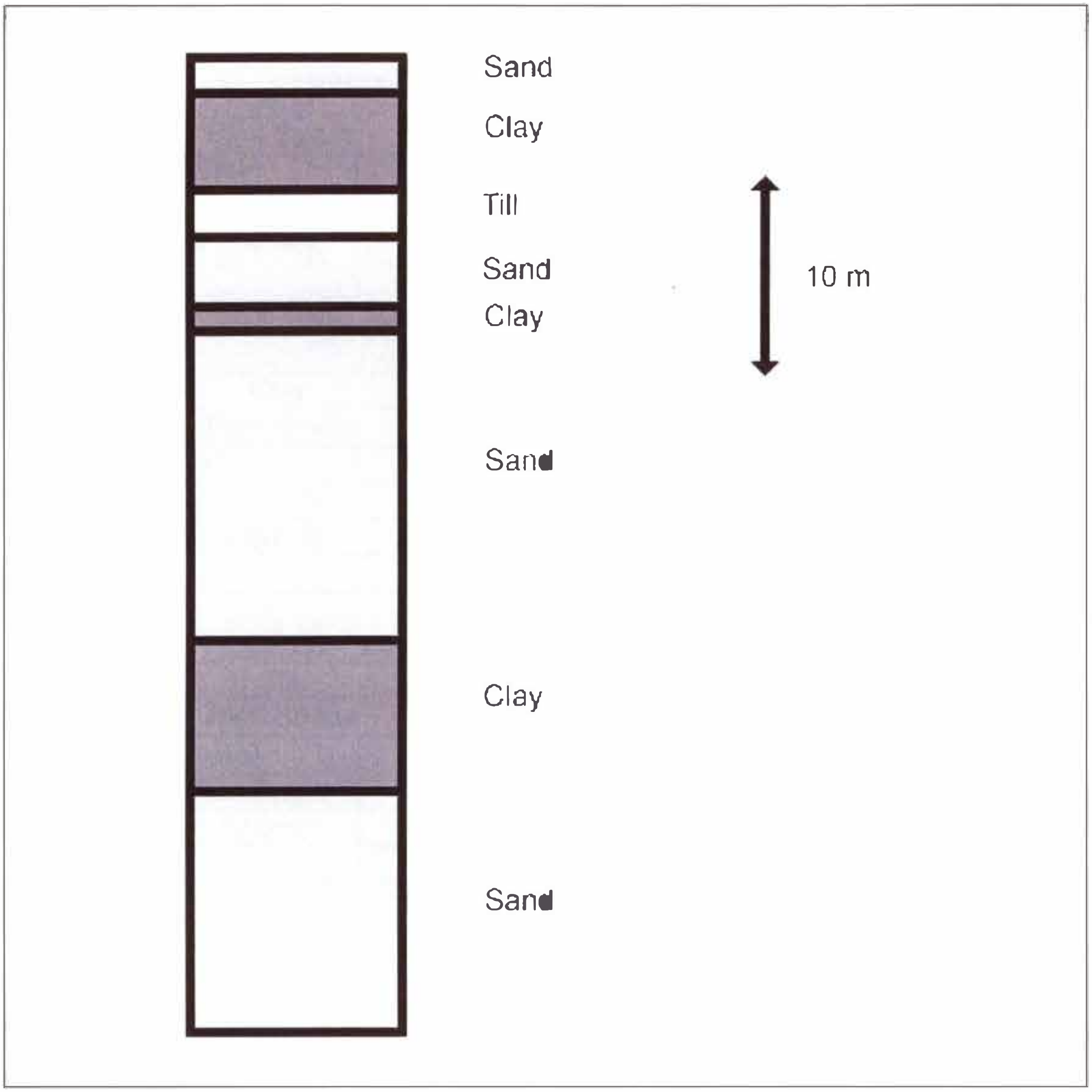


Figure 8

IDEALIZED ON-SITE ROTOSONIC SOIL BORING LOG

(Chase and Kehew, personal communication)

Sand	
Total Stress	
Cohesion (kPa)	0
Critical Failure Angle	34
Unit Weight (kN/m3)	20.4
Ru	0

Sand	
Effective Stress	
Cohesion (kPa)	0
Critical Failure Angle	34
Unit Weight (kN/m3)	22.7
Ru	0.12

Clay	
Total Stress	
Cohesion (kPa)	0.30
Critical Failure Angle	12.8
Unit Weight (kN/m3)	18.7
Ru	0

Clay	
Effective Stress	
Cohesion (kPa)	0.20
Critical Failure Angle	25.9
Unit Weight (kN/m3)	21.6
Ru	0.16

Till	
Total Stress	
Cohesion (kPa)	0.20
Critical Failure Angle	19.8
Unit Weight (kN/m3)	17.6
Ru	0

Till	
Effective Stress	
Cohesion (kPa)	0
Critical Failure Angle	34.4
Unit Weight (kN/m3)	20.97
Ru	0.18

Table 3

SOIL PROPERTIES

(Montgomery, 1998)

normal (five feet above lake level) and very high normal (ten feet above lake level). In the two perched systems at the bluff, each was evaluated at low, medium and high groundwater positions. Low represents a saturated clay layer (the aquitard), medium represents a saturated clay layer and the material above it saturated to five feet above the clay, and high represents a saturated clay layer and the material above it saturated to ten feet above the clay. A large degree of variation in the water levels was needed to produce results with a sufficiently large difference in the Factor of Safety in order to achieve noticeable changes and trends. In addition, water at the surface was evaluated in two positions; present and not present. Water present at the surface is represented by saturation at the surface and one foot down from the top of the bluff. All of the possible water combinations were then analyzed with three different toe erosion scenarios; none, moderate and severe. Moderate toe erosion is represented by removing ten feet of material at the base of the bluff horizontally. Severe toe erosion is represented by removing twenty feet of material at the base of the bluff horizontally. Again, a large degree of variation in toe erosion was needed to produce results with a sufficiently large difference in the Factor of Safety in order to achieve noticeable changes and trends.

In the reverse modeling mode, GALENA models were produced with two different shear surfaces, deep shear and shallow shear. Both deep shear and shallow shear scenarios were evaluated beginning with a completely dry state and continuing with incremental additions of water to the base aquifer and separately with incremental additions of water to the lower of the two perched aquifers. Models of the dry state were also evaluated with moderate and severe erosion of toe material. In addition, an equilibrium slope profile with a Factor of Safety of 1.000 was produced.

CHAPTER 7

RESULTS

Field Data

Of the four pole and cable lines monitored at Miami Park North, two lines, B and D, showed significant movement during the spring of 2002 and again during the spring of 2003. Lines A and C showed only minimal movement and will not be discussed further except to point out that the minimal translational movement recorded along C also occurred in the spring of 2002 and the spring of 2003 (Figures 9-16 and Tables 4-10).

Weather during both spring 2002 and 2003 consisted of warming, above freezing, temperatures that enabled melted snow left from the previous winter to filter through the soil. Precipitation for spring 2002 and 2003 averaged about three inches per month, which is about average for the season. Lake levels remained low and no storm events produced waves that reached the base of the slope and caused toe erosion (NOAA data).

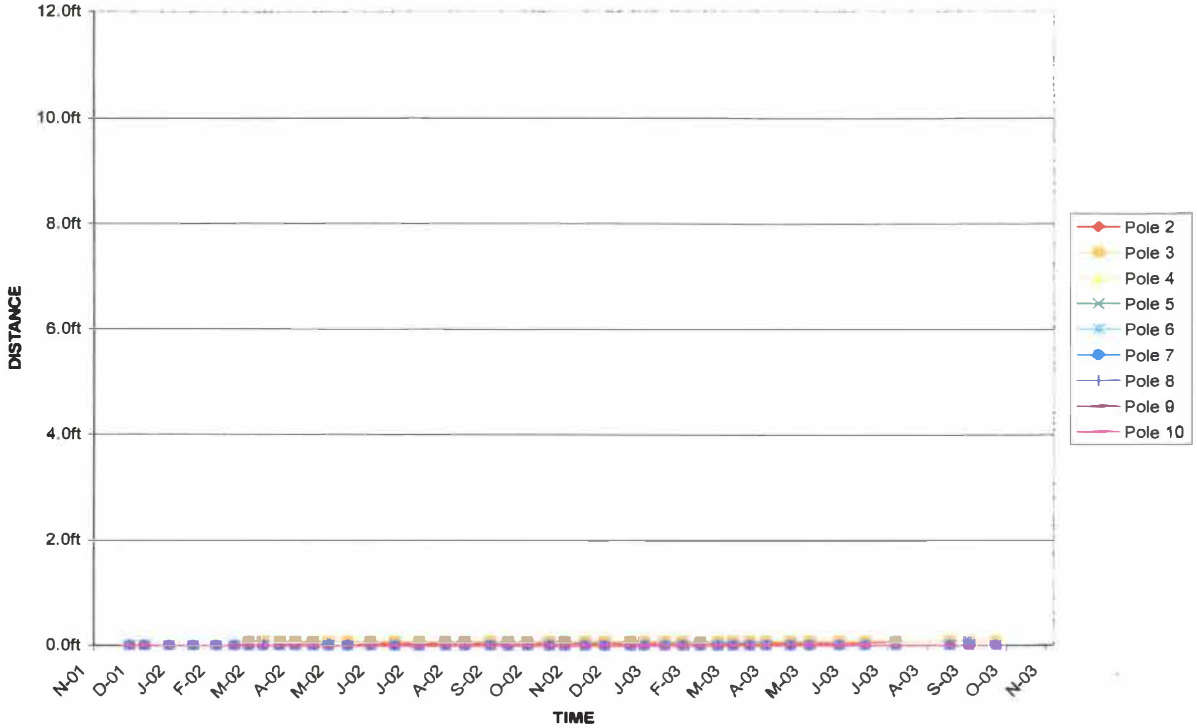


Figure 9

LINE A TRANSLATIONAL MOVEMENT

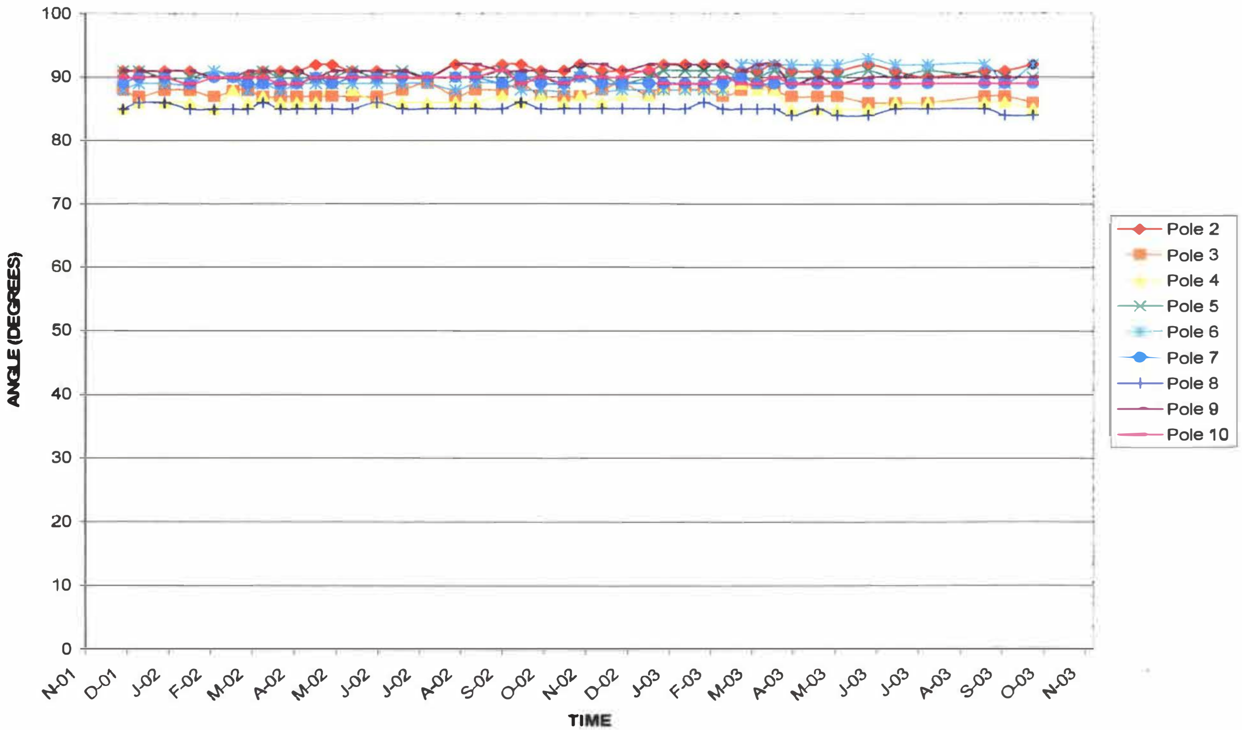


Figure 10

LINE A ROTATIONAL MOVEMENT

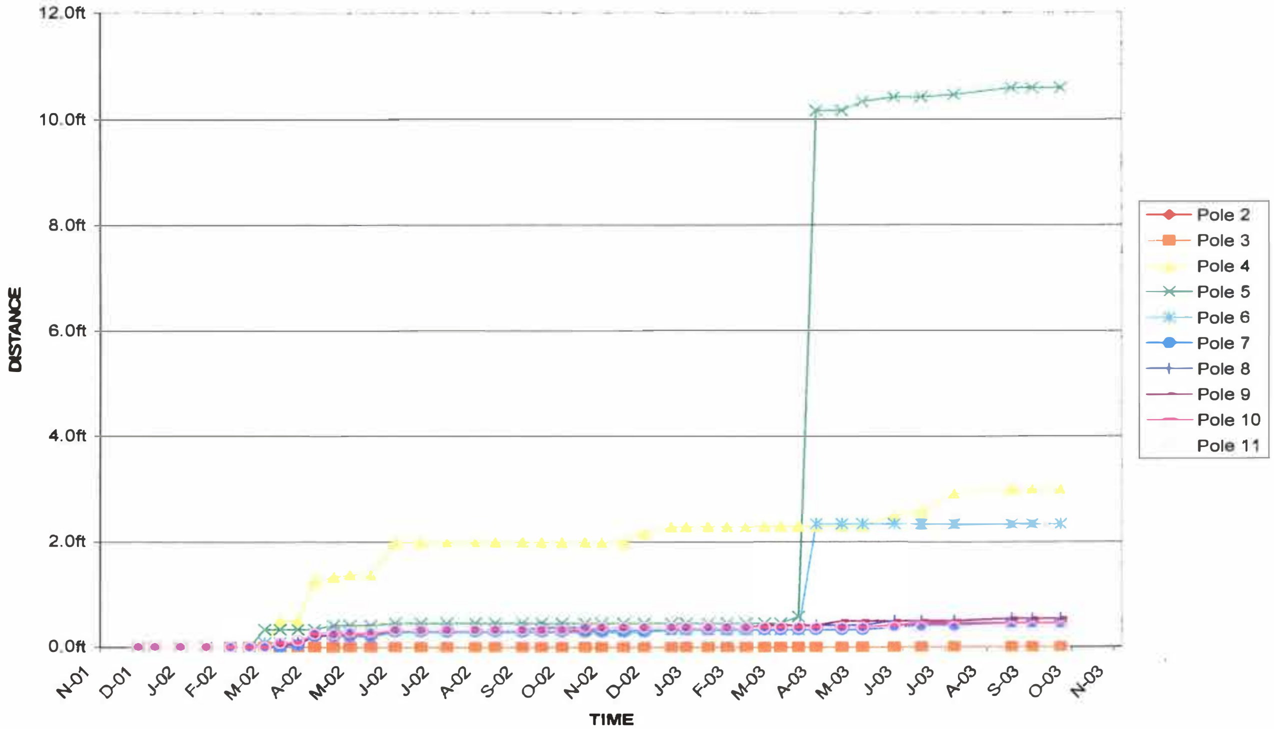


Figure 11

LINE B TRANSLATIONAL MOVEMENT

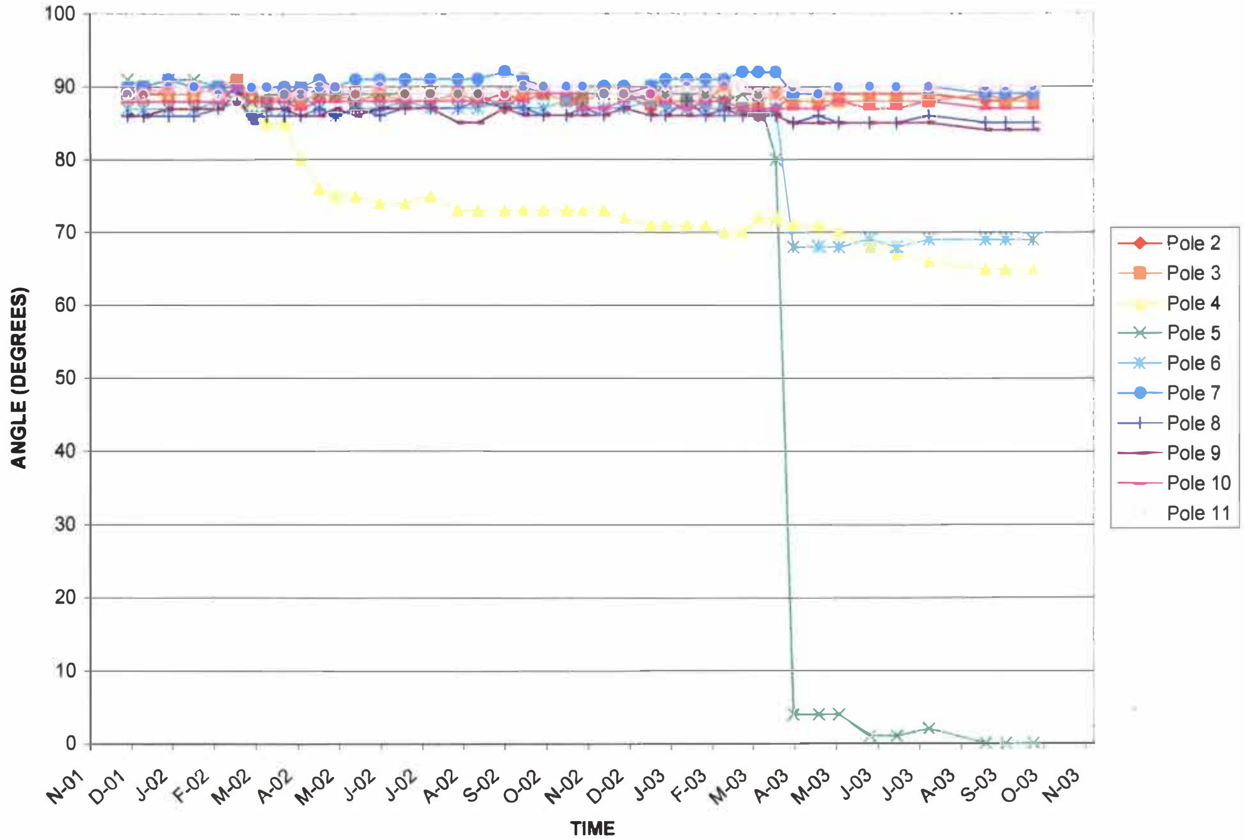


Figure 12

LINE B ROTATIONAL MOVEMENT

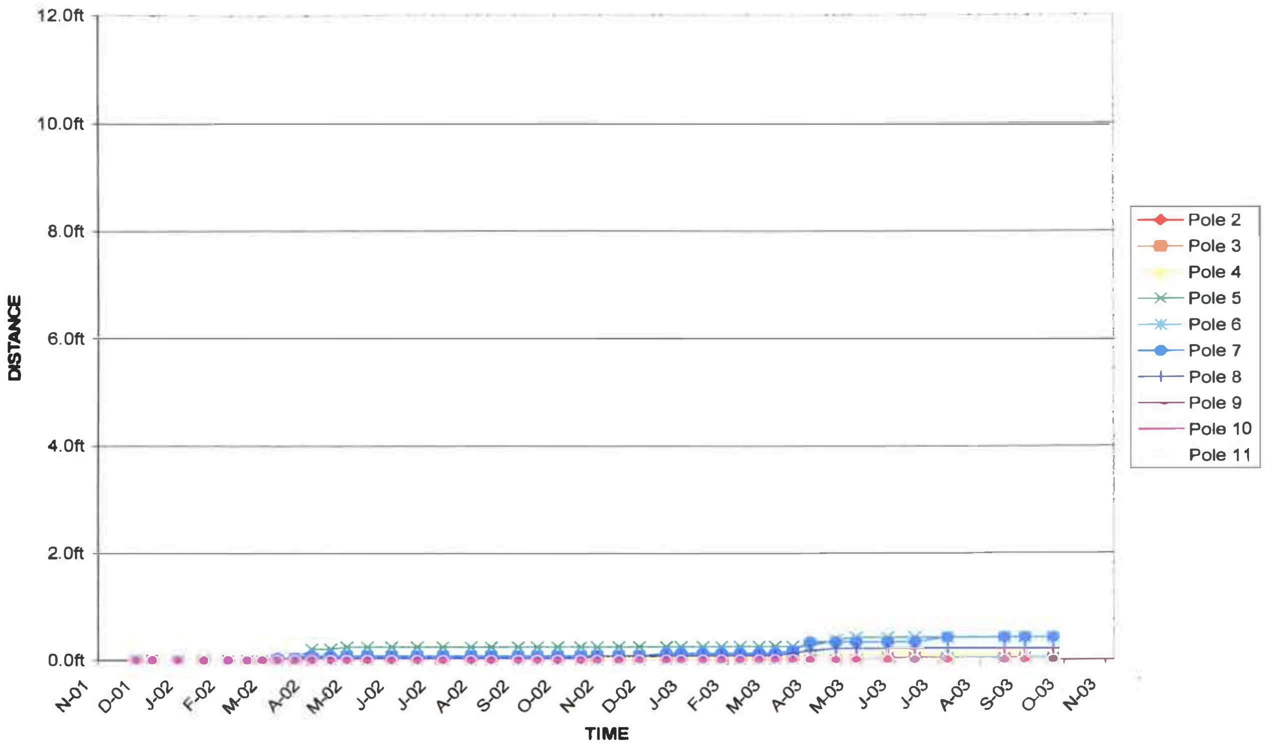


Figure 13

LINE C TRANSLATIONAL MOVEMENT

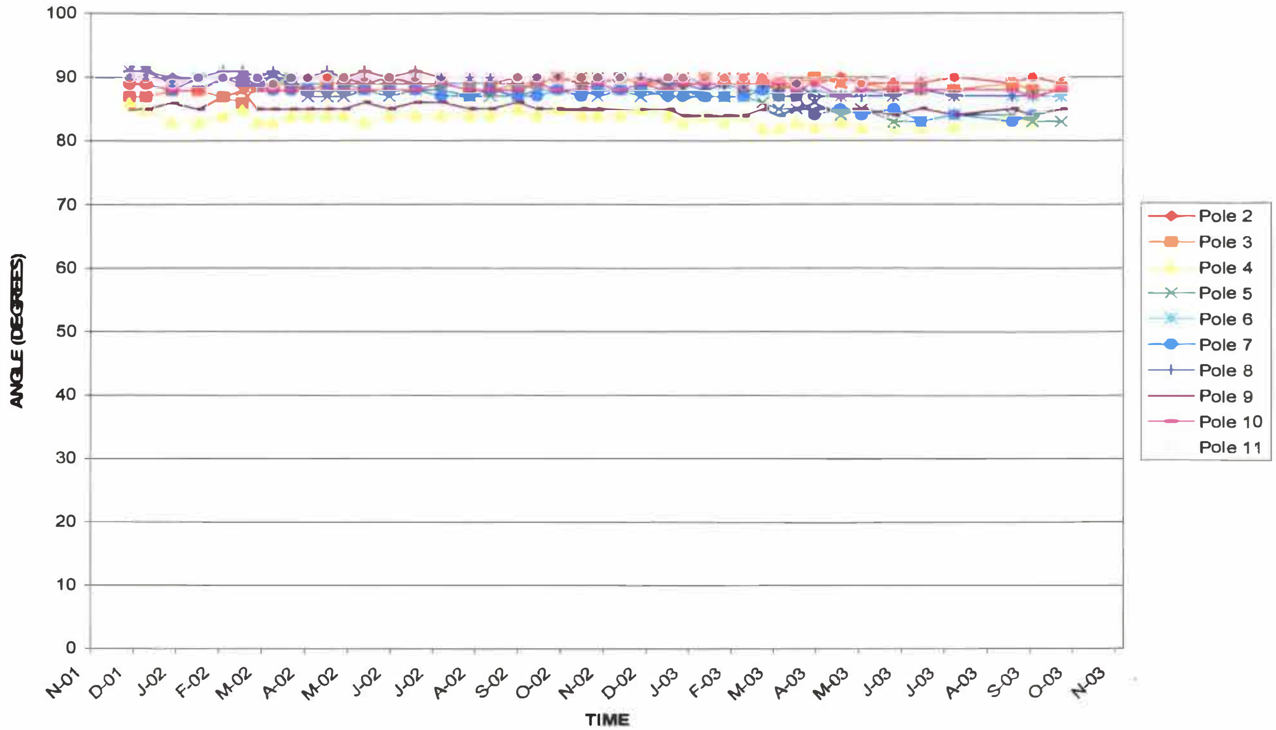


Figure 14

LINE C ROTATIONAL MOVEMENT

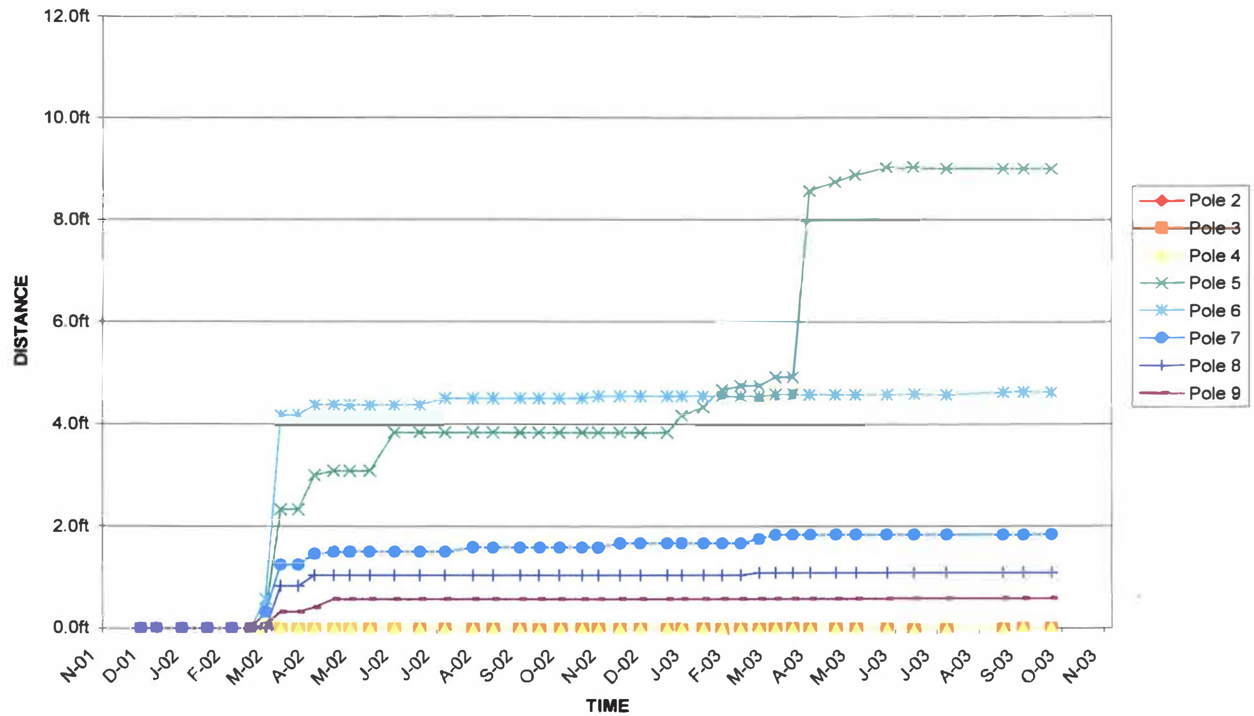


Figure 15

LINE D TRANSLATIONAL MOVEMENT

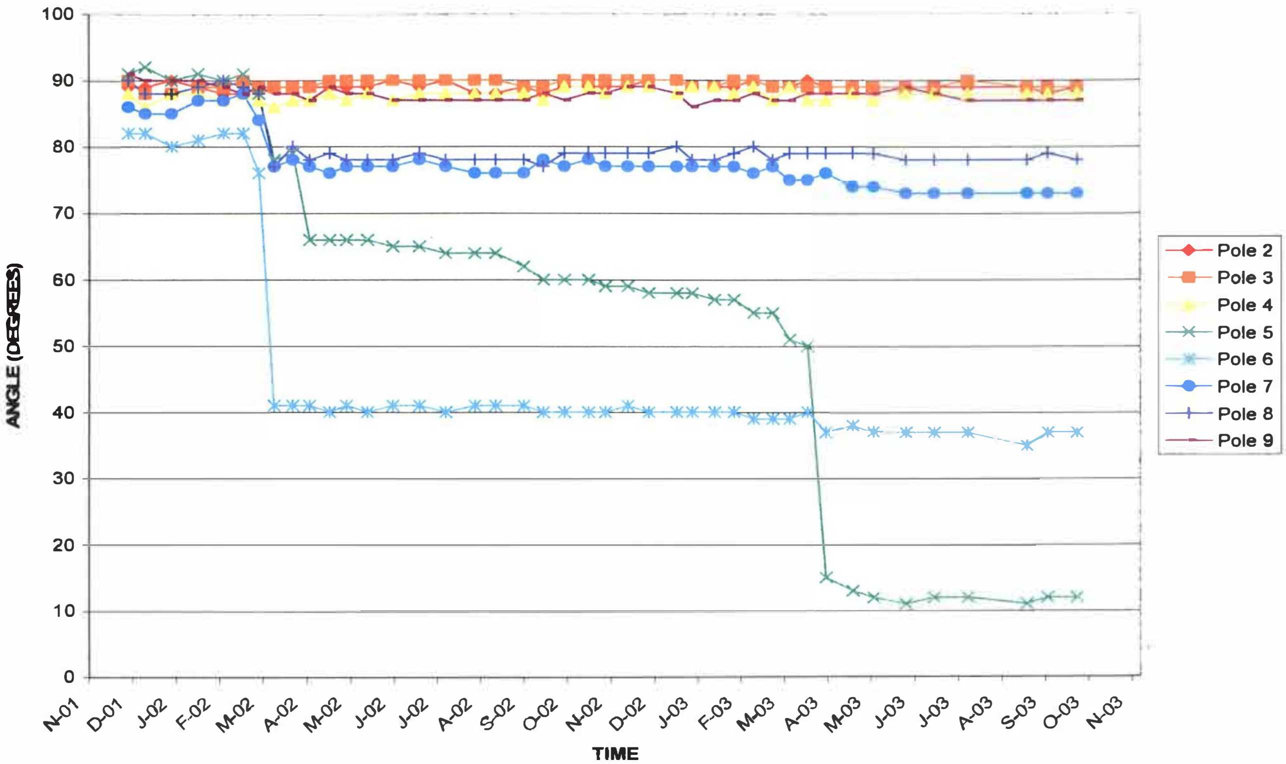


Figure 16

LINE D ROTATIONAL MOVEMENT

Date	Total Translational Movement (inches)	E-W Plunge (degrees)
12/3/2001	0	88
12/15/2001	0	88
1/3/2002	0	88
1/22/2002	0	88
2/9/2002	0	88
2/23/2002	0	89
3/6/2002	0	88
3/17/2002	6	85
3/30/2002	6	85
4/11/2002	15	80
4/25/2002	16	76
5/7/2002	16.5	75
5/22/2002	16.5	75
6/9/2002	24	74
6/28/2002	24	74
7/17/2002	24	75
8/7/2002	24	73
8/22/2002	24	73
9/11/2002	24	73
9/25/2002	24	73
10/10/2002	24	73
10/27/2002	24	73
11/8/2002	24	73
11/24/2002	24	73
12/9/2002	26	72
12/29/2002	27.5	71
1/9/2003	27.5	71
1/25/2003	27.5	71
2/8/2003	27.5	71
2/22/2003	27.5	70
3/8/2003	27.5	70
3/20/2003	27.5	72
4/2/2003	27.5	72
4/15/2003	27.5	71
5/4/2003	27.5	71
5/19/2003	27.5	70
6/11/2003	30	68
7/1/2003	31	67
7/25/2003	35	66
9/5/2003	36	65
9/20/2003	36	65
10/11/2003	36	65

Table 4

POLE MOVEMENT HISTORY: LINE B, POLE 4

Date	Total Translational Movement (inches)	E-W Plunge (degrees)
12/3/2001	0	91
12/15/2001	0	90
1/3/2002	0	91
1/22/2002	0	91
2/9/2002	0	90
2/23/2002	0	89
3/6/2002	4	88
3/17/2002	4	89
3/30/2002	4	89
4/11/2002	4	90
4/25/2002	5	89
5/7/2002	5	89
5/22/2002	5	89
6/9/2002	5.5	89
6/28/2002	5.5	89
7/17/2002	5.5	89
8/7/2002	5.5	89
8/22/2002	5.5	89
9/11/2002	5.5	89
9/25/2002	5.5	90
10/10/2002	5.5	89
10/27/2002	5.5	89
11/8/2002	5.5	89
11/24/2002	5.5	89
12/9/2002	5.5	89
12/29/2002	5.5	90
1/9/2003	5.5	89
1/25/2003	5.5	89
2/8/2003	5.5	89
2/22/2003	5.5	88
3/8/2003	5.5	89
3/20/2003	5.5	89
4/2/2003	7	80
4/15/2003	122	4
5/4/2003	122	4
5/19/2003	124	4
6/11/2003	125	1
7/1/2003	125	1
7/25/2003	125.5	2
9/5/2003	127	0
9/20/2003	127	0
10/11/2003	127	0

Table 5

POLE MOVEMENT HISTORY: LINE B, POLE 5

Date	Total Translational Movement (inches)	E-W Plunge (degrees)
12/3/2001	0	87
12/15/2001	0	87
1/3/2002	0	87
1/22/2002	0	87
2/9/2002	0	88
2/23/2002	0	89
3/6/2002	1	86
3/17/2002	1	87
3/30/2002	1	87
4/11/2002	3	89
4/25/2002	3.5	87
5/7/2002	3.5	86
5/22/2002	3.5	87
6/9/2002	4	87
6/28/2002	4	88
7/17/2002	4	87
8/7/2002	4	87
8/22/2002	4	87
9/11/2002	4	88
9/25/2002	4	87
10/10/2002	4	87
10/27/2002	4	88
11/8/2002	4	87
11/24/2002	4	87
12/9/2002	4	87
12/29/2002	4	88
1/9/2003	4	87
1/25/2003	4	87
2/8/2003	4	87
2/22/2003	4	87
3/8/2003	4	87
3/20/2003	4	86
4/2/2003	4	87
4/15/2003	28	68
5/4/2003	28	68
5/19/2003	28	68
6/11/2003	28	69
7/1/2003	28	68
7/25/2003	28	69
9/5/2003	28	69
9/20/2003	28	69
10/11/2003	28	69

Table 6

POLE MOVEMENT HISTORY: LINE B, POLE 6

Date	Total Translational Movement (inches)	E-W Plunge (degrees)
12/3/2001	0	91
12/15/2001	0	92
1/3/2002	0	90
1/22/2002	0	91
2/9/2002	0	90
2/23/2002	0	91
3/6/2002	3.5	88
3/17/2002	28	78
3/30/2002	28	79
4/11/2002	36	66
4/25/2002	37	66
5/7/2002	37	66
5/22/2002	37	66
6/9/2002	46	65
6/28/2002	46	65
7/17/2002	46	64
8/7/2002	46	64
8/22/2002	46	64
9/11/2002	46	62
9/25/2002	46	60
10/10/2002	46	60
10/27/2002	46	60
11/8/2002	46	59
11/24/2002	46	59
12/9/2002	46	58
12/29/2002	46	58
1/9/2003	46	58
1/25/2003	50	57
2/8/2003	52	57
2/22/2003	56	55
3/8/2003	57	55
3/20/2003	57	51
4/2/2003	59	50
4/15/2003	59	15
5/4/2003	103	13
5/19/2003	105	12
6/11/2003	106.5	11
7/1/2003	108	12
7/25/2003	108	12
9/5/2003	108	11
9/20/2003	108	12
10/11/2003	108	12

Table 7

POLE MOVEMENT HISTORY: LINE D, POLE 5

Date	Total Translational Movement (inches)	E-W Plunge (degrees)
12/3/2001	0	82
12/15/2001	0	82
1/3/2002	0	80
1/22/2002	0	81
2/9/2002	0	82
2/23/2002	0	82
3/6/2002	7	76
3/17/2002	50	41
3/30/2002	50	41
4/11/2002	52.5	41
4/25/2002	52.5	40
5/7/2002	52.5	41
5/22/2002	52.5	40
6/9/2002	52.5	41
6/28/2002	52.5	41
7/17/2002	53	40
8/7/2002	54	41
8/22/2002	54	41
9/11/2002	54	41
9/25/2002	54	40
10/10/2002	54	40
10/27/2002	54	40
11/8/2002	54	40
11/24/2002	54.5	41
12/9/2002	54.5	40
12/29/2002	54.5	40
1/9/2003	54.5	40
1/25/2003	54.5	40
2/8/2003	54.5	40
2/22/2003	54.5	39
3/8/2003	54.5	39
3/20/2003	54.5	39
4/2/2003	55	40
4/15/2003	55	37
5/4/2003	55	38
5/19/2003	55	37
6/11/2003	55	37
7/1/2003	55	37
7/25/2003	55	37
9/5/2003	55	35
9/20/2003	55.5	37
10/11/2003	55.5	37

Table 8

POLE MOVEMENT HISTORY: LINE D, POLE 6

Date	Total Translational Movement (inches)	E-W Plunge (degrees)
12/3/2001	0	86
12/15/2001	0	85
1/3/2002	0	85
1/22/2002	0	87
2/9/2002	0	87
2/23/2002	0	88
3/6/2002	4	84
3/17/2002	15	77
3/30/2002	15	78
4/11/2002	17.5	77
4/25/2002	18	76
5/7/2002	18	77
5/22/2002	18	77
6/9/2002	18	77
6/28/2002	18	78
7/17/2002	18	77
8/7/2002	19	76
8/22/2002	19	76
9/11/2002	19	76
9/25/2002	19	78
10/10/2002	19	77
10/27/2002	19	78
11/8/2002	19	77
11/24/2002	20	77
12/9/2002	20	77
12/29/2002	20	77
1/9/2003	20	77
1/25/2003	20	77
2/8/2003	20	77
2/22/2003	20	76
3/8/2003	21	77
3/20/2003	22	75
4/2/2003	22	75
4/15/2003	22	76
5/4/2003	22	74
5/19/2003	22	74
6/11/2003	22	73
7/1/2003	22	73
7/25/2003	22	73
9/5/2003	22	73
9/20/2003	22	73
10/11/2003	22	73

Table 9

POLE MOVEMENT HISTORY: LINE D, POLE 7

Date	Total Translational Movement (inches)	E-W Plunge (degrees)
12/3/2001	0	90
12/15/2001	0	88
1/3/2002	0	88
1/22/2002	0	89
2/9/2002	0	90
2/23/2002	0	89
3/6/2002	0	88
3/17/2002	10	77
3/30/2002	10	80
4/11/2002	12.5	78
4/25/2002	12.5	79
5/7/2002	12.5	78
5/22/2002	12.5	78
6/9/2002	12.5	78
6/28/2002	12.5	79
7/17/2002	12.5	78
8/7/2002	12.5	78
8/22/2002	12.5	78
9/11/2002	12.5	78
9/25/2002	12.5	77
10/10/2002	12.5	79
10/27/2002	12.5	79
11/8/2002	12.5	79
11/24/2002	12.5	79
12/9/2002	12.5	79
12/29/2002	12.5	80
1/9/2003	12.5	78
1/25/2003	12.5	78
2/8/2003	12.5	79
2/22/2003	12.5	80
3/8/2003	13	78
3/20/2003	13	79
4/2/2003	13	79
4/15/2003	13	79
5/4/2003	13	79
5/19/2003	13	79
6/11/2003	13	78
7/1/2003	13	78
7/25/2003	13	78
9/5/2003	13	78
9/20/2003	13	79
10/11/2003	13	78

Table 10

POLE MOVEMENT HISTORY: LINE D, POLE 8

During the spring of 2002, Line B had significant translational and rotational movement of pole four and minor translational movement along poles five, six, seven, eight, nine, and ten.

Pole four, situated in the till material above the upper clay layer, exhibited translational movement along the cable that began in the early spring of 2002 and continued to late spring 2002. During this time pole four moved a total of two feet along the cable. In spring 2002 rotational movement was also recorded at pole four. The east-west plunge of pole four at line B decreased from 88° east of vertical to 74° east of vertical.

Also during the spring of 2002, translational movement was recorded along line B at pole five, positioned within the upper clay layer, totaling five and one half inches. Four inches of translational movement was recorded at pole six, located at the top of the lower clay perched aquitard. At pole seven movement in this manner totaled three inches, and movement at the locations of poles eight, nine and ten each totaled four inches in a translational manner. Poles seven, eight, nine and ten were all positioned within the large sand deposit at the base of the slope.

Movement during the spring of 2002 along line D occurred primarily at poles five and six although poles seven and eight also displayed movement both in a translational and rotational manner. Pole nine, the last pole along line D, had minor movement in the translational sense.

During the spring of 2002, translational movement measured along pole five, situated at the base of the upper clay layer, was three foot ten inches. At pole six, situated at the base of the middle clay layer, translational movement was four and one half feet along the cable. Both poles also had movement of the pole angle in the east-west orientation during this period. Pole five began the spring of 2002 with a plunge of 88° east of vertical and by the end of spring the plunge was 65° east of vertical. During the late winter of 2002, the measured plunge at pole six was 82° east. By early spring, the plunge was 41° east.

Poles seven, eight and nine along line D were all located within the large sand deposit at the base of the slope. Movement was recorded at pole seven at this time of one and one half feet in the translational sense and the plunge decreased from 88° east to 76° east. Pole eight displayed translational movement totaling one foot and the plunge decreased ten degrees from 88° east of vertical to 78°. Pole nine had translational movement of seven inches during this period.

Between the spring of 2002 and the spring of 2003, two poles exhibited movement worth noting. Along line B, pole four had minor, irregular movement in the rotational sense. Pole four finished the spring of 2002 with a plunge of 74°

east of vertical. At the end of the measurement phase of the project, in October 2003, the plunge was 65° east. In the translational sense, this same pole had movement during the winter of 2002/2003 totaling three and one half inches. Along line D, pole five had steady rotational movement beginning the spring of 2002 with a plunge of 65° east and ending in the spring of 2003, before a major movement event, with a plunge of 50° east. In addition, pole five along line D exhibited regular translational movement in the four months preceding the same major movement event. Between January 2003 and April 2003, pole five moved, at a regular pace, a total of thirteen inches.

During the spring of 2003, lines B and D again had major movement events. Pole five along line B, which previously had been associated with only minor movement, exhibited the greatest movement episode displayed in the monitoring phase of the project. On April 2, 2003, pole five had a measured translational change of one and one half inches and a rotational change of nine degrees from the previous measurement, thirteen days earlier. At the time of the next measurement on April 15, 2003, pole five had moved in the translational sense an additional nine feet seven inches and the plunge had decreased from 80° east of vertical to 4° east of vertical. Also on April 15, 2003, pole six of line B had moved in the translational sense two feet since the last measurement and the rotational measurement had changed nineteen degrees from 87° east of vertical to 68° east of vertical.

An additional major movement event was recorded along line D nineteen days later. Pole five of line D, which had been measured to have had incremental translational and rotational movement as mentioned previously, moved a total of three foot eight inches between April 15, 2003 and May 4, 2003. In addition, during this same time span the plunge of pole five decreased from 50° east of vertical to 15° east of vertical.

Also of note was translational and rotational movement recorded along line B at poles four and five in the summer of 2003. Pole four, throughout the summer, displayed regular movement that totaled nine and one half inches in the translational sense and a decrease in the plunge angle of five degrees. Pole five, during this same time, had a total of five inches translational movement and a four degree decrease in the plunge angle.

Laboratory Data

GALENA analyses, in the reverse modeling mode, were performed in both deep shear and shallow shear scenarios with increasing amounts of water added to the basal aquifer and separately to the lower of the two perched aquifers (Table 11). In addition, dry bluffs were evaluated with either no erosion of toe material, moderate toe erosion or severe toe erosion in both shallow shear and deep shear scenarios. An equilibrium profile was also produced (Figures 17-23). The deep shear scenario is similar to a massive failure that occurred approximately five hundred meters south of the study site (Chase et al., 1999). When GALENA was used in the forward modeling mode, also with different combinations of water within the bluff and erosion of toe material, the shallow shear scenario was the dominant predicted failure geometry.

In the deep shear scenario, a completely dry bluff produced a Factor of Safety of 1.087 indicating a stable condition, but only slightly (Figure 17). Nearly twenty feet of water would need to be added to the base aquifer to reach a Factor of Safety below one and produce a deep shear failure (Table 11). When the base of the bluff was left dry but water was incrementally added to the lower of the two perched aquifers to the top of the bluff, no amount of water produced a Factor of Safety below one (Table 11).

A completely dry bluff in a shallow shear scenario produced a Factor of Safety of 0.829 indicating a failed slope condition (Figure 20). Addition of water to the base of the slope further decreased the Factor of Safety (Table 11). Similarly, when the base of the bluff was left dry and water was incrementally added to the lower of the two perched aquifers, a failed slope condition with a Factor of Safety lower than one was always produced (Table 11). Due to the strength of the clay, models of wet systems produced a small Factor of Safety increase averaging 0.032 where the water level increase included a clay layer.

	Deep Shear		Shallow Shear
Location of Water Table	**Factor of Safety**	**Material at Upper Limit of Water Table**	**Factor of Safety**
Dry	1.087	N/A	0.829
Base Aquitard	1.091	Clay	0.826
Base + 5'	1.056	Sand	0.820
Base + 10'	1.033	Sand	0.799
Base + 15'	1.014	Sand	0.775
Base + 20'	0.995	Sand	0.755
Perched Aquitard	1.101	Clay	0.843
Perched + 5'	1.086	Sand	0.822
Perched + 10'	1.072	Sand	0.803
Perched + 12.5'	1.066	Sand	0.796
Perched + 17.5'	1.067	Clay	0.805
Perched + 22.5'	1.060	Till	0.801
Perched + 27.5'	1.055	Till	0.800
Perched + 30'	1.053	Till	0.800
Perched + 35'	1.051	Sand	0.800

Table 11

GALENA FACTOR OF SAFETY CALCULATIONS

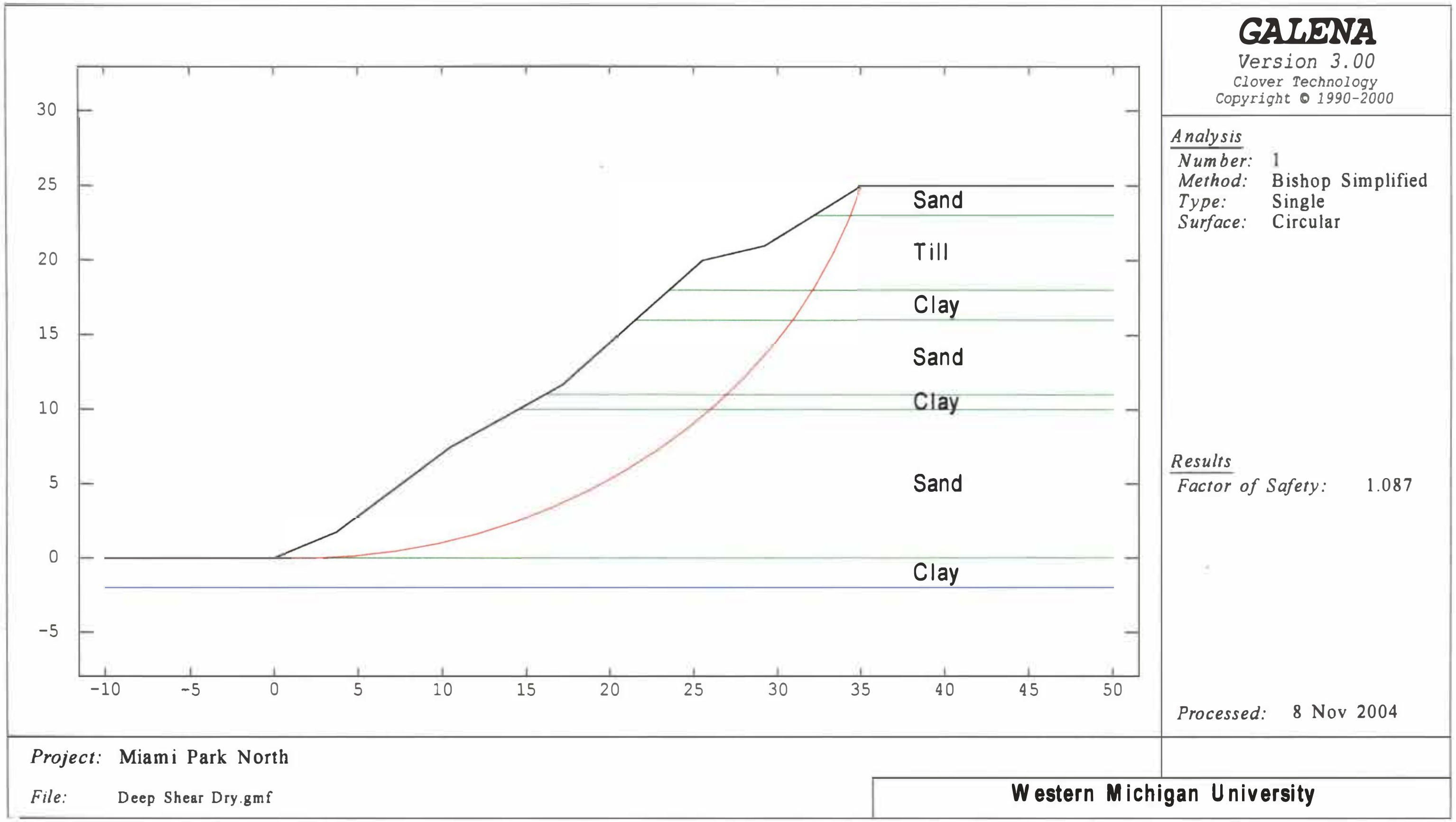
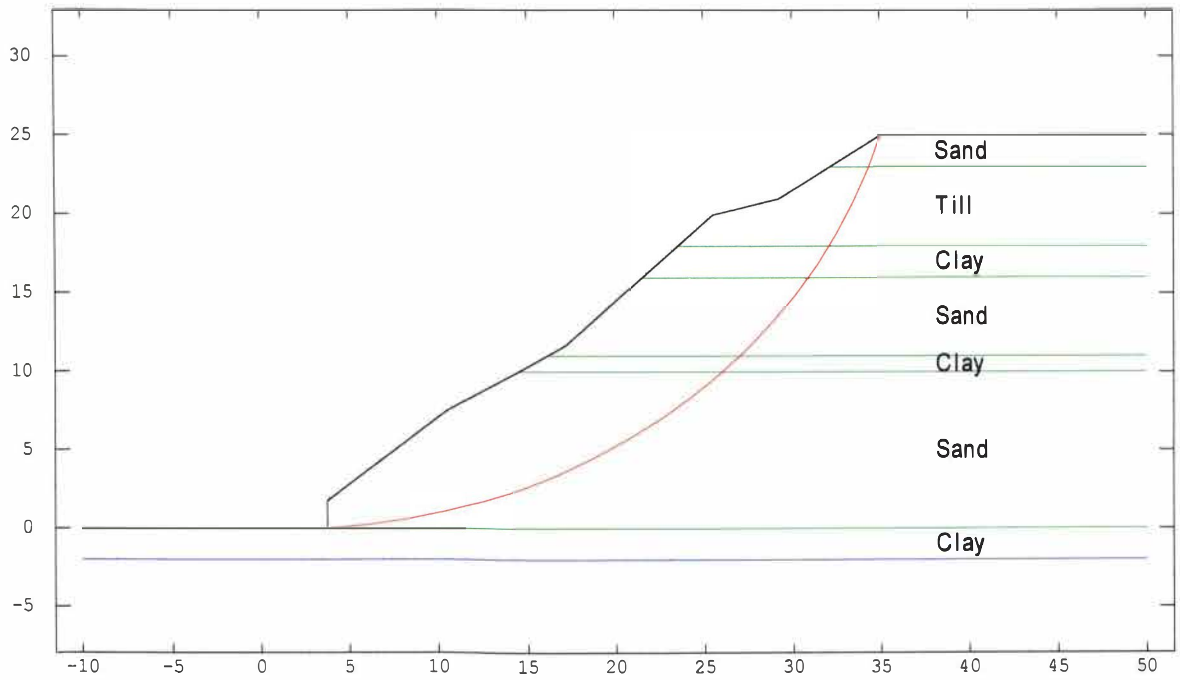


Figure 17

DEEP SHEAR DRY MODEL



GALENA
Version 3.00
Clover Technology
Copyright © 1990-2000

Analysis
Number: 1
Method: Bishop Simplified
Type: Single
Surface: Circular

Results
Factor of Safety: 1.071

Processed: 8 Nov 2004

Project: Miami Park North

File: Deep Shear Dry - Moderate Erosion.gmf

Western Michigan University

Figure 18

DEEP SHEAR DRY, MODERATE EROSION MODEL

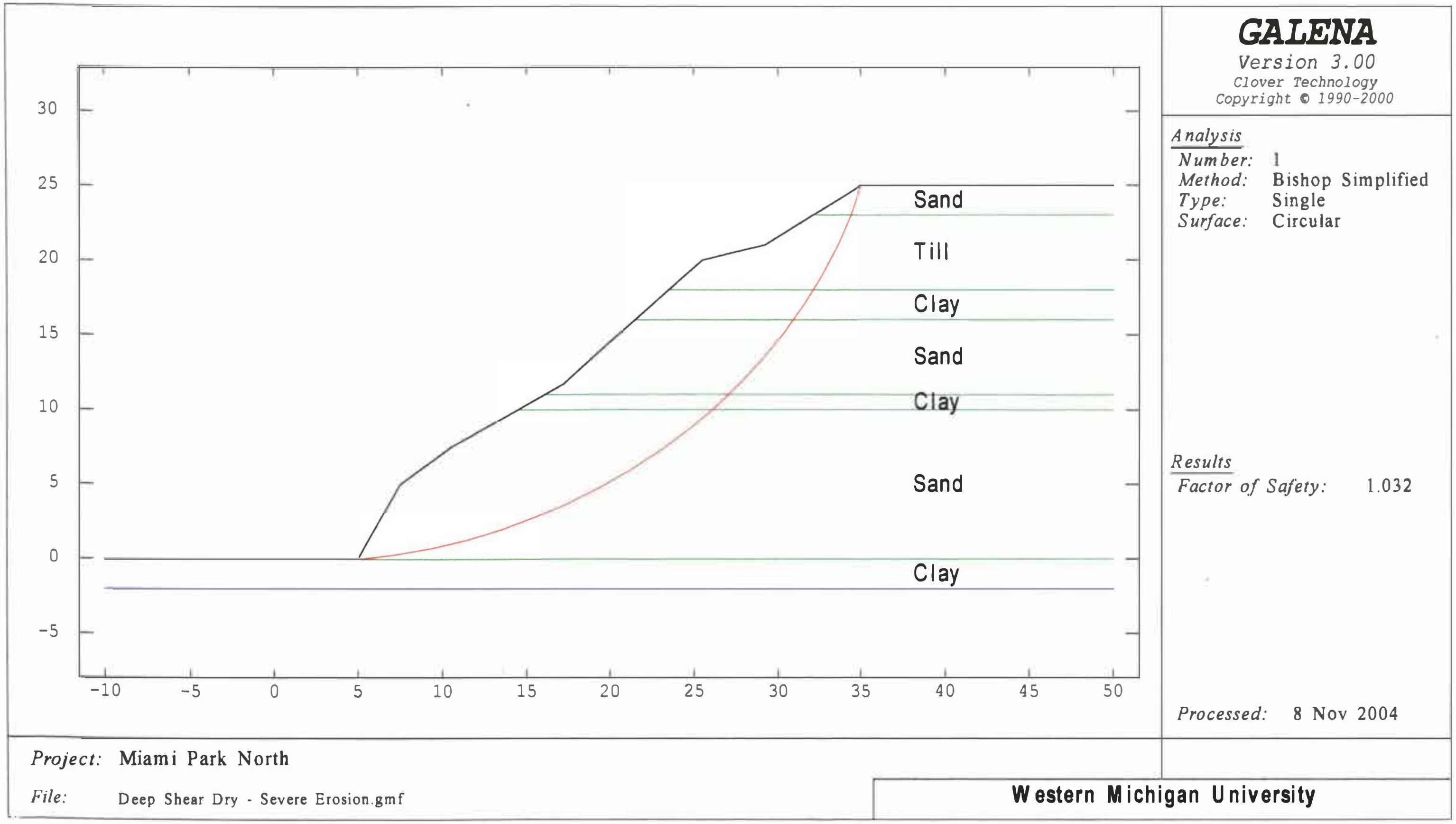


Figure 19

DEEP SHEAR DRY, SEVERE EROSION MODEL

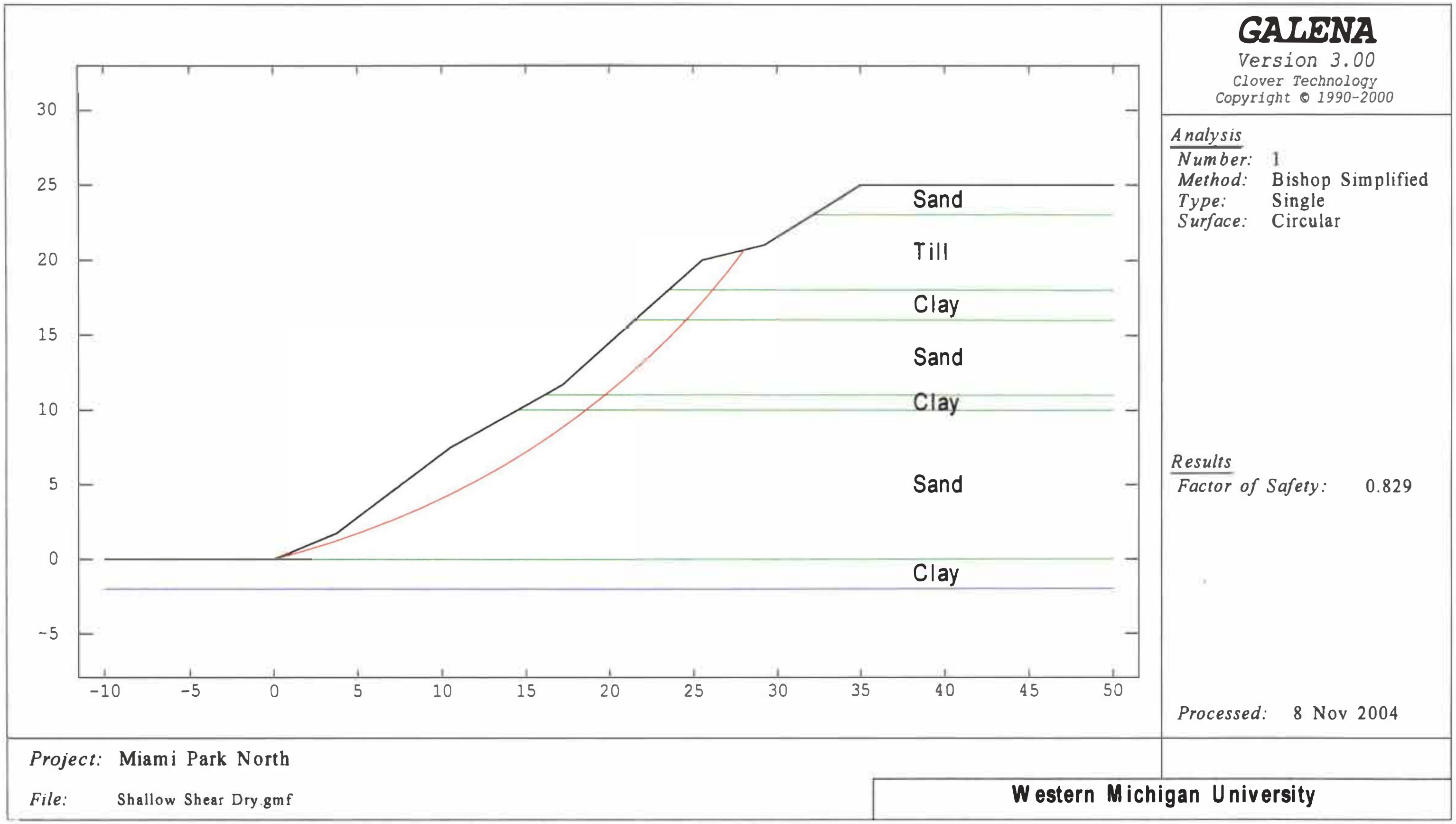


Figure 20

SHALLOW SHEAR DRY MODEL

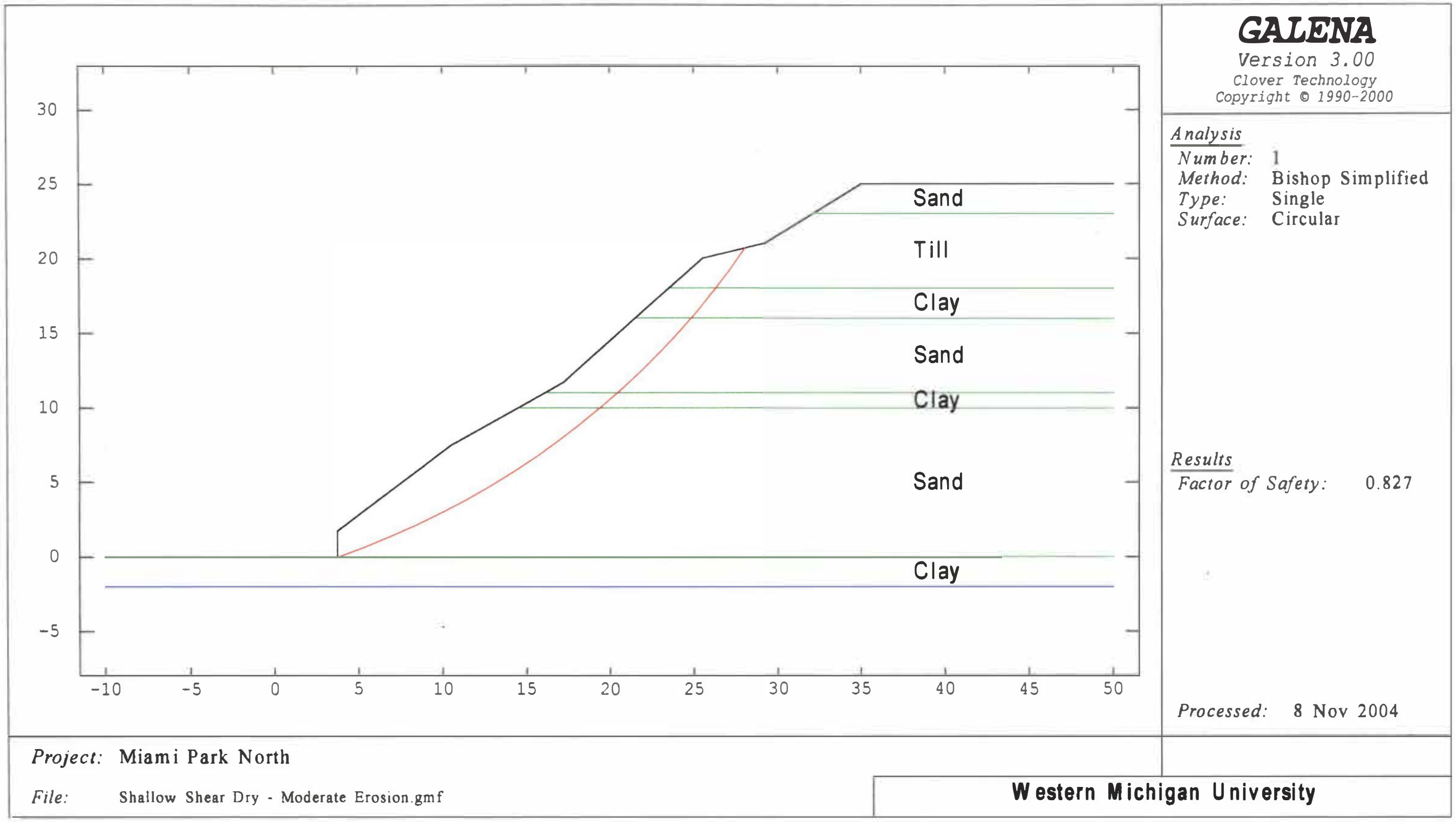


Figure 21

SHALLOW SHEAR DRY, MODERATE EROSION MODEL

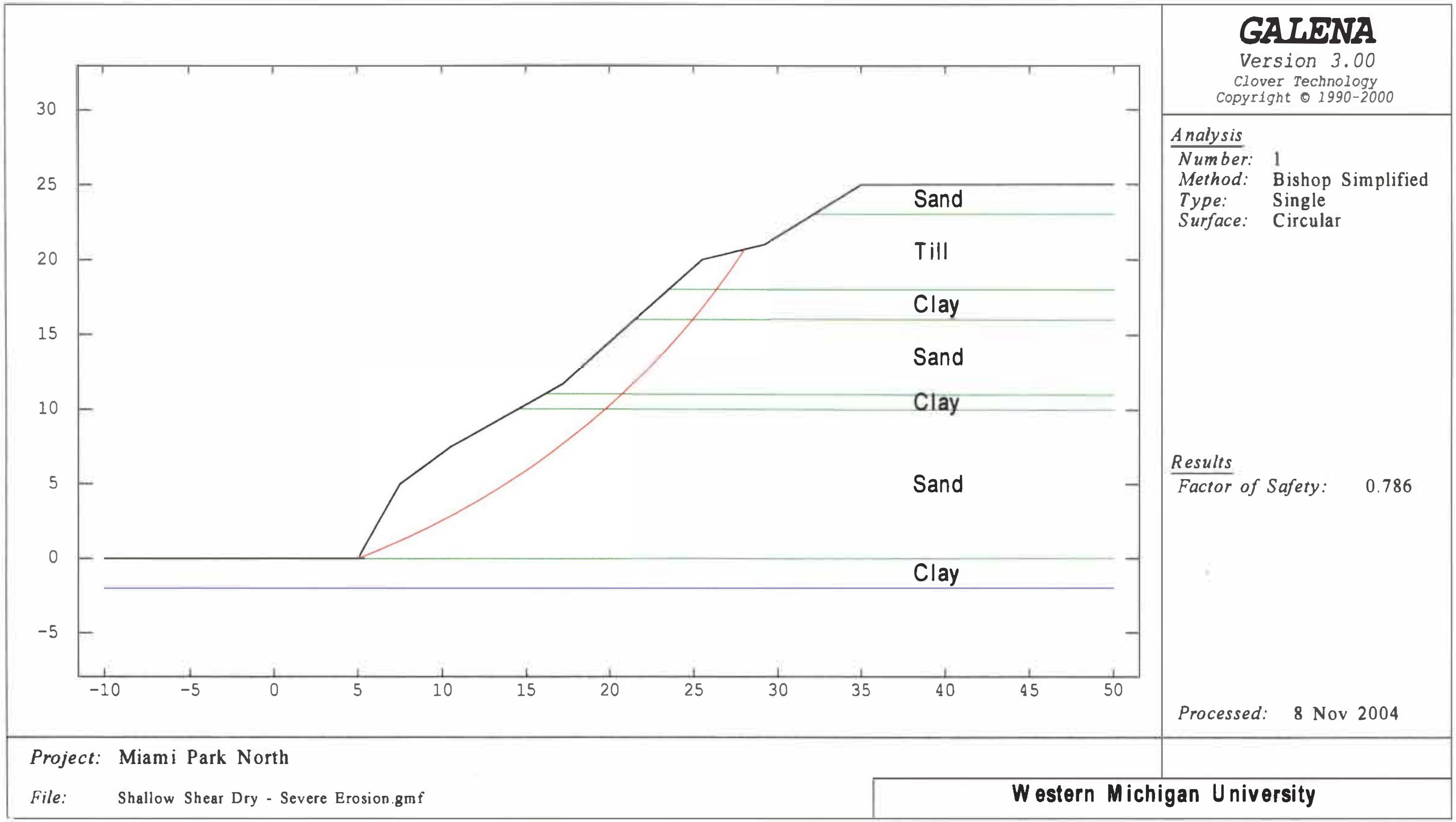


Figure 22

SHALLOW SHEAR DRY, SEVERE EROSION MODEL

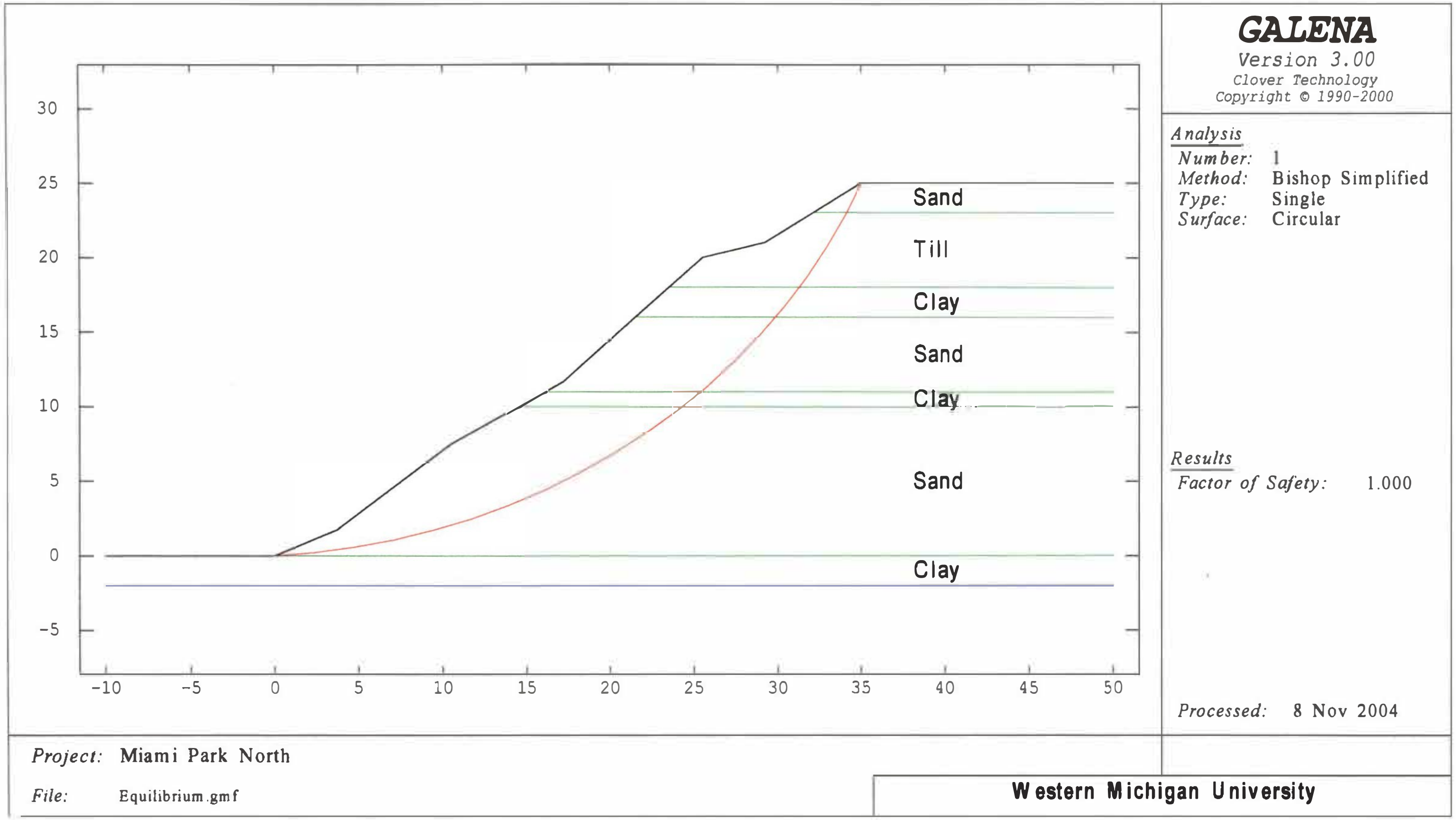


Figure 23

EQUILIBRIUM MODEL

Proceeding from a dry state, increasing amounts of toe erosion produced decreasing Factors of Safety. In the deep shear scenario, no toe erosion in a completely dry bluff produced a Factor of Safety of 1.087 (Figure 17). When toe erosion was modeled to proceed to moderate and severe, the Factor of Safety decreased to 1.071 at the moderate stage and to 1.032 in the severe stage (Figures 18 and 19). In the shallow shear scenario, no erosion of toe material produced a Factor of Safety in a completely dry bluff of 0.829 (Figure 20). The Factor of Safety with moderate toe erosion was 0.827 (Figure 21) and severe erosion produced a Factor of Safety of 0.786 (Figure 22).

The equilibrium scenario with a Factor of Safety of exactly one was produced in a model with a failure surface between the deep shear scenarios and the shallow shear scenarios (Figure 23).

CHAPTER 8

ANALYSIS

Terzaghi (1950) grouped mass movements into internal and external categories. At the study site, internal processes correspond to increased pore pressure acting within the perched aquifers and external processes correspond to wave erosion at the base of the slope. As of October 2003, the internal category seems to be the condition that is promoting failure. The failures at the site have been confined to the upper levels of the bluff and removal of toe material due to wave erosion did not occur during the monitoring phase of the project. The measured movement is attributed to increased pore pressure within the cohesive and non-cohesive soils above the clay layers.

Future significant bluff failures at the site are forecast to be influenced by the stratigraphic succession that includes the alternating layers of sand acting as perched aquifers and the impermeable clay acting as the associated aquitard. Increased pore water pressure in the perched aquifers weakens the soil and changes the mechanical properties. Water reduces the strength of the soil by adding weight and reducing friction between the soil particles.

GALENA models that had deep shear failure surfaces always produced Factors of Safety above one, even when modeled with severe erosion of the toe.

Models with shallow shear failure surfaces always produced Factors of Safety below one regardless whether erosion of toe material was present or not. This coincides with the movement taking place at the bluff during the monitoring phase of the project. The bluff regularly experiences episodes of shallow failure. The movement style of the poles during the two year monitoring phase is evidence of shallow failure surfaces. The geometry of the pole movements cannot be demonstrated to have occurred with any means of subsurface movement other than shallow shear. Furthermore, it has been shown that the movement history of the poles occurs primarily in the winter and spring seasons.

Analysis of the models produced suggests that while erosion of toe material at the base of the bluff may play a role if it occurs, it is not expected to be the primary trigger mechanism. Based on observations over the two year monitoring phase, the failed material currently on the bluff face, or at the base of the bluff, is the result of shallow rotational slumps due to high groundwater levels. The trigger mechanism is the increased pore pressure in the perched aquifers.

During the monitoring phase of the project, the mean water level of Lake Michigan was at the lowest it had been since 1964 (Figure 3) and no evidence of toe erosion due to wave action from storms was witnessed. The periods of movement, primarily from winter through early summer, correlate well with freeze/thaw during the winter and early spring and with high groundwater levels during the spring and early summer due to snowmelt and precipitation. The theorized trigger mechanism of increasing pore water pressure due to high groundwater levels is supported by both the data and the models.

Line B exhibits two primary areas of movement (Fig 24). The upper slump extends from pole four, located within the till, to near the mid-section of the bluff at pole six, located within the middle sand layer just above the lower clay aquitard. This slump is inclusive of the upper clay layer but discontinues at a location within the till layer. The slump is also deeper between poles five and six. This suggests that the till, composed of a poorly sorted matrix of gravel, sand, silt and clay, is stronger than the sand layer below it. The excessive movement of pole five, within the upper clay aquitard, suggests three possibilities: 1) the clay layer is permanently saturated, 2) the clay contains a fair amount of silt and/or 3) this layer of clay is fractured and discontinuous in this area. Silty clay is often permanently saturated because the silt tends to hold water. If a fair amount of silt is present within the clay, water within the bluff weakens this aquitard as well as the sand and till layers on either side of it. The behavior of a fractured clay layer would be similar. The presence of water in and around the fractured clay layer would weaken the clay as well as the till above it and the sand below it. Indeed, silt lenses holding water within the clay may be the source of the fractures and all three theories would be supported simultaneously. The lower slump is shallower, extends from pole seven to near the base of the bluff, and encompasses a layer of the bluff composed entirely of sand. However, based on the movement of pole eight, the lower slump has a deeper portion as well.

Line D exhibits minimal surface erosion near the top of the bluff within the till above the upper clay layer and only slight pole movement, so this portion of line D is presently stable (Figure 25). Within the sand between the upper and middle clay layers, line D has shown significant movement, particularly of pole five. Some eroded material from the immediate north of line D has been transported to the area between poles five and six, as indicated by the increase of material in this portion of the bluff. However, movement near poles five and six is certainly caused by subsurface forces including increased pore pressure within the sand above the middle clay layer. Moderate surface elevation increases between poles seven and eight to the base of the bluff are also the result of failed material being deposited in this area. However, their down slope movement is also attributed to a shallow slump.

Both lines B and D have two distinct areas of movement and both areas are separated by the lower of the two clay aquitards. Above this clay layer both lines have had their most dramatic movement. This is attributed to the strength of the lower perched clay aquitard and suggests that it is not permanently saturated, that it does not contain much silt and/or that it is not fractured; attributes that the clay layer above it are theorized to have. The lower clay layer

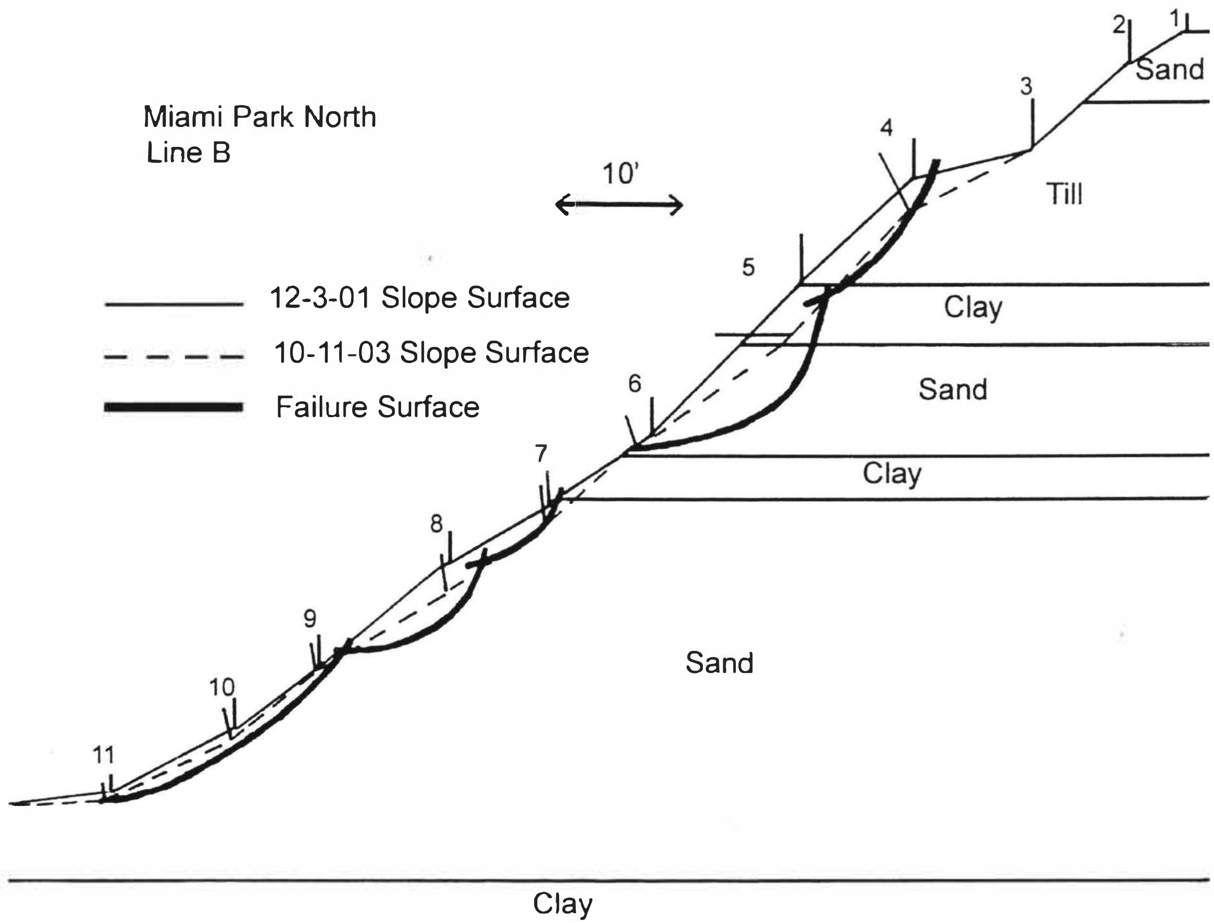


Figure 24

LINE B POLE MOVEMENT PROFILE

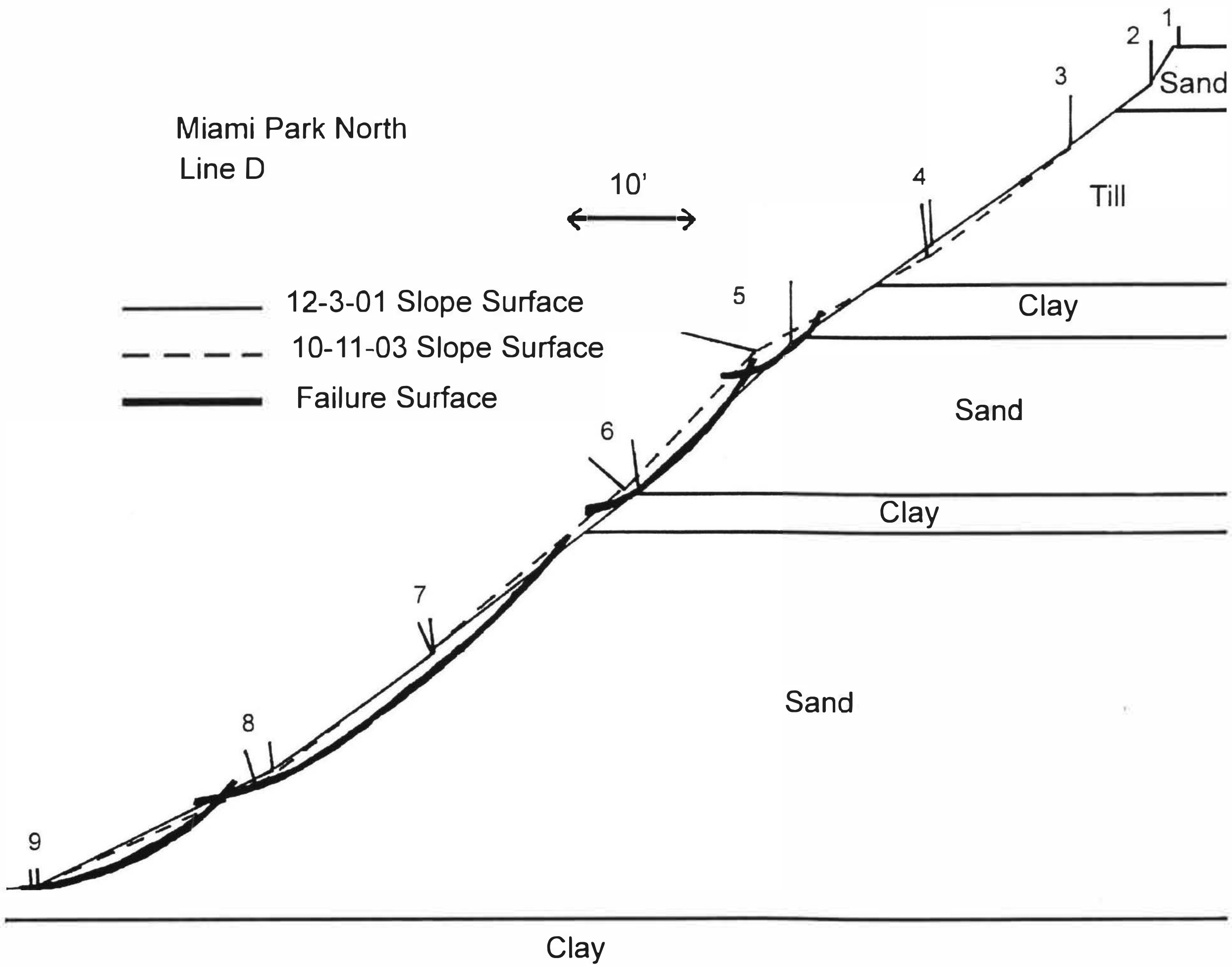


Figure 25

LINE D POLE MOVEMENT PROFILE

acts as an effective aquitard thus weakening the material above it and resists failure making it less susceptible to movement.

Past slope failure to the immediate south of the study area between Miami Park North and Miami Park South (Chase et al, 1999) and analysis of the GALENA models produced for this site suggests that significant future slope movements may be catastrophic events associated with large amounts of water recharging the perched aquifers from rapid snowmelt, prolonged precipitation or a combination of the two. Significant future slope movements may also be the result of successive incremental movement associated with snowmelt, precipitation or from the introduction of water from human activity such as leaking septic tanks or over watering lawns and gardens or a combination of these events.

The seasons anticipated to be the most problematic, as the data shows, are winter, spring and early summer. In winter, the bluff face is typically frozen and water within the perched systems has difficulty escaping. This increases the pore water pressure and triggers failure. Spring snowmelt and spring/early summer precipitation are also conditions that increase pore pressure and trigger failure at these times. Precipitation events that are of a long duration are particularly problematic, especially if the soil has been previously saturated. However, the late summer and fall seasons can also have long duration precipitation events. Significant movement could happen during these seasons if conditions are optimal, but they are not expected to be an annual occurrence.

CHAPTER 9

SUMMARY

It is generally agreed that in most landslides groundwater constitutes the most important single contributory cause of failure (Bell, 1993). Surface water flowing over a sloped surface erodes, steepens and undercuts the slope. Groundwater adds weight, increases pore water pressure, decreases internal friction and can change the mechanical properties of the soil. At Miami Park North, the site is currently experiencing surface erosion near the top of line B and moderate slope failure along portions of lines B and D due to increased pore pressure within the bluff. If conditions are right and the perched water tables receive significantly more water than they are able to emit effectively, then the increase in effective pore pressure will decrease the effective shear strength of the soil within the bluff and promote failure on a larger scale. Examination of the GALENA models and measured movements suggests that future moderate to severe failures will most probably involve shallow rotational shear slides or rotational slumps with earth flow at the toe and a concave up slip surface but not necessarily a circular arc with uniform curvature. It should, however, be noted that the failures anticipated will not necessarily occur at one time. The failure may occur in a series of incremental movements over a period of time or the failure may be catastrophic and instantaneous. The anticipated failure mechanism is an increase in pore pressure within the slope material caused by:

1) frozen soil at the face of the bluff in the winter restricting the near continuous seeps observed along the bluff face and/or 2) increased amounts of groundwater that are not removed or dissipated quickly enough due to snow melt from inland areas or heavy precipitation near the lake shore. However, in the absence of extreme toe erosion or high water levels in the perched aquifers, shallow failures at this site are predicted to continue as they have over the past two years.

APPENDIX

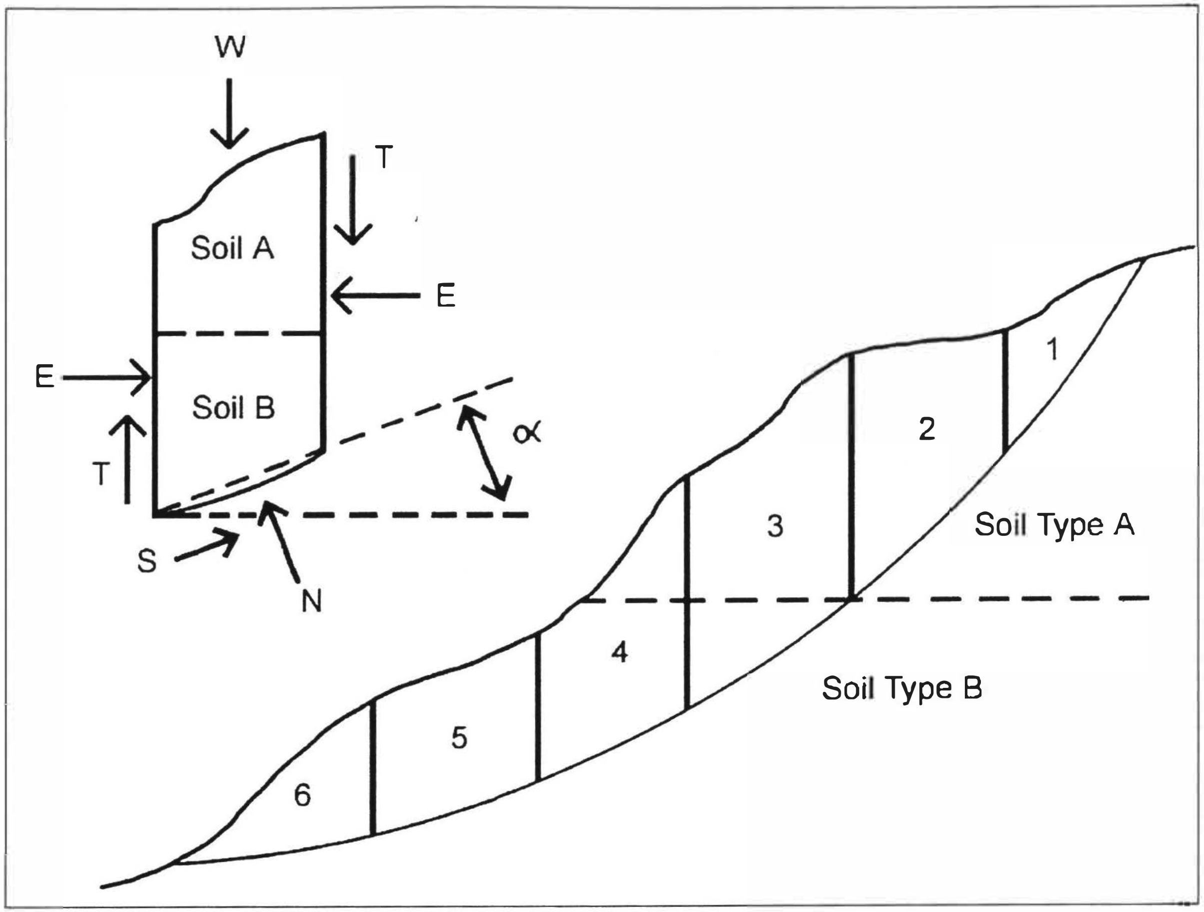
Bishop's Simplified or Ordinary Method of Slices assumes a circular failure surface and the slope is divided into a number of vertical slices. Each slice must contain only one type of soil at the base and the slice must be sufficiently narrow so as to approximate a straight line along the circular failure surface. It can be used to analyze the stability of a slope with numerous soil properties and various pore water pressures.

The forces acting on each slice are summed together and a Factor of Safety is obtained. If the sum of the forces on the slices promoting failure is greater than the sum of the forces resisting failure then the Factor of Safety will be below one. Likewise, if the forces resisting failure for each slice are summed and are larger than those promoting failure, then the Factor of Safety will be above one and the slope is computed as stable at the conditions used in the analysis.

The forces acting on the slices include the weight of the slice (W), the normal force acting on the base of the slice (N) (assumed to act at the center of the base of the slice), the shear force acting on the base of the slice (S), and the normal (E) and shear (T) forces acting on the vertical sides of the slice. In Bishop's Simplified Method, the forces along the vertical sides of the slice (E and T) are assumed to cancel each other out and are set to zero.

Additional geotechnical information required for computation includes, for each soil unit, cohesion (c), the angle of internal friction or critical failure angle (Φ), the unit weight (γ), and the pore pressure (u). The length of the slice at the base (ℓ) and the average height (h) of the two sides of the slices are needed as well.

It should be noted that the computation is along an assumed failure surface and that an alternate failure surface may produce a lower Factor of Safety. In addition, an initial estimate of the Factor of Safety is used to obtain the first computed Factor of Safety. The second computation uses the Factor of Safety obtained in the first computation to obtain the next result and so on. When two subsequent Factors of Safety of equal value are computed, the analysis for that slip surface is complete. Thus the use of a computer program that can quickly calculate the final Factor of Safety for a given failure surface and analyze a great number of possible failure surfaces is highly desirable. Equations for the Bishop method and a table to enable visualization of the process are included (Bishop, 1955; Nash, 1987; Duncan, 1996).


W
T
Soil A
E
E
Soil B
α
T
S
N
1
2
3
4
5
6
Soil Type A
Soil Type B

							Fa=				
Slice No.	W	ℓ	α	c	Φ	u	N_1	N_2	N_2	N_2	N_2
1											
2											
3											
4											
5											
6											
7											
8											
9											
10											
						Σ=					
							Fc=				

W = weight of slice (kN/m)

ℓ = length of slice base (m)

α = angle between slice base and horizontal (degrees)

c = cohesion (kN/m^2)

Φ = angle of internal friction (degrees)

u = pore pressure (kN/m^2)

F_a = assumed F

F_c = calculated F

$N_1 = W \sin \alpha$

$$N_2 = \frac{\{[(W/\cos \alpha) - u\ell\,] \tan \Phi + c\ell\}}{\{1+[(\tan \alpha \tan \Phi) / Fa]\}}$$

$F_c = \Sigma\,(N_2) / \Sigma\,(N_1) =$ ____

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