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Regional Approach to Landslide Interpretation and Repair

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Table of Contents

List of Figures

List of Tables

Abstract

The objective of this report is to provide Iowa county engineers and highway maintenance personnel with procedures that will allow them to efficiently and effectively interpret and repair or avoid landslides. The research provides an overview of basic slope stability analyses that can be used to diagnose the cause and effect associated with a slope failure. Field evidence for identifying active or potential slope stability problems is outlined. A survey of county engineers provided data for presenting a slope stability risk map for the state of Iowa. Areas of high risk are along the western border and southeastern portion of the state. These regions contain deep to moderately deep loess. The central portion of the state is a low risk area where the surficial soils are glacial till or thin loess over till. In this region the landslides appear to occur predominantly in backslopes along deeply incised major rivers, such as the Des Moines River, or in foreslopes. The south-central portion of the state is an area of medium risk where failures are associated with steep backslopes and improperly compacted foreslopes. Soil shear strength data compiled from the Iowa DOT and consulting engineers files are correlated with geologic parent materials and mean values of shear strength parameters and unit weights were computed for glacial till, friable loess, plastic loess and local alluvium. Statistical tests demonstrate that friction angles and unit weights differ significantly but in some cases effective stress cohesion intercept and undrained shear strength data do not. Moreover, effective stress cohesion intercept and undrained shear strength data show a high degree of variability. The shear strength and unit weight data are used in slope stability analyses for both drained and undrained conditions to generate curves that can be used for a preliminary evaluation of the relative stability of slopes within the four materials. Reconnaissance trips to over fifty active and repaired landslides in Iowa suggest that, in general, landslides in Iowa are relatively shallow (i.e. failure surfaces less than 6 feet (2 m) deep) and are either translational or shallow rotational. Two foreslope and two backslope failure case histories provide additional insights into slope stability problems and repair in Iowa. These include the observation that embankment soils compacted to less than 95% relative density show a marked strength decrease from soils at or above that density. Foreslopes constructed of soils derived from shale exhibit loss of strength as a result of weathering. In some situations multiple causes of instability can be discerned from back analyses with the slope stability program XSTABL. In areas

where the stratigraphy consists of loess over till or till over bedrock, the geologic contacts act as surfaces of groundwater accumulation that contribute to slope instability.

1.0 INTRODUCTION

1.1 Objective and Scope

The objective of this report is to provide procedures, specific to Iowa, that will provide efficient and effective interpretation and repair of landslides. The scope includes criteria to distinguish locations of potential slope instability, field evidence to identify incipient failure conditions, selected methods of slope stability analysis, and repair options that will restore stability to the site.

1.2 Acknowledgements

Many people contributed to this research by providing advice, data, and their individual experiences. Bob Stanley of the Soils Design Section, Iowa Department of Transportation (DOT) was a major contributor who reviewed the original proposal, gave access to DOT data files, and reviewed the findings at several stages of the investigations. Discussions with Bob both in the office and in the field helped clarify practical issues, especially repair methods used by the Iowa DOT. Lee Hansen of the Iowa DOT contributed ideas regarding stabilization strategies and helped in developing the soil strength database. Andy Barnes, Iowa DOT, wrote Section 8.3 of this report describing DOT slope repair methods. Iowa DOT drilling crews collected undisturbed soil samples used for case histories in Chapter 9.

Several county engineers reviewed the proposal and provided help in identifying sites of active and repaired landslides. These contributors included: Jim Christensen, Page County; Royce Ficthner, Marshall County; John Goode, Monroe County; Dennis Osipowicz, Lee County; Bob Sperry, Webster County; and Tom Stoner, Harrison County.

Several regional consulting firms provided shear strength data described in Chapter 6. The individuals and firms who contributed this information are Dennis Whited, Terracon – Cedar Rapids; Roch Player, CH2M Hill – Des Moines; and Brad Levich, Terracon – Omaha.

The research work of graduate research assistants: Bhooshan Karnik and Norman Chu is acknowledged. Bhooshan's contributions are included in Chapter 9 while Norman contributed to Chapters 3, 5, and 9. References to their theses appear throughout the report.

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of The Project Development Division of the Iowa Department of Transportation or the Iowa Highway Research Board.

2.0 REGIONAL APPROACH TO ANALYSIS AND REPAIR OF LANDSLIDES

It is impossible to develop an all-inclusive "design manual" for slope repair, because each landslide is unique and the result of a combination of factors. It has been observed, however, that the types of slope failures within a given physiographic region are limited (Baker and Chieruzzi, 1959). Because the number of variables is reduced in a given region, it is possible to classify landslides according to failure mode and to correlate the slope failures with engineering characteristics of the soils, topography and hydrology. This approach is used here.

2.1 Overview of Iowa Physiographic Regions

Seven physiographic regions have been defined to classify landform regions of Iowa (Prior, 1976). For this research, Iowa will be divided into three upland regions of significantly different topography and surface geology. Figure 2.1 shows the regions as defined for this study. More detailed descriptions of landforms and geology can be found elsewhere (Prior, 1976 and Ruhe, 1969)

In the north central portion of the state, glacial till comprises the nearly flat uplands and has been called the Des Moines Lobe (1976). Local relief in the uplands is generally less than 6 m (20 ft) and the only locations where relief is greater is along major streams such as the Des Moines River.

The western portions of the state, immediately adjacent to the Missouri River floodplain, have deep loess soils that form very steep hillslopes and narrow drainage divides. The loess deposits in this region have depths up to 50 m (160 ft). Local relief in uplands here is often in excess of 46 m (150 ft). The main source for the loess in western Iowa is interpreted to be the floodplain of the Missouri River, therefore the physical characteristics of the material in this region differ from the loess that is further away from the source. This western Iowa loess is often referred to as friable loess.

The remainder of the state is covered with loess of variable thickness, from 10 m (32 ft) to less than 2.4 m (8 ft), overlying glacial till. Local relief varies from 30 m (100 ft) to 9 m (30 ft) and the hillslopes are intermediate in slope angle between the Des Moines lobe and the loess hills of western Iowa. The loess here is often described as plastic loess. Paleosols (buried soil profiles) occur in this region and could cause localized slope instability.

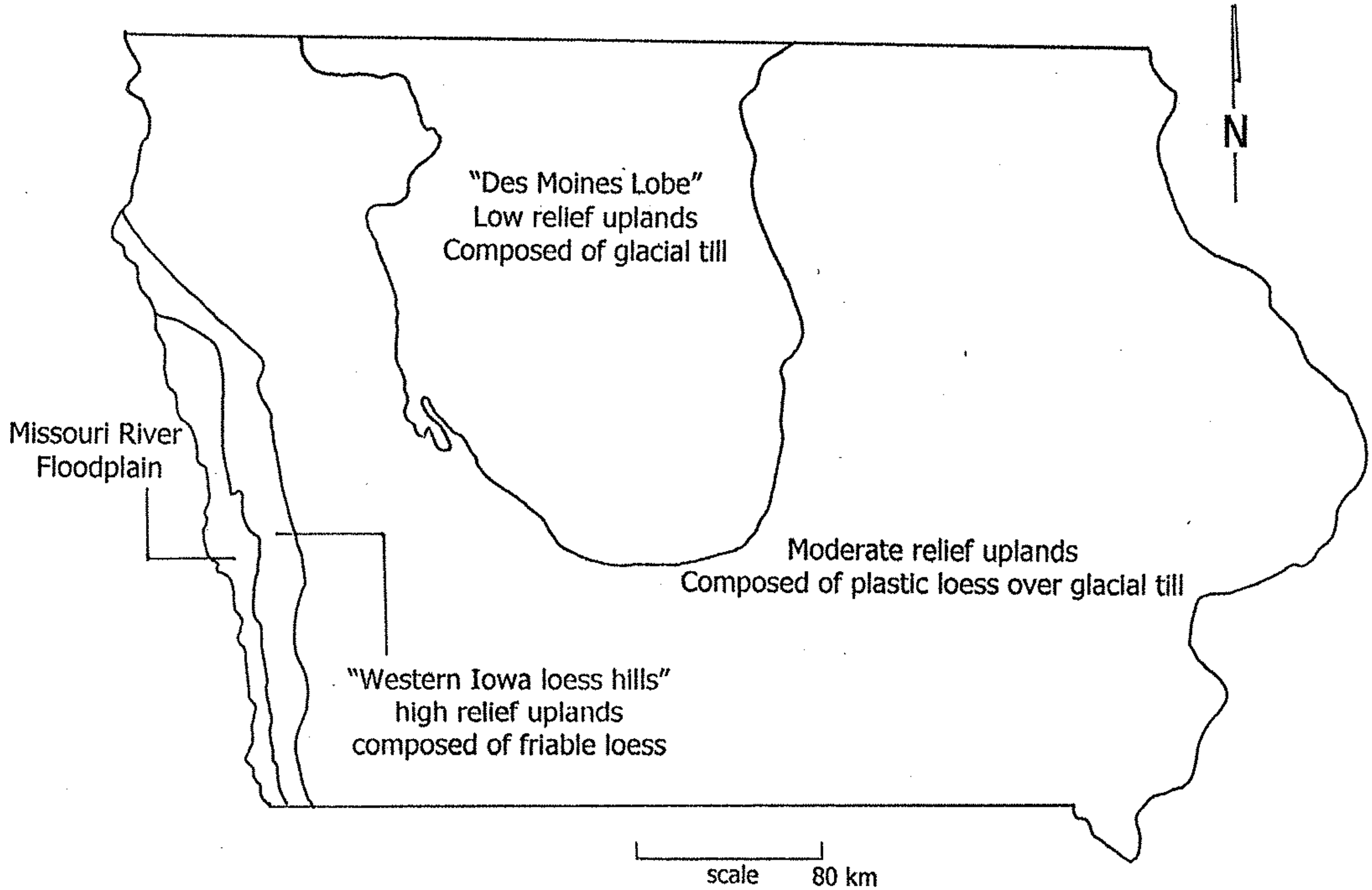


Figure 2.1 Iowa map showing physiographic regions as defined for this study

In any region of the state where large streams have deeply entrenched their valleys these Pleistocene deposits are in contact with the underlying bedrock. Areas of shale within each region will be treated as special problem areas.

3.0 SCOPE OF LANDSLIDE PROBLEMS IN IOWA

A survey of Iowa County engineers was conducted to assess the extent and nature of slope stability problems in the state and to determine successful repair methods. A questionnaire was prepared and sent to all the county engineers. The questions focused on landslides that have occurred since 1993. A copy of the questionnaire is included in the Appendix. A total of 99 questionnaires were sent, and 60 were received giving a response rate of 61%. The percentages reported here are based on only the counties that responded to the questionnaire.

The data from the survey were compared with the topographic map of the state and a correlation between frequency of landslides and relief was apparent. This resulted in a landslide susceptibility map, Figure 3.1, that categorizes regions of Iowa as either low risk, medium risk, or high risk regions for landslides. This map is an interpretative document based on incomplete data but does suggest regions of the state where landslides might be problematic.

3.1 The Statewide Distribution of Landslides

The data show that 48 counties, or 80% of those responding, have experienced landslides or slope stability problems. There are 44% of the counties with 1 to 5 landslides, 25% with 6 to 10 landslides, and 14% with 11 to 15 and 17% with more than 15 landslides since January 1993. Figure 3.2 summarizes the frequency of landslides on a statewide basis.

In the deep loess region of western Iowa, an average of 10 landslides occurred in counties that responded to the questionnaire. There are 3 counties with more than 15 landslides and one county with 6-10 landslides on the Nebraska border. Two of the southern most counties have 1-5 landslides.

In the glacial till area of Central Iowa an average of 5 landslides occurred in counties that responded to the questionnaire. Most of the counties in Central Iowa have experienced 1-5 landslides; except Emmet, Pocahontas, and Webster counties that have 6-10 landslides. Most of the counties adjacent to the Des Moines lobe have no or 1-5 landslides except Cherokee County

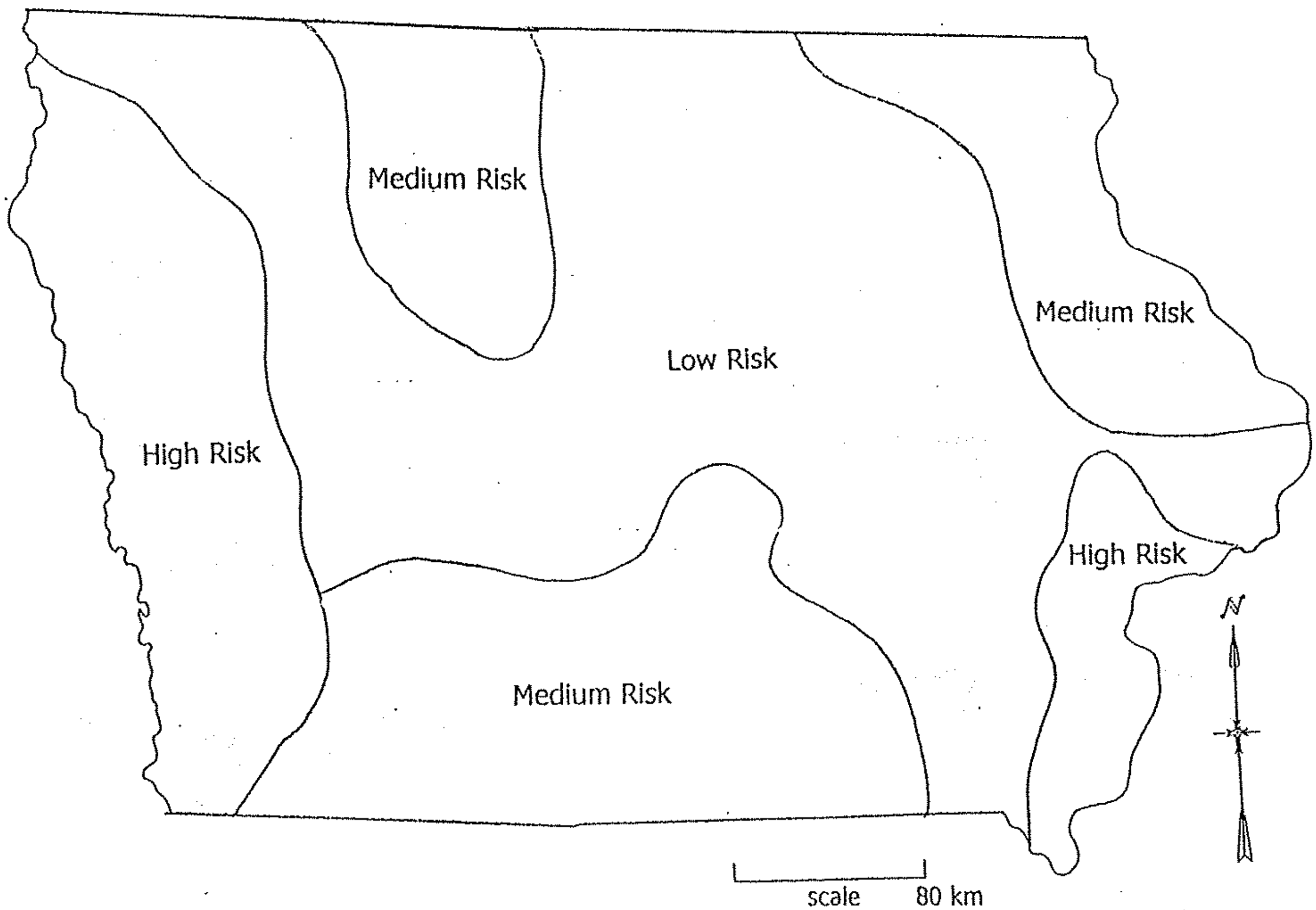
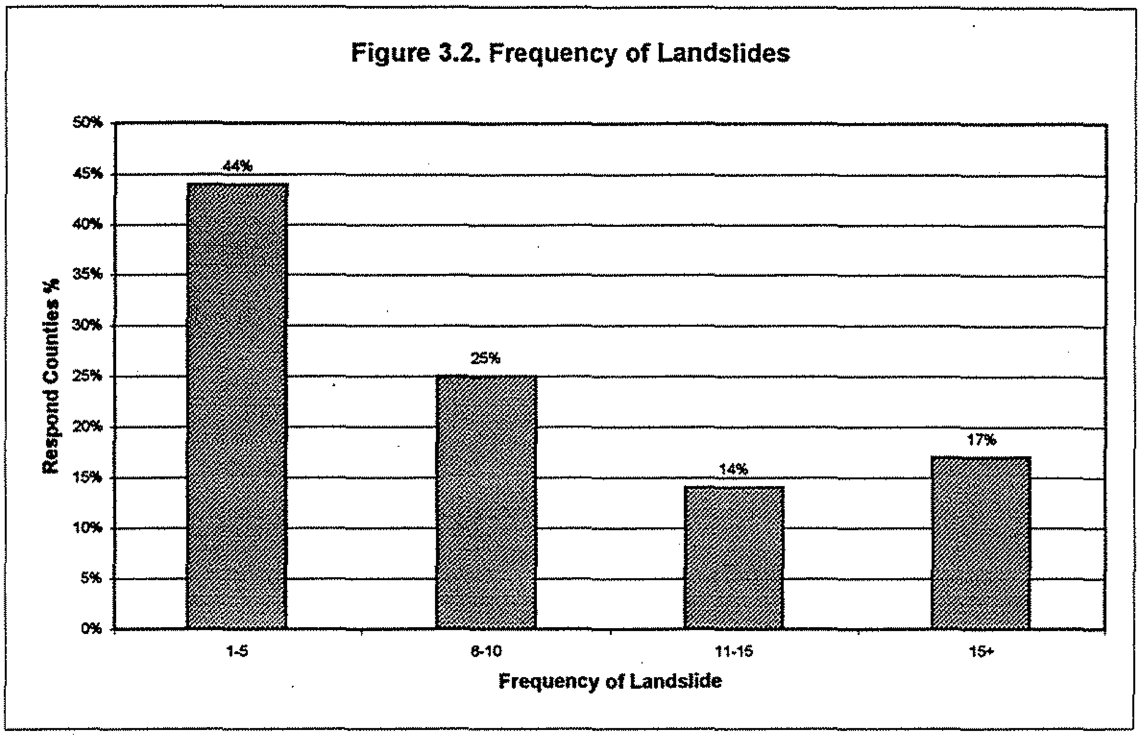


Figure 3.1 Landslide risk map based on survey of county engineers



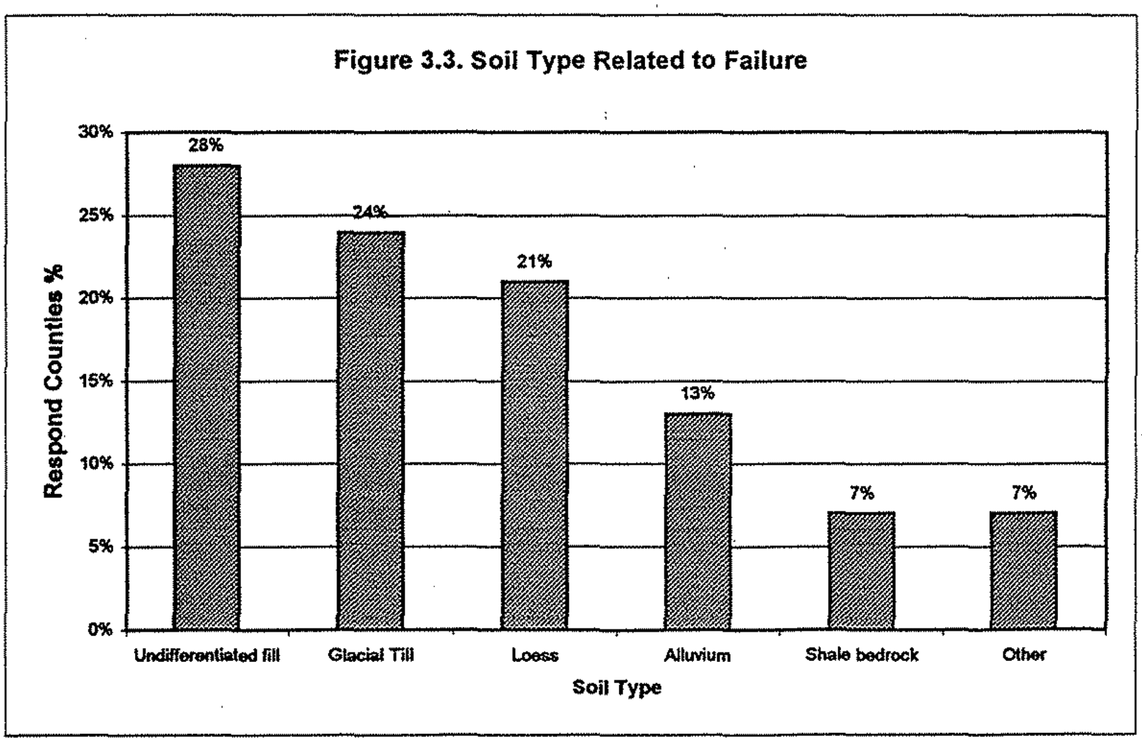


Figure 3.2 Frequency of landslides in Iowa since 1993

Figure 3.3 Soil types associated with landslides in north-western Iowa, and Madison and Jasper county in southern Iowa which have 11-15 landslides.

In the loess-mantled area, an average of 8 landslides occurred in the responding counties. Twelve of the counties have 1-5 landslides, and 7 counties have 6-10 landslides. Four counties in this region have 11-15 landslides: Allamakee and Clayton county in the northeastern part of Iowa, Jones county in the eastern part of Iowa, and Taylor county in southern Iowa. There are 4 counties with more than 15 landslides: Cedar in the east, Louisa and Lee in the southeast, and Monroe in the south-central part of Iowa. Most of the counties in the eastern part of Iowa have a significant number of landslides, ranging from 6 to more than 15, except Scott County with 1-5 landslides.

On a statewide basis, the soil most frequently associated with slope failures is undifferentiated fill with 28% of the failures. Glacial till and loess account for 24% and 21%, respectively, of the landslides. Alluvium is the soil associated with 13% of the slides and shale is the material in 7% of the slides. Figure 3.3 summarizes the soil type that is related to landslides.

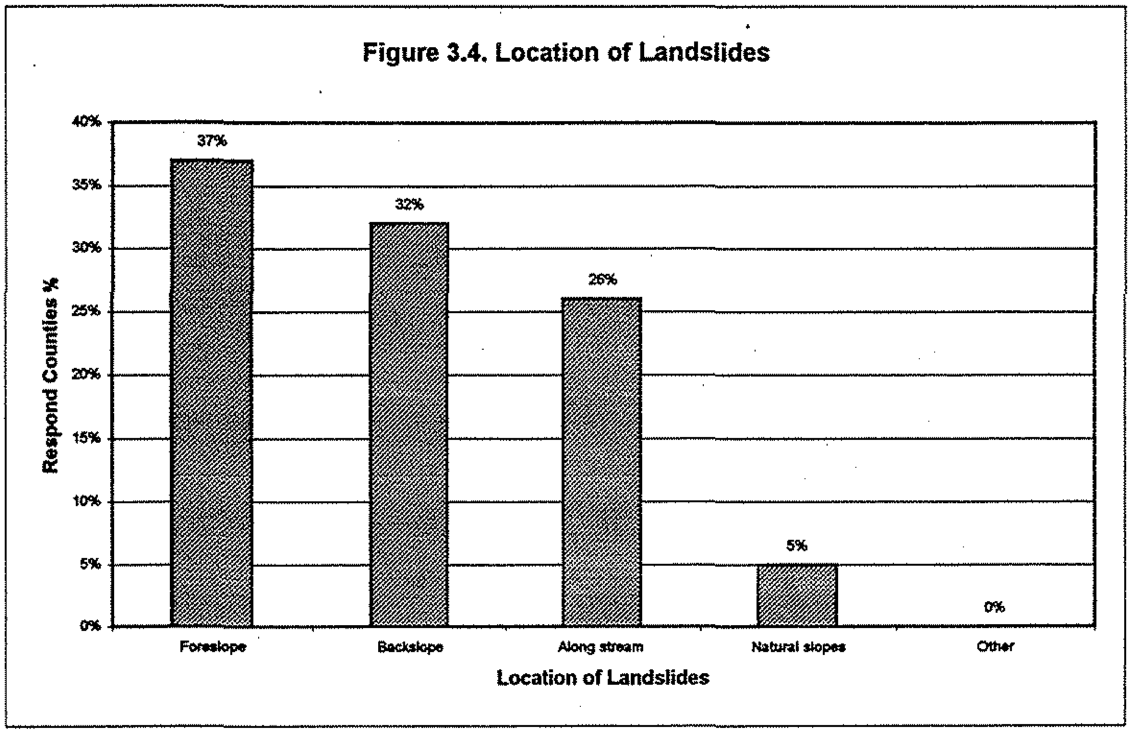
3.2 Foreslopes and Backslopes

One question in the survey asked if the slides occurred in foreslopes (embankments), backslopes (cuts), natural slopes, or along stream banks. Figure 3.4 shows the distribution of landslides. Foreslopes and backslopes were locations where stability problems are most frequent, with 37% and 32% of the slides in foreslopes and backslopes, respectively. In addition, 26% of the landslides occurred along streams or riverbanks and landslides in natural slopes comprised the remaining 5%. Most of the landslides in the northeastern and eastern part of Iowa occurred on backslopes; however most of the landslides in southeastern part of Iowa are in foreslopes.

3.3 Causes of Slope Failures in Iowa

All of the landslides occurred during spring and summer. Most, 78%, of the failures occurred in spring and the remaining 22% of the landslides happened in summer.

Fifty percent of the failures are associated with water where 28% of the slope failures occurred after heavy rainfall and 22% are associated with high ground water table conditions.



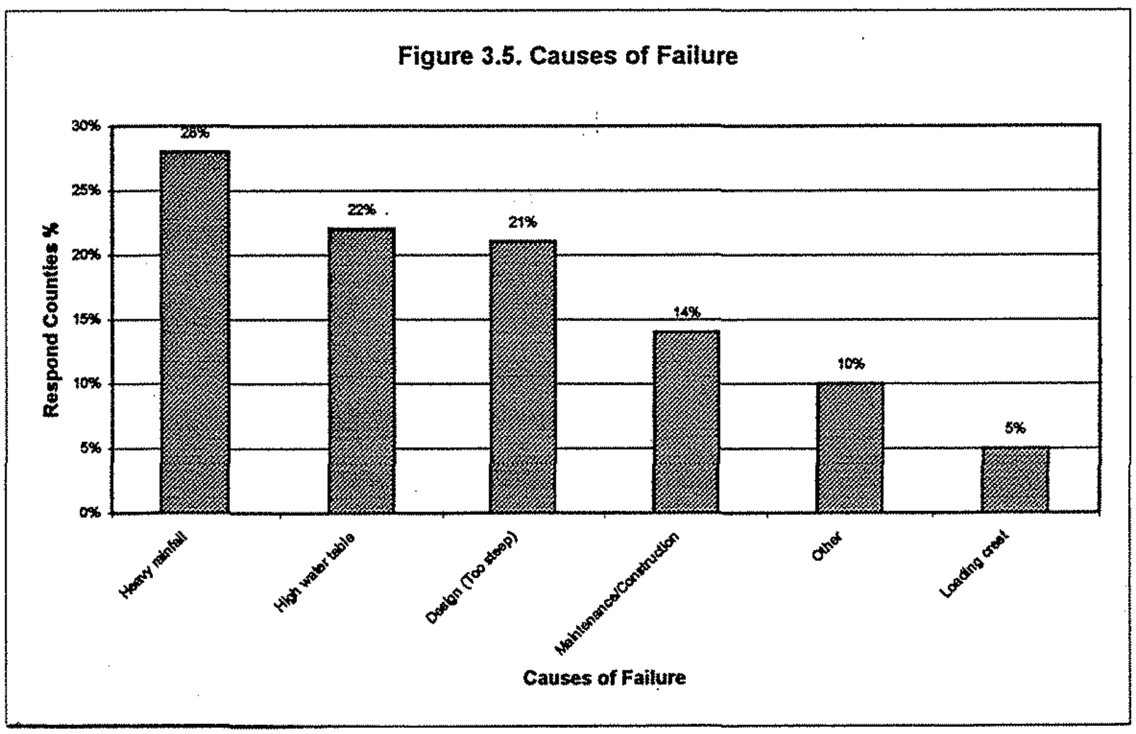


Figure 3.4 Location of landslides

Figure 3.5 Causes of failure

Twenty one percent of the slope failures occurred due to design issues. In addition, maintenance or construction activities accounted for 14% of the stability problems while loading at the crest of slope and other causes account for 5% and 10%, respectively. These data are summarized in Figure 3. 5.

3.4 Angles and Heights of Slopes before Failure

Ninety-six percent of the slopes before failure were steeper than 3:1. Eighteen percent of the failures are steeper than 1:1, 49% are in between 1:1 and 2:1, 29% are between 2:1 and 3:1, 3% are between 3:1 and 4:1, and only 1% are flatter than 4:1. Figure 3.6 shows the frequency of slides versus the slope angle before failure.

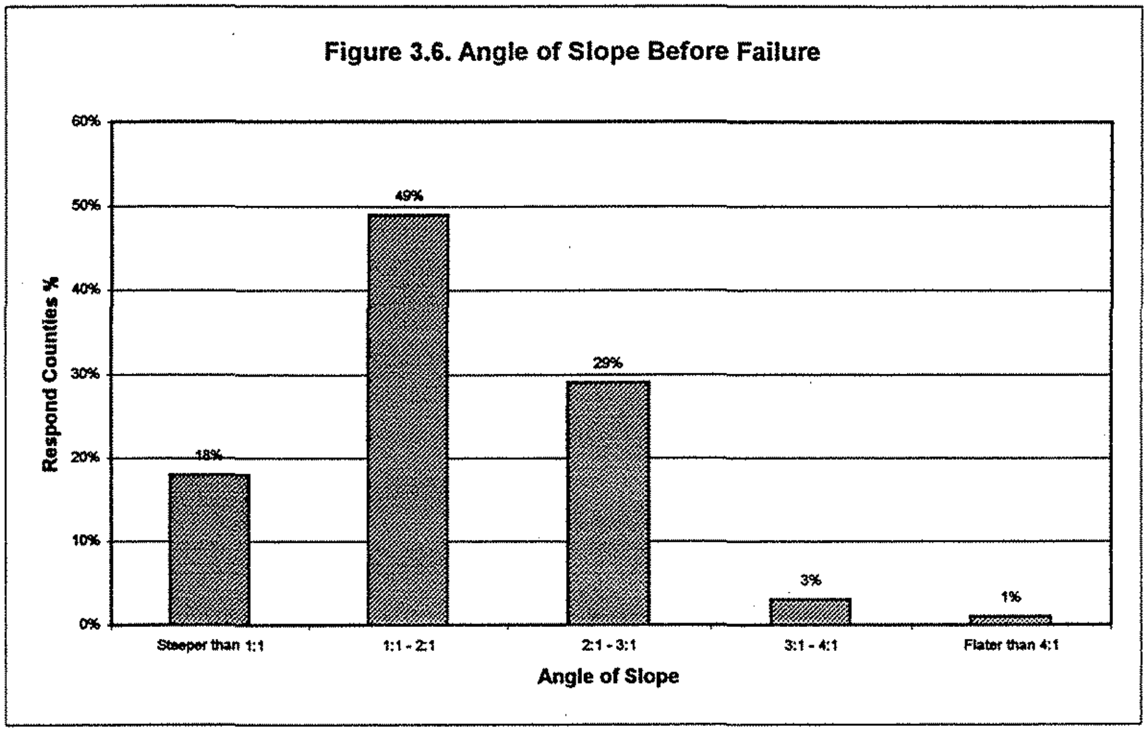
Nearly half, 41%, of the slopes were 11 ft. to 20 ft. high before failure. Twenty five percent, 25%, of the slopes were between 1 ft. and 10 ft. before failure, 21% and 13 % of the slopes were between 21 ft. and 30 ft. and greater than 30 ft. respectively. These data are shown in Figure 3.7.

3.5 Methods Applied to Prevent and/or Repair Iowa Landslides

Several methods have been applied to prevent and/or repair landslides. The most common methods used were decreasing the slope angle, with 27%, and water control and lowering the water table, with 26%. The next most frequent remedies used were loading the toe, with 13%, slope flattening by benching, with 12%, and structural support, with 8%. Geosynthetic stabilization, with 3%, was not used widely and chemical stabilization was not applied at all. In addition to the methods mentioned above, the county engineers have applied their own methods to deal with the landslides; these made up 11%. Figure 3.8 summarizes these data. The methods include using rip rap placement, sealing a utility trench cut at the top of the slope failure, and installing drainage tile near the toe of the slope failure.

3.6 Conclusions

Of the 60 counties that responded to the survey on landslides, 80% reported landslide activity and 31% of the counties had more than 11 landslides since January 1993. On a statewide basis, most of the slides occur in foreslopes composed of undifferentiated fill. Both curvilinear and planar failure surfaces were observed throughout the state.



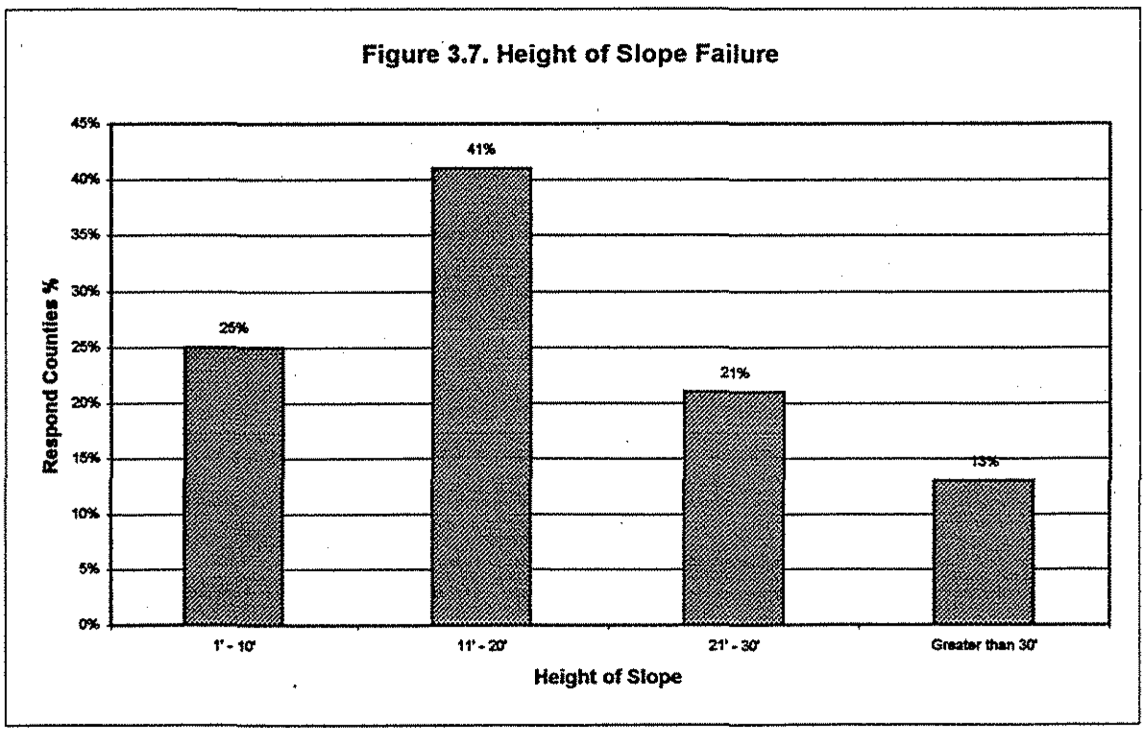


Figure 3.6 Slope angle prior to failure

Figure 3.7 Slope height prior to failure

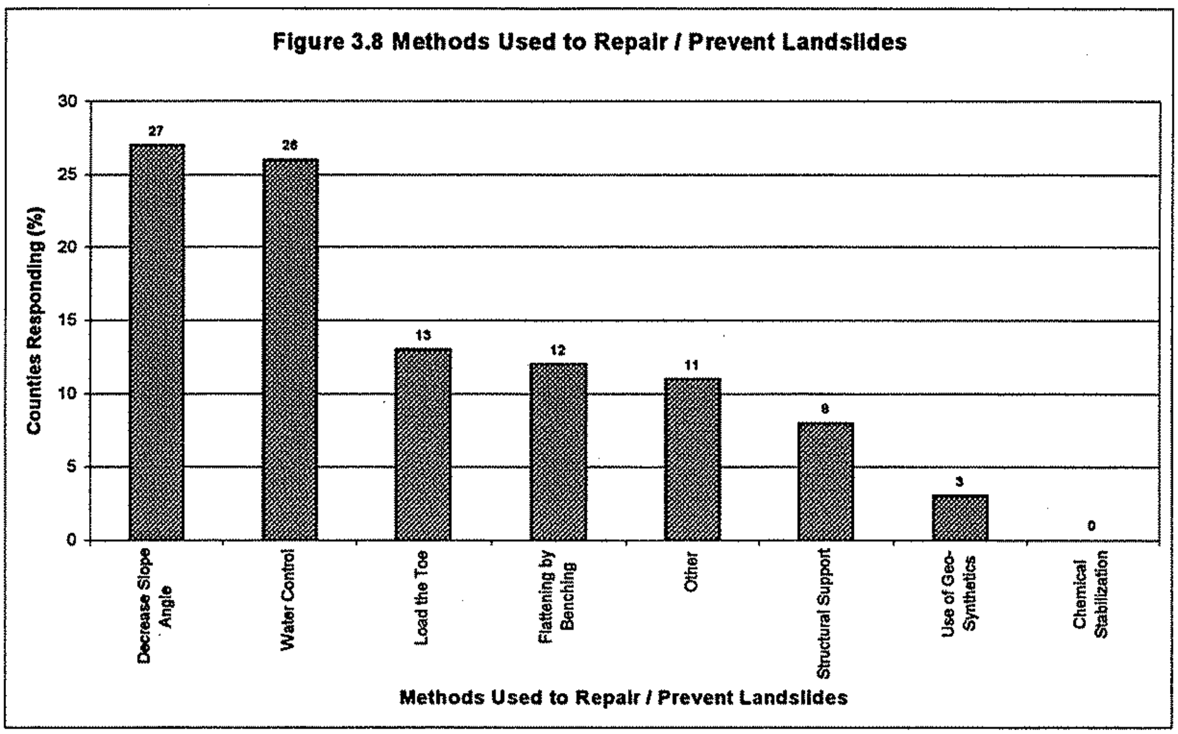


Figure 3.8 Methods of landslide prevention and repair

All of the landslides occurred during spring and summer with 50% of the failures caused by groundwater. Twenty one percent of the failures are associated with design issues. Nearly all of the slope failures occurred in slopes greater than 3:1 with the majority of failures on slopes between 1:1 and 2:1. Most of the slides occurred in slopes between 11 to 20 ft high before failure.

The most common and successful repair procedures have employed drainage and slope flattening. Structural support and geosynthetic stabilization are used very infrequently. Chemical stabilization has not been employed in the state.

4.0 LANDSLIDE INTERPRETATION

4.1 Conditions that Affect Slope Stability

Four general factors that affect slope stability are geology, slope geometry, hydrology and human activities. These factors are useful to identify sites of potential landslide susceptibility.

Geology influences slope stability in terms of soil and rock properties and the contacts between two soil or rock types and tension cracks. Soils with low shear strengths and high unit weights are more likely to fail. Paleosols with high clay content and shales that are exposed to weathering are two materials that are especially vulnerable to slope failure. A geologic contact between two soil types that dips in the same direction as the slope is a potential failure plane. Also some soils and rocks develop tension cracks when excavated or exposed by geologic processes. These tension cracks can lead to toppling failures on natural and cut slopes.

Slope geometry affects stability in terms of height, slope angle, plan-form and aspect. High steep slopes are more likely to fail than low gentle angle slopes. This generalization is modified somewhat by consideration of the three-dimensional character of a slope so that the plan-form of a foreslope affects stability. Slopes that are concave in plan-form tend to be more stable than convex plan-form slopes. The concave plan-form slopes have compressive stresses in the soil mass that tends to strengthen the soil mass. Finally slope aspect, its exposure to sunlight, influences its stability. North facing slopes are more susceptible to slides than south facing slopes because of moisture retention.

Hydrology is usually a major factor contributing to instability in slopes. Static ground water has the combined effect of increasing the driving forces and decreasing the shear strength. Seepage parallel or through the slope creates seepage forces that reduce slope stability.

Human activities such as construction and maintenance may result in modifying drainage, loading the top of slope, removing soil from the toe of the slope or weakening the soil strength. Any of these activities could result in decreasing the slope stability and cause a slide.

4.2 Modes or Types of Slope Failures

Varnes (1978), in an attempt to provide an all-inclusive classification of landslides, used types of movement and earth material as the main characteristics of his system. He classified earth materials as bedrock or engineering soils and subdivided soils into coarse material called debris and fine grained material called earth.

Types of movement most observed in Iowa include: falls, topples, slides, and flows. Figure 4.1 illustrates these types of movement.

Both falls and topples occur on nearly vertical slopes where tensile forces are dominant. Topples include a movement where the top of the material rotates outward while falls include only a vertical downward movement. Flows involve down slope movement in which the entire sliding mass behaves as a viscous liquid.

Slides represent shearing or sliding on a surface or in a zone within the earth materials. Slides can be either translational or rotational. Translational slides involve planar failure surfaces with movement in which the vector is primarily down slope with no upward component. In these situations the movement may be essentially parallel to the original slope surface. Translational slides can also occur on a contact between two materials if that contact dips at an angle smaller than the slope angle. In shallow slides the dominant movement is be translational with a small downward component at the top of the slide and an upward component at the bottom of the slide.

The movement of rotational slides includes a large downward component near the top of the slide and an upward component at the bottom of the slide. The failure may occur on a curvilinear, almost circular, surface; on a series of surfaces; or on a combination of circular and planar surfaces. These slides are deeper than translational slides.

In Iowa, falls or topples are likely to be limited to bedrock gorges of northeast Iowa, the loess bluffs of western Iowa adjacent to the Missouri River and its tributaries, or along major streams incised into the glacial till of central Iowa. Nearly vertical slopes formed in deep loess and desiccated glacial till have obvious tension cracks in the upper portions and piles of talus at the toe that suggest falls and topples may occur.

Flows may occur in some situations. In order for the soil to behave as a viscous liquid, the field moisture content must exceed the liquid limit. This can occur if high intensity rain falls on a soil with high void ratio. This is possible in friable loess where the liquid limit is less than 40% and a dry unit weight of 12.6 kN/m^3 (80 pcf) results in a saturated moisture content of 41%. This condition is not found in glacial till, plastic loess or most alluvium where the dry unit weights are higher causing the saturation moisture contents to be lower. The only other possibility for flows to occur in Iowa soils involves soils containing high amounts of

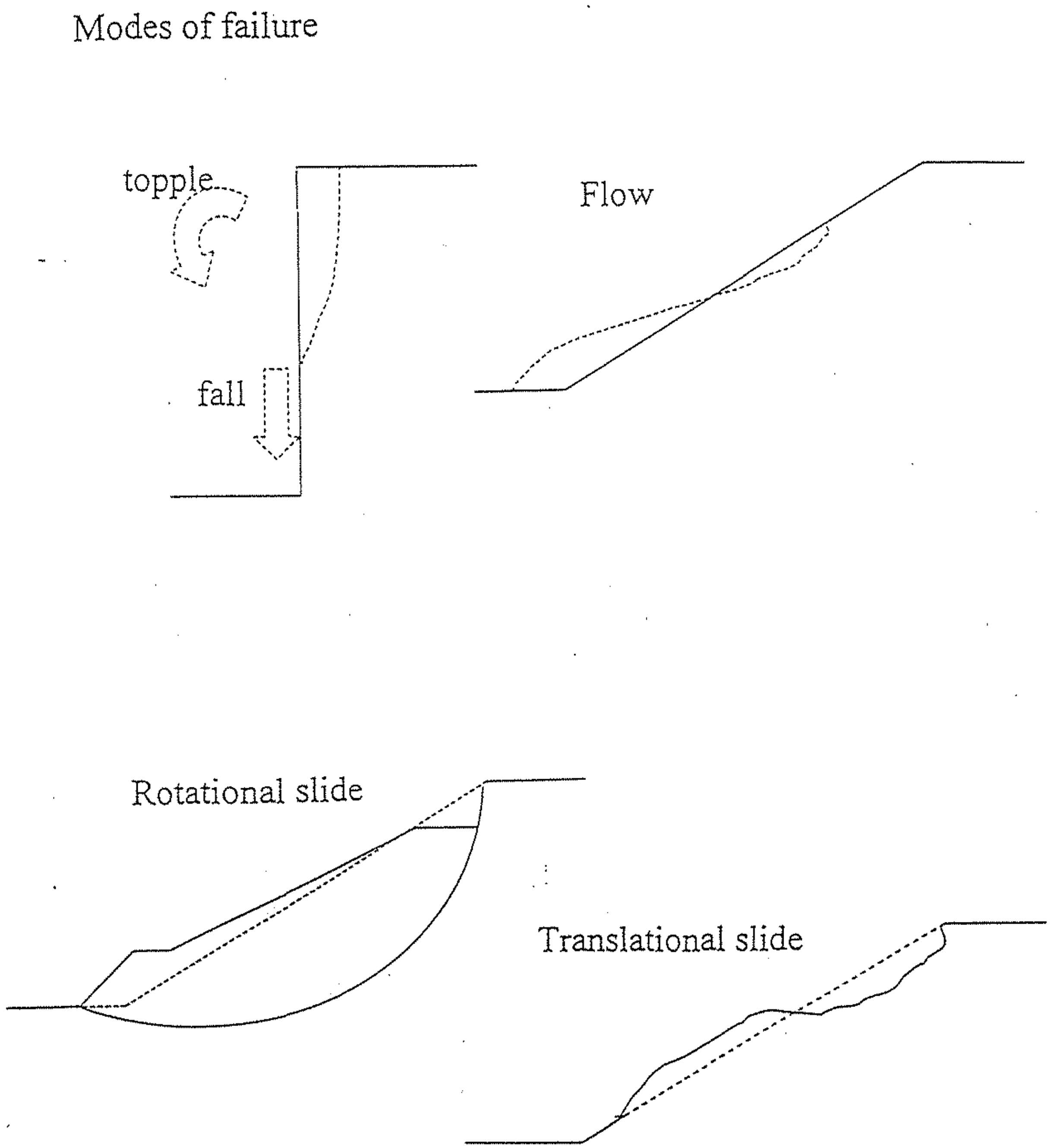


Figure 4.1 Landslide failure modes observed in Iowa

montmorillonite. Those soils are ones derived from shale or paleosols developed on glacial till. The latter material has limited geographic extent in the state.

4.3 Field Evidence of Movement or Potential Future Instability

Field evidences for a slope stability problem are classified as those associated with any slope, those occurring on slopes below the roadway, and those above the roadway. Observations that indicate active or potential slope instability are summarized in Table 4.1.

Evidence of instability for any slope	Evidence of instability for backslopes and foreslopes below roadway	Evidence of instability for foreslopes above roadway
Cracks or scarp at top of slope	Pavement settlement	Debris on roadway
Bulge at bottom of slope	Deformed guard rails	Blocked drainage ditches
Diagonal cracks along slope	Erosion at outlet of drainage structure	
Ponded water indicating localize seepage		
Cattails or willows indicate localized seepage		
Tilted tree trunks		

Table 4.1 Field Evidence of slope instability

The thick vegetation that covers most Iowa slopes makes identification of these features difficult during the summer months; therefor reconnaissance for slope stability problems is most effective in early spring or late fall when soil conditions are more easily observed.

For any slope one of the earliest indicators of slope instability is the occurrence of cracks at the top of the slope. These cracks usually are approximately parallel to the crest and curved in plan view with the ends of the arc pointing down slope. Isometric views are shown in Figure 4.2. After the soil mass above the failure surface has begun to move, one or more scarps appear near the top of the slope and a budge or mud wave appears at a lower elevation or at the toe of the slope as shown in plan view, Figure 4.3. The lateral extent of the slope is often defined by diagonal cracks en echelon down slope, also shown in Figure 4.3. A hummocky or irregular surface on the slope may indicate a series of shallow rotational slides or a vegetative mat that is sliding translationally down slope.

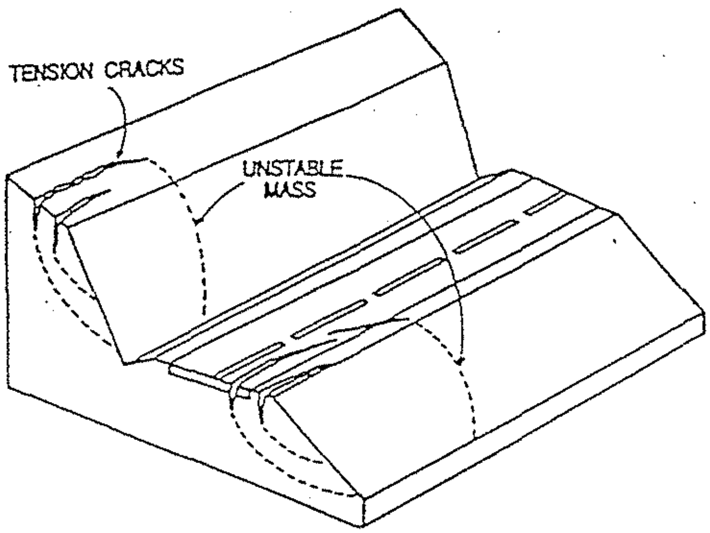


Figure 4.2 Tension cracks, an early sign of slope instability (FHWA, 1988)

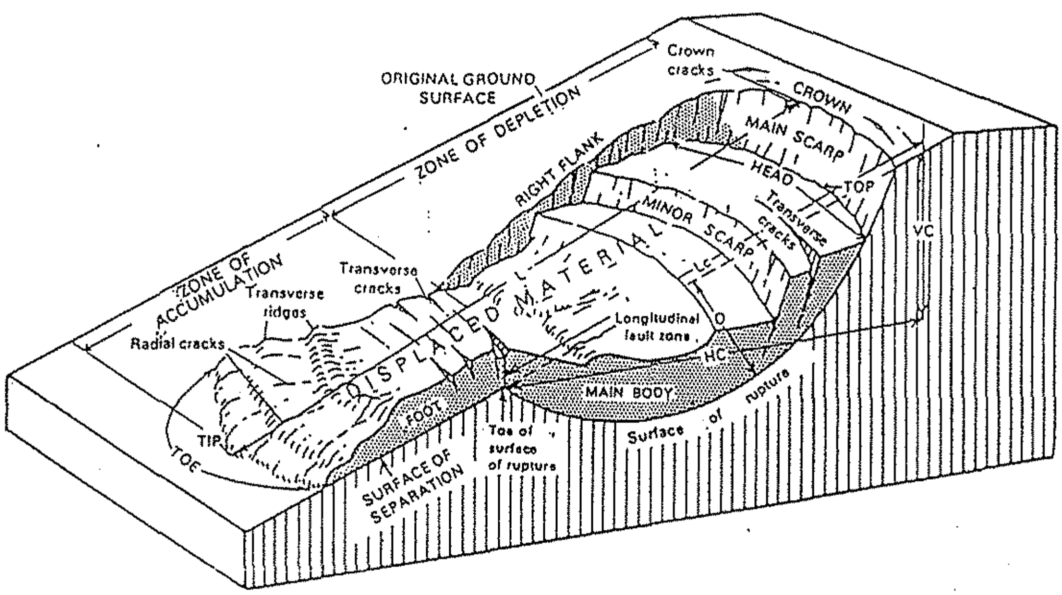


Figure 4.3 idealized morphology of a landslide (Varnes, 1978)

Many slope failures are associated with groundwater seepage so ponded water or obvious seepage on the slope surface is evidence of existing or potential slope instability. Indirect evidence of near surface water is hydrophilic vegetation such as cattails and willows.

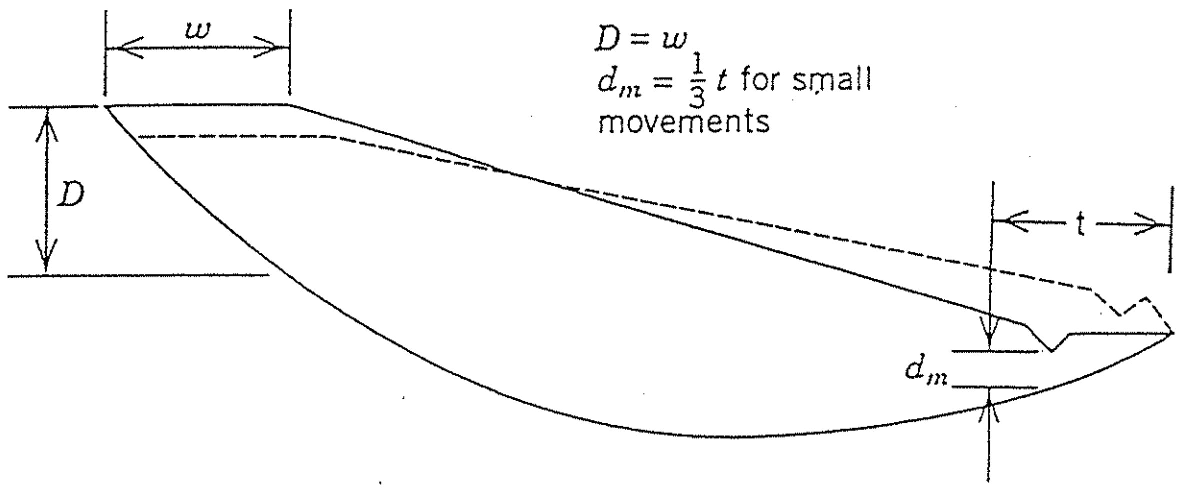
Vegetation, in the form of tilted trees, can be another clue for slope movement. Depending on the mode of failure and the depth of the root zone relative to the failure zone the trees may be titled either up slope or down slope. Rotational slide below the root zone may cause the tree to tilt up slope while translational slide can cause a down slope tilt.

Evidence of instability in backslopes or foreslopes below the roadway can be found in irregularities of engineered structures. Pavement cracking and settlement may indicate that the slope below the road is failing. Guardrails that are out of alignment or tipping suggest slope instability. Two other conditions that may lead to future instability are lateral stream bank erosion or erosion below a culvert outlet. Both of these situations may cause steeper slope angles and subsequent failure.

Unstable cut slopes or natural slopes above the roadway are most easily identified. Obvious indicators of slope failure are soil or rock debris on the road and blocked drainage ditches.

4.4 Estimating depth of failure surface

Once evidence exists that a slope failure has occurred, it may be possible to estimate the extent of the failure surface according to a method suggested by McGuffy (1991) and shown in Figure 4.4. Cracks or scarps at the crest of the slope indicate the top of the failure surface. The failure surface is at a depth below the crest of the slope equal to the distance from the furthest crack at the top of the slope to the crest. A bulge down slope indicates the lowest extent of the slide. If the bulge is exposed on the slope, the failure surface is at the bottom of the bulge. If the bulge occurs below the toe of the slope, the depth to the failure surface is one-third the distance from the toe to the edge of the bulge. From these estimated depths, it is possible to reconstruct the shape of the failure surface.



Typical depth to failure surface (McGuffey, 1991).

Figure 4.4 Method for estimating depth of failure surface (McGuffey, 1978)

5.0 SLOPE STABILITY ANALYSES

5.1 Considerations Prior to Stability Analysis

Before an analysis of a failed slope can be conducted, it is necessary to answer six questions: Is long term or short term stability more critical? What is the configuration of the failure surface? Are discontinuities present in the soil mass? What is an appropriate safety factor? Is the observed failure a new or reactivated slide? Are the appropriate soil strength parameters and unit weight conditions known?

The shear strength of saturated soils depends on the drainage conditions. If a soil is loaded slowly so that pore pressures can dissipate, the stresses are effective stresses and the shear strength of over consolidated clays is characterized with cohesion, c, and friction angle, ϕ. If loading is rapid, and pore pressures cannot dissipate, the friction angle becomes zero and the strength is characterized by s_u, undrained strength. In general the effective stress (c - ϕ) analysis is applicable to long term conditions while the total stress (s_u or $\phi = 0$) analysis applies to end of construction situations.

In considering short term versus long term slope stability, variations in shear stress and pore pressures on a potential failure surface are critical issues that have been discussed in detail by Bishop and Bjerrum (1960). In the case of cut slopes with the water table at an elevation above the bottom of the cut, the pore pressure will decrease due to unloading if the construction proceeds rapidly and the slope is not allowed to drain. However given sufficient time to dissipate the negative excess pore pressures, the pore pressure distribution will be determined by the flow net. For this long-term condition the analysis can be conducted with effective stress parameters. For natural slopes, geologic processes that have established steady state drainage control the long-term equilibrium. The safety factor of the slope will typically reach its minimum value under long term conditions. It is also possible that an increase in water table elevation could change the drainage of the system and lead to instability. In general as long as the slope is free draining, the long-term stability is more important for cut slopes and natural slopes.

For embankments the shear stress and the pore pressures increase as construction proceeds. If the construction proceeds more rapidly than the excess pore pressures can dissipate, pore pressure will build up and could reach the critical stage causing a slope failure. In this situation the undrained analysis is more appropriate. At the end of construction the shear stresses are constant and given sufficient time the soils will drain. This dissipation of excess pore pressures should result in increased stability. If a highway embankment has survived the construction without landslides, it should remain stable. If failure occurs in an embankment at some time after the end of construction, it is likely due to incomplete compaction, disruption of drainage so that water is impounded on the upstream side of the embankment or the soils have weathered and lost strength or initially unsaturated compacted soils have taken on water.

The shape of the slide failure surface influences the difficulty of the analysis. If the failure surface is a plane, simple closed form analyses are possible, however if the failure surface is curved, more comprehensive analyses are required. For some analyses the curved failure surfaces may be approximated with the arc of a circle.

Geologic discontinuities such as contacts between different soil types or tension cracks provide potential planes of weakness that may become failure planes. Failures on geologic contacts are possibilities in the loess mantled till region of Iowa while tension cracks occur in deep loess deposits of western Iowa or in the Des Moines lobe where deep erosion or construction cuts go through glacial till.

A safety factor is selected based on data reliability, uncertainty and consequences of failure, and potential future perturbations. If strength test data show considerable scatter or the assumptions used in the stability analysis are many, a larger safety factor may be required. In general a safety factor of 1.5 is considered adequate for most stability analyses and Iowa DOT has had very good results working with a safety factor of 1.3.

Finally the history of the slide area should be considered. If the slide occurs along a preexisting failure surface, residual strength parameters should be used. Peak shear strength parameters are used only if the slide does not occur on an older failure surface. Reconnaissance from this study suggests that reactivated slides are not common in Iowa.

5.2 Scope of Analyses Used Here

Dozens of slope stability analyses exist, but this report focuses on four analyses. The analyses differ in terms of assumptions regarding the shape of the failure surface, stress distribution within the failed soil mass and the shear strength of the soil or rock. Two of these analyses assume planar failure surfaces and have closed form solutions. These are the infinite slope analysis and the Culmann analysis. Both analyses apply to long term stability or effective stress conditions. A third method of analysis assumes that the failure surface is an arc of a circle and that the soil is undrained or the analysis applies to short term conditions. This analysis has been presented as stability charts by Taylor (1948) that are easy to use. The last method and most rigorous of the group applied here is applicable to short term and long term drainage conditions as well as to analyses with steady state seepage. The last, most sophisticated analysis has been adapted to a compute program, XSTABL, that is relatively straightforward to apply assuming both circular and noncircular failure surfaces.

5.2.1 Infinite slope analysis

The infinite slope analysis is applicable to slopes that are long (tens of meters) in comparison with the depth of the failure surface that are shallow (a few meters). As with all forms of stability analysis the weight of the soil mass above the failure surface is resolved into normal and tangential components. The tangential component becomes the activation shear force. In a slope that is at limiting equilibrium, this activating force is equal to the shear strength of the soil. A factor of safety is defined as the ratio of shearing resistance or strength to the activating shear force.

For the infinite slope analysis, the failure plane is assumed to be a plane parallel to the surface of the slope at a depth, d, from the surface. The angle between horizontal and the slope surface or the failure plane is β. A static analysis gives the following equation for the Factor of safety Fs:

$$Fs = \frac{c}{\gamma d \cos^2 \beta \tan \beta} + \frac{\tan \phi}{\tan \beta}$$

where c is the soil cohesion, γ is the unit weight of the soil, and ϕ is the friction angle of the soil. If the soil has no cohesion and the slope is at equilibrium, i.e. Fs = 1, it can be seen that the friction angle equals the slope angle.

For an infinite slope with a saturated soil and seepage parallel to the surface the factor of safety becomes:

$$Fs = \frac{c}{\gamma d \cos^2 \beta \tan \beta} + \frac{\gamma' \tan \phi}{\gamma \tan \beta}$$

In this equation γ’ is buoyant unit weight of the soil.

5.2.2 Culmann analysis

For this analysis, the failure surface again is assumed to be a plane but the angle between the horizontal and the failure plane, θ, is less than the slope angle. The average shear stress is equal to the shear strength along the plane and the critical shearing plane is one that has the minimum ratio of shearing stress to shear strength of the soil. For this analysis the safety factor is applied by dividing the shear strength parameters, c and ϕ, by an appropriate number. For static equilibrium, the maximum height, H, of stable slope is determined from:

$$H = \frac{4c\sin\beta\cos\phi}{\gamma(1-\cos[\beta-\phi])}$$

where all terms are as previously defined.

The angle the failure plane makes with the horizontal, θ, can be determined from the following equation:

$$\theta = \frac{\beta+\phi}{2}$$

5.2.3 Taylor's stability charts

For short term or unconsolidated, undrained slope stability analyses, $\phi = 0$, and the strength of the soil is derived only from its cohesion. For this special case, the cohesion is referred to as the undrained strength, s_u. Taylor (1948) developed a series of stability charts for this analysis assuming that the failure surface is circular. One of these charts is reproduced as Figure 5.1 in which D is the depth factor describing the extent of soil beneath the toe of the slope, H is the maximum height of stable slope, and γ_t is the total unit weight of the saturated soil. The stability number characterizes the relative stability:

$$\text{stability number} = \frac{s_u}{\gamma_t H}$$

As the stability number increases, higher steeper slopes are stable. The maximum possible height of slope is the same for all slope inclinations less than 54° when D becomes very large.

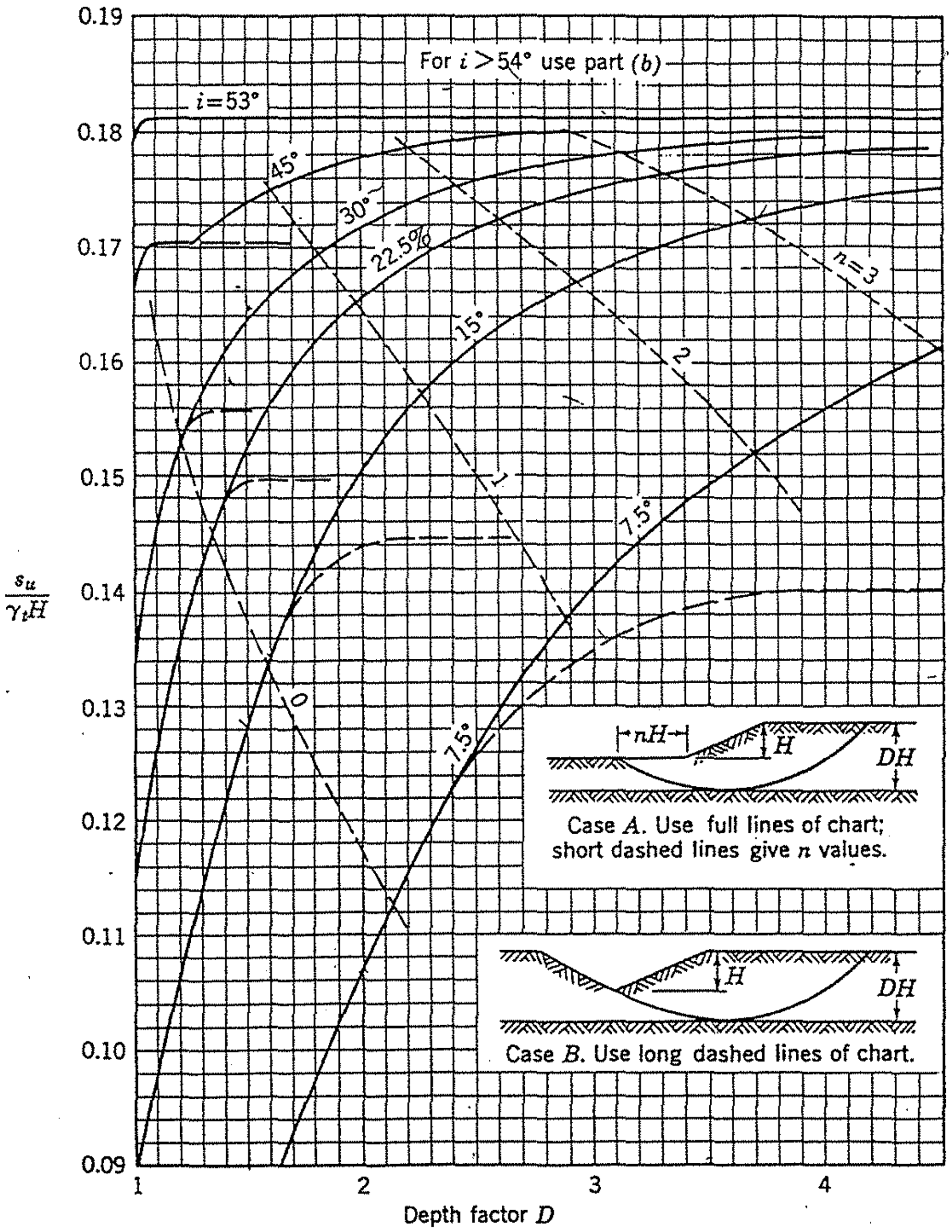


Figure 5.1 Stability numbers chart for various depth factors (Taylor, 1948)

For failure surfaces that pass through the toe of the slope, D equals one. Figure 7.1 provides a tool for rapid evaluation of simple slopes with uniform undrained strength.

5.2.4 XSTABL

XSTABL is a PC-based method of slices slope stability analysis program. The generalized equilibrium method that is used by this program allows factors of safety to be calculated for force and moment equilibrium or for force equilibrium only. Spencer's method, the Morgenstern-Price method, Corps of Engineers methods, the simplified Bishop method and Janbu's method are available for calculating Factors of Safety. The program allows searching for the most critical circular, noncircular or block shaped surface or allows analysis of a single circular or noncircular failure surface. Automatic failure surface generation functions, that use either initiation/termination ranges of the failure surface or use search boxes to generally define failure surface location, may be effectively used to locate the critical failure surface. Slope stability analyses can include the effects of external surcharge loads, limiting boundaries and anisotropic, undrained and nonlinear Mohr-Coulomb shear strength types. For effective stress analyses, pore pressures may be simulated by applying a piezometric surface, multiple phreatic surfaces, a pore pressure grid, constant pore pressure or a pore pressure parameter (r_u).

The software, therefore, can be used: to effectively determine the factor of safety for the most critical failure surface for a given slope or to back analyze a failed slope. The program can also be used to carry out parameter sensitivity analyses that relate factors of safety to parameters such as soil stratigraphy and shear strength parameters, slope angle and geometry, pore pressure conditions and slope loading/perturbations.

6.0 DATA BASE FOR PRELIMINARY SLOPE STABILITY GUIDELINES

6.1 Effective Stress Data

The objective of this part of the research is to compile effective stress shear strength data in order to develop preliminary guidelines for stable slope angles and heights in different geologic materials found in Iowa. Two kinds of triaxial test data were used in this analysis; consolidated, undrained (CU) data with pore pressure measurement from the consultants and consolidated, drained (CD) data from Olson (1958) and Benak (1967). Both types of data produce effective stress parameters that are appropriate to long term stability analyses.

Two consulting engineering companies, Terracon and CH2M HILL provided triaxial test data. Geologic parent materials for Terracon data were interpreted by comparing the sample location, depth and description from the boring reports with a surficial geologic map in the book Quaternary Landscapes of Iowa (Ruhe, 1969). CH2M HILL provided the interpretation of geologic parent material for their samples. The data from these two sources represented glacial till and alluvium. Data for friable loess were from theses (Olson, 1958 and Benak, 1967). The data were sorted according to geologic parent materials, and the mean and standard deviation calculated for cohesion, friction angle, dry unit weight, total unit weight and moisture content. Values of the means are presented in Table 6.1.

Geologic Material	Cohesion kPa		Friction angle degrees		Dry unit weight kN/m^3	
	Mean	Std.dev	Mean	Std. dev.	Mean	Std.dev.
Glacial till	7.65	5.59	28	1.2	15.1	2.0
Friable loess	5.21	4.00	25	1.4	13.5	0.5
Plastic loess	6.91	4.19	29	4.2	14.3	1.2
Alluvium	2.28	1.90	31	1.3	15.3	0.7

Table 6.1 Average effective stress strength parameters and dry unit weights for Iowa geologic materials. (Std. Dev. denotes standard deviations)

In addition, t-tests (Neville and Kennedy, 1964) at 5% and 10% levels of significant difference were carried out to determine if the differences in parameter means between various geological materials are statistically significant. The results of the statistical analyses are presented in Chu (2001). The standard deviations for cohesion are high which indicates a high degree of scatter in this parameter. The results show a significant statistical difference for cohesion intercept between alluvium and glacial till, between alluvium and plastic loess, and between glacial till and plastic loess. A significant difference exists for friction angles between glacial till and plastic loess, between friable loess and plastic loess, between friable loess and alluvium, and between friable loess and glacial till. The only significant difference for dry unit weight is between friable loess and alluvium. There are significant differences for total unit weight between glacial till and plastic loess, and between friable loess and alluvium. A significant difference of moisture content can be found between alluvium and plastic loess, and also between friable loess and alluvium.

6.2 Undrained Shear Strength Data

The objective of this task is to compile shear strength data that were available from Iowa DOT design plans. The purpose of compiling the data is to develop guidelines for stable slope angles and heights in different geologic materials found in Iowa.

Iowa DOT projects from six counties were selected to obtain shear strength information representing different geologic materials encountered in Iowa. The exact locations of the borings were found from the DOT project reports. The boring locations were transferred to the appropriate USDA soil survey reports (Sherwood and Culver, 1977, Oelmann, 1984, Clark and McWillianms, 1978, Jury and Fisher, 1976, Worster, Harvey and Hanson, 1972, Lockridge, 1979, Koppen, 1975), and the geologic parent materials were interpreted. Statistical analyses were applied to the data to calculate means and standard deviations of cohesion, friction angle, dry unit weight, total unit weight, and moisture content. In addition, t-tests (Neville and Kennedy, 1964) at 5% and 10% levels of significance were carried out to determine if the differences in the mean values of the parameters for different geological materials are statistically significant.

The data were reported as consolidated, undrained (CU) test results; however the friction angles (1 to 5 degrees) are very low (usually less than 4^{o}) for CU tests. The test results, therefore, are interpreted as an unconsolidated undrained (UU) response and the cohesion intercepts are interpreted as undrained shear strength (s_u) values and summarized in Table 6.2.

Geologic Material	Undrained strength kPa		Total unit weight kN/m^3	
	Mean	Std.dev.	Mean	Std.dev.
Glacial till	30.3	9.3	19.1	1.7
Friable loess	21.9	7.0	18.2	1.1
Plastic loess	31.5	18.4	18.7	1.4
Alluvium	29.9	19.8	19.0	1.1

Table 6.2 Average undrained shear strengths and total , saturated unit weights for geologic materials in Iowa. (Std. Dev. Denotes standard deviations)

The results of the statistical analyses are presented in Chu (2001). The data in Table 6.2 show that the standard deviations for undrained strength (s_u) are high, indicating a high degree of variability in these data. The mean and standard deviation results indicate a significant amount of overlap of parameter values between the different geological materials. This is confirmed by the t-test results. The only results showing a significant difference across the different geologic parent materials are total unit weight between alluvium and glacial till, dry unit weight between loess derived alluvium and glacial till, and between glacial till and friable loess.

The tabulation of existing strength and unit weight data was conducted to provide parameters that could be used in preliminary assessment of slope stability for various geologic materials. The statistical analyses indicate for the most part that differences between the mean strength values of many of these materials is not significant. In the following chapter regarding preliminary assessment of slope stability, mean values of strength and unit weight are used for each parent material. The stability curves presented in Chapter 6 are intended only as guidelines for stability and the high variability of strength data for each parent material further indicates that the curves should not be used for design, but only as semi-quantitative guidelines for assessing slope stability.

7.0 PRELIMINARY ASSESSMENT OF SLOPE STABILITY

7.1 Objective and Scope

The objective of this section is to provide guidance on stable slope angles and slope heights for different geological materials encountered in Iowa. The curves presented are not for design but only preliminary evaluation of stable slope angles and heights. Two analyses were selected to calculate the relationship between stable slope angle and slope height. The Taylor (1948) analysis was used for an unconsolidated undrained condition and the Culmann analysis (Spangler, 1960) was selected to represent the slope in a drained condition.

7.2 Taylor Analysis

The Taylor (1948) analysis was used to analyze the stability of slopes immediately after construction, before pore pressure equalization and establishment of steady state seepage conditions. As used here the failure surface is assumed to be a circular arc that pass through the toe of slope i.e. D = 1, and that the factor safety = 1. In over 20 slope failures observed in Iowa, none exhibited surfaces that extended below the toes of the slopes. Undrained strength parameters interpreted from the DOT files were used in this analysis. The Stability Number chart shown in Figure 7.1, provides stability numbers, $\frac{s_u}{\gamma_t H}$, for any value of slope angle where s_u is the undrained strength and γ_t is total unit weight and H is the stable slope height. From the graph where D = 1, stability numbers were determined for slope angles between 10 degrees and 90 degrees. From the theoretical stability numbers and mean values for s_u and γ_t, for the four materials shown in Table 6.1 the stable height (H) can be determined. This calculation is then repeated for a variety of slope angles to generate a curve of slope height (H) versus slope angle (β) for each geologic material.

7.3 Culmann Analysis

Culmann analysis was used to represent the slopes in a drained condition i.e. the analysis is in terms of effective stresses. The solution is based upon the assumption that the failure surface is a plane passing through the toe of the slope. Field observations show that this assumption is approximately valid for high angle slopes, whereas lower-angle slopes tend to fail along a circular arc or a logarithmic spiral.

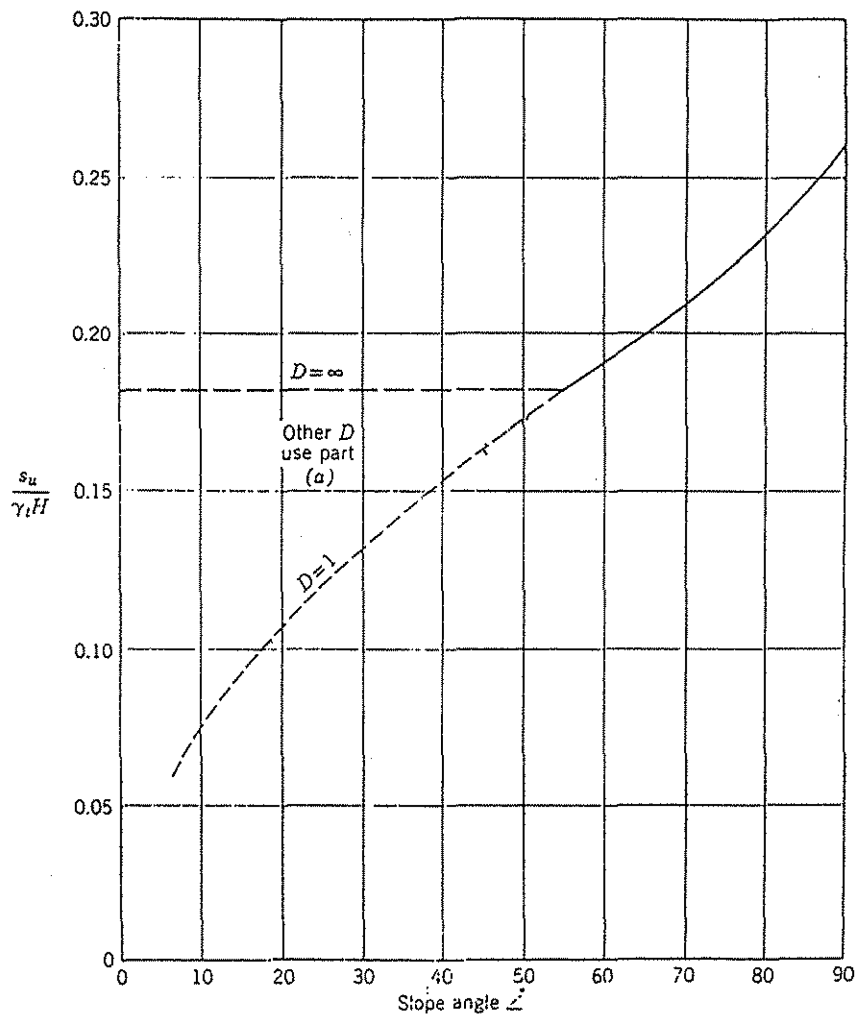


Figure 7.1 Stability numbers chart for failure surface through toe of slope (Taylor, 1948)

The equation

$$H = \frac{4\,c\,Sin\beta\,Cos\phi}{\gamma_t(1 - Cos(\beta - \phi))}$$

was used to calculate the maximum stable slope height where, H = maximum stable slope height, c = cohesion intercept of the soil, β = slope angle, ϕ = internal friction angle, and γ_t = total unit weight of soil. Mean effective stress strength parameters, shown in Table 6.1, were used in the Culmann analysis for various slope angles to generate curves of slope height (H) versus slope angle (β) for each geologic material.

7.3 Results of the Stability Analyses

The results for the Taylor analysis and the Culmann analysis are included in Figure 7.2. No factors of safety or reduction factors are applied to the results and pore pressures are not included in the Culmann analysis. Combinations of slope height and slope angle that fall below the curves represent stable conditions while those above the curves represent instability. These curves are for preliminary assessment of potential stability problems and are not to be used for design.

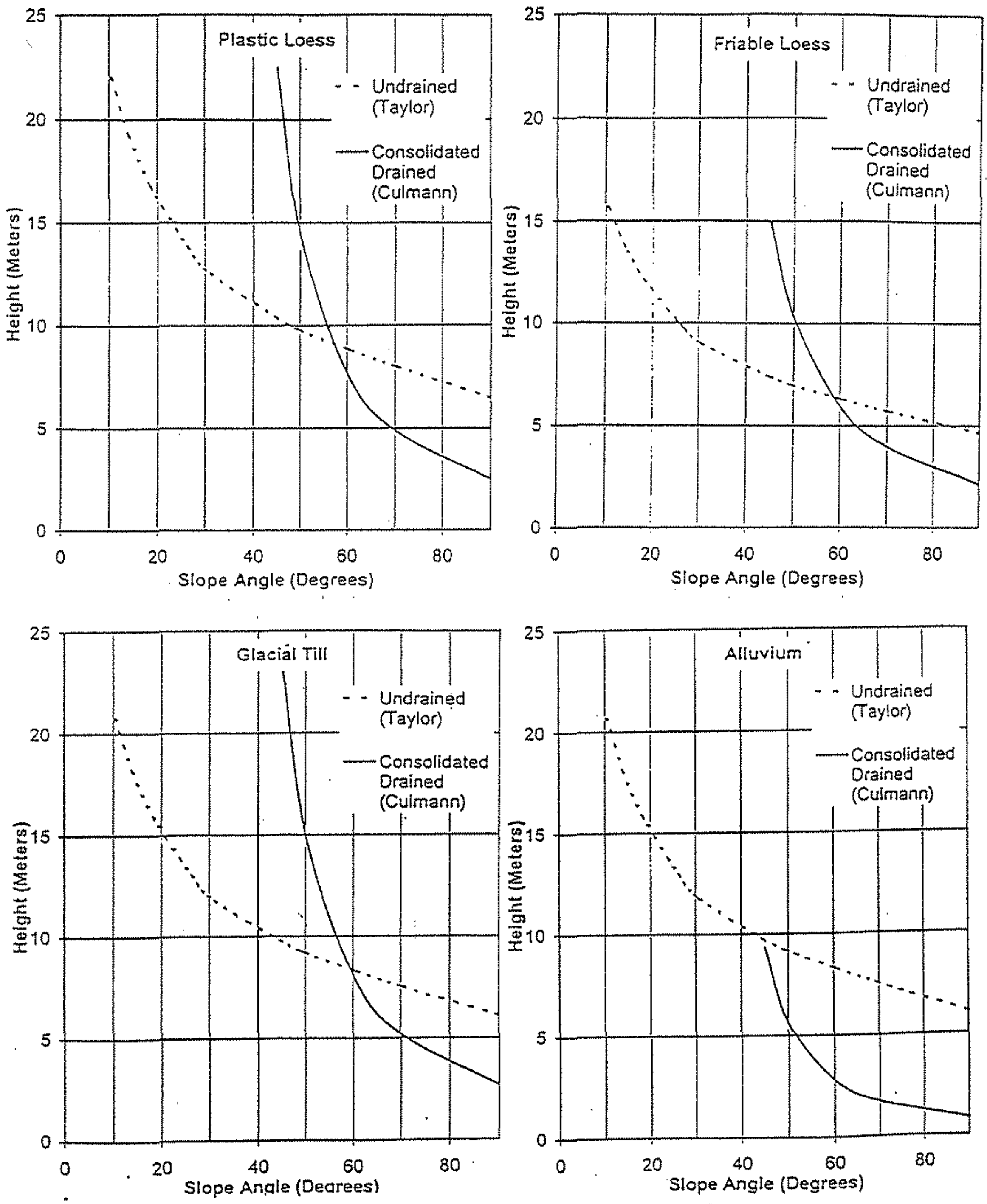


Figure 7.2 Charts for estimating slope stability of various Iowa soils (Not for design)

8.0 SLOPE REPAIR

This chapter of the report contains a general overview of slope remediation synthesized from a variety of publications and repair recommendations based on Iowa DOT experience. The overview is covered in sections 8.1 and 8.2 while the DOT recommendations are in section 8.3. Two general approaches to repairing unstable slopes are to reduce the driving forces and/or to increase the resisting forces acting in the slopes. A third strategy is to replace the slope with a retaining structure. Further discussion of retaining structures is beyond the scope of this study; however details on retaining wall analysis and design can be found in works such as Huntington (1957).

Ideally the repair method is analyzed for stability using standard methods. The same analyses can be used for both embankments and cut slopes; however, in general, long-term stability is more critical in cut slopes while end-of-construction is more critical in embankments. Exceptions to this generalization are for embankments that are constructed of materials containing shale that may weather and lose strength with increasing time or initially unsaturated compacted soils that take on water in the longer term.

Reduction of driving forces is achieved through excavation, replacement of soil with lightweight material, or drainage. An increase in soil shear strength can be accomplished through drainage, elimination of weak material, buttressing, in-situ reinforcement, or chemical stabilization. These methods can be subdivided into twenty specific techniques. The classification of slope repair activities is outlined along with common types of retaining walls in Table 8.1. The following paragraphs provide brief descriptions of the specific stabilization techniques.

8.1 Reduction of Driving Forces

8.1.1 Excavation

Excavating soil at the top of the slope will reduce driving forces. Stability analyses can be used to estimate the weight and volume of material needed to be removed. This remediation is unlikely to be successful if the failure is an infinite slope type. Flattening of backslopes will create lower driving forces but requires adequate right of way. Benching or replacing a single high slope with several short slopes consisting of horizontal treads and sloping risers will provide a more stable slope. Obviously the treads need to be of sufficient width to accommodate grading

Slope Repair Methods				Application and Limitations
Reduce driving forces	Excavation		Remove top	Deep seated slides
			Flattening	Effective for most slopes
			Benching	Effective for most slopes
	Light weight fill			Low cost material needed
	Drainage	Surface	Ditches	Essential to all repairs
			Permeable aprons	Controls surface water
			Sealing cracks	Beneficial to all slopes
		Subsurface	Horizontal drains	Deep seated slides
			Drainage galleries	Deep seated slides
			Horizontal wells	Rock and soil slopes
			Vertical wells	Deep slides (expensive)
Increase strength	Drainage			
	Eliminate weak material			Small shallow slides
	Buttressing		Soil and rock fill	Beneficial to all slopes
			Counter berm	Opposing slope at toe
			Shear key	Beneficial to all slopes
	In-situ reinforcement		Pneusol	Recycles scrap tires
			Soil nailing	Moderately deep slides
			Reticulated piles	Moderately deep slides
			Piles	Deep slides
			Grids	Deep slides
	Chemical treatment		Lime	Fine grained soils
			Portland cement	Stiff clays and shales
Retaining walls			Gravity	Limited right of way
			Cantilever	Limited right of way
			Segmental	Limited right of way
			Geosynthetics	Limited right of way
			MSE	Limited right of way

Table 8.1 Flow chart of slope repair methods

equipment. The design geometry of individual risers and composite slopes can be analyzed individually for long term stability and for end of construction conditions.

8.1.2 Light-weight fill

A relatively new technique to reduce the driving forces in slopes is to replace the existing failing soil with lightweight fill. Encapsulated wood chips, cinders, expanded shale, polystyrene foam, seashells, and shredded rubber tires have been used successfully (Abramson et al, 1995). The feasibility of this approach depends upon the availability of economical fill. Shredded, scrap tires provide a potential low cost lightweight fill in Iowa.

8.1.3 Drainage

Drainage that removes or controls pore water pressures within unstable soil masses is probably the most important slope repair practice when used alone or in combination with other techniques. The reason for this is that effective drainage decreases driving forces and also increases soil shear strength. The rigorous evaluation of a proposed slope drainage system requires flow net seepage analyses combined with slope stability analyses. Filter systems, such as filter fabric, need to be provided between high permeability soils or drains and lower permeability soils to prevent erosion and drain plugging.

Surface drainage is important in the stability of any slope and becomes critical in situations where failure has occurred previously (Cedegren, 1989). This is accomplished with ditches, permeable aprons, and crack sealing that diverts or controls the surface water flow so that it does not have the opportunity to infiltrate the slope. Ditches constructed at the top of the slope can intercept surface water and divert it away from the slope face.

Permeable aprons placed on the slope surface allow the runoff to infiltrate the permeable layer and then flow down slope above the less permeable soil substrate. Sealing surface cracks with concrete, shotcrete, or asphalt will slow or eliminate surface water infiltration.

Subsurface drainage is achieved with horizontal and vertical gravity drains, trench drains, drainage galleries, and horizontal and vertical wells. Horizontal drains, the most common type of drainage systems, are actually sub-horizontal at slopes of 2° to 5° installed by conventional rotary drilling. The drains consist of slotted polyvinylchloride (PVC) plastic pipe 60 mm (2.5 inches) to 100 mm (4 inches) in diameter. The drains discharge to surface drainage ditches. Vertical gravity drains discharge to drainage galleries. Wells are a more expensive means of dewatering, but may be economical where drainage depths are too deep for construction of drainage trenches. It is difficult to imagine situations in Iowa where wells might be feasible.

8.2 Increase Soil Strength

8.2.1 Eliminate weak material

In slopes where the slides are small or shallow, it is possible to stabilize the slope by excavating the weak soil and replacing it with higher strength material. This technique has the result of increasing the resisting forces in the slope.

8.2.2 Buttressing

A commonly used slope repair technique is to load the toe of the slope with material heavy enough to provide adequate resisting force. This approach, called buttressing, can be implemented with rock fills, counter berms, shear keys, and pneusols. Where adequate rock fills are locally available, this is a practical method to increase slope stability. Shear keys can add resistance to rock fill or counter berm buttresses by forcing the failure surface into deeper, high strength, undisturbed strata. Pneusol stabilization involves excavating slide debris to depths below the failure surface and reconstructing the slope with whole tires or sidewall mats held together with plastic or metal clips (Hausmann, 1992).

8.2.3 In-situ reinforcement

Increase in resistance can be achieved through in-situ reinforcement. Reinforcement techniques include soil nailing, reticulated micropiles, piles & stone columns, and grid systems. In general, when used as a slope repair method, these techniques are most useful for fairly deep slides where the failure surface location is known.

Soil nails are steel rods or tubes on the order of 25 mm (1 inch) in diameter that are driven or grouted in predrilled holes in the soil. The nails must be long enough to penetrate below the failure surface and are generally normal to the slope surface. One case study of a nail reinforced slope described nails 12.5 m (40 ft) long (Alston, 1991). Stability between the nails is achieved with a thin 100 mm to 150 mm (4 to 6 inches) layer of shotcrete reinforced with wire mesh. This technique may be more useful than piles in boulder rich soils such as the glacial till of central Iowa because the soil nails are of such small diameter.

A similar stabilization technique employs recycled plastic pins (RPPs) (Bowers and Loehr, 2000 and Loehr et al., 2000). RPPs have square cross sections 100mm by 100 mm (4 inches by 4 inches) and are 2.4 m (8 ft) long. This limits the application to relatively shallow slides; however they have been used successfully in Missouri at a cost of $42/m^2 ($1.20/ft^2) per unit area of slope face (1999 data). Because the majority of slides observed in this study have shallow failure surfaces, RPPs should be given consideration as a stabilization technique in Iowa.

Conventional piles such as steel H piles driven vertically to depths below the failure surface increase slope shear resistance. Arching within the soil mass provides stability between the piles. This technique has been successfully used to repair slides along the Des Moines River valley wall in Webster County (Sperry, 1999).

Stone columns provide slope stability by replacing or displacing unstable soil with vertical columns of compacted stones installed by either vibro-replacement or vibro-displacement. This technique is limited to cohesive soils with undrained shear strengths greater than 10 kPa (200 psf). If the undrained strength exceeds 50 kPa (1000 psf) stone columns may not be needed (Bachus and Barksdale, 1989).

Reticulated micropiles, also called root piles or pin piles, are reinforced concrete piles with diameters from 75 to 300 mm (3 inches to 12 inches). These piles are driven at various angles to form a three-dimensional root-like pattern within the soil mass that extends below the failure surface.

Soil reinforcement can be obtained by removing the weak soil and compacting the material in layers interspersed with layers of metallic or plastic strips arranged in a rectangular pattern. The soil grid interaction results in greater strength from friction acting longitudinally and passive resistance acting transversely. Geogrids composed of high strength polymers have been used succesfully in slope repair since the 1980's (O'Rourke and Jones, 1990).

8.2.4 Chemical treatment

Shear strength increase can also be achieved with chemicals including Portland cement, lime, and flyash. Chemical stabilization requires adequate laboratory studies prior to design (Abrahamson et al, 1995). In some situations weak soil can be excavated from the slope, mixed with Portland cement and or lime, and compacted to produce a high shear strength mixture. Another technique is to pressure grout the chemicals from boreholes placed in a grid pattern. A pervasive problem with grouting is the penetration of the stabilizing agents into the failure zone. Bedding planes, fractures and the discontinuity of the failure surface itself facilitate movement of the chemicals. The shrinkage cracks common in central Iowa glacial till might make this material a likely candidate for stabilization by chemical grouting. In addition to the pressure gradient from the grouting it is possible that stabilizing agents would diffuse in response to a concentration gradient according to Fick's law; this was verified by measurements of lime diffusion through plastic loess from Iowa (Davidson, 1964). Chemical movement by diffusion requires adequate moisture contents within the soil pores and sufficient time for the process to occur. Chemical treatment should be given further consideration as a possibility for slope repair in Iowa.

8.3 Iowa DOT Recommendations

Most landslide repairs can be broken down into four components for discussion of repair of different types of landslides. These components are geometric, materials, earthwork, and drainage. The following techniques have been successfully used by the Iowa DOT.

8.3.1 Geometric Components

Slope Flattening decreases the slope angle. The major impact of this repair option is to right of way (ROW), both in foreslopes shown in Figure 8.1 and backslopes shown in Figure 8.2.

Stability Berms, also called toe berms, load the toe of an unstable slope (Figure 8.1). This option also impacts ROW, but sometimes to a lesser extent than simple flattening. The key to this option is designing the berm such that the gravity component of the berm contributes more to resisting than driving forces. Stability berms are applicable in foreslope configurations.

Backslope Benches, analogous to stability berms, achieve the same goal as flattening, but with less ROW impact.

8.3.2 Materials Classification

The following materials are used in slope repair. Suitable earth fill consists of cohesive or granular material, or a mixture thereof, which is free of deleterious material and suitable for embankment construction. Relatively clean sand is used both as a free-draining layer in conjunction with drains, and as high-friction granular material in a shear key. Roadstone is crushed limestone that can be used as a high-friction granular material in a shear key. Erosion Stone is clean, crushed limestone material with a nominal size around 3-6 inches (76 to 152 mm) and used for shoulder fillets and erosion protection and could be used for rock buttresses. Riprap is clean, crushed limestone material, nominal size around 12 inches (305 mm) that is used for erosion protection and rock buttresses. Porous Backfill is clean pea gravel used as a filter material around perforated drainpipes and in drainage trenches. Porous backfill should be a minimum of 3 inches (76 mm) thick but preferably 6 inches (152 mm) thick around drainpipes.

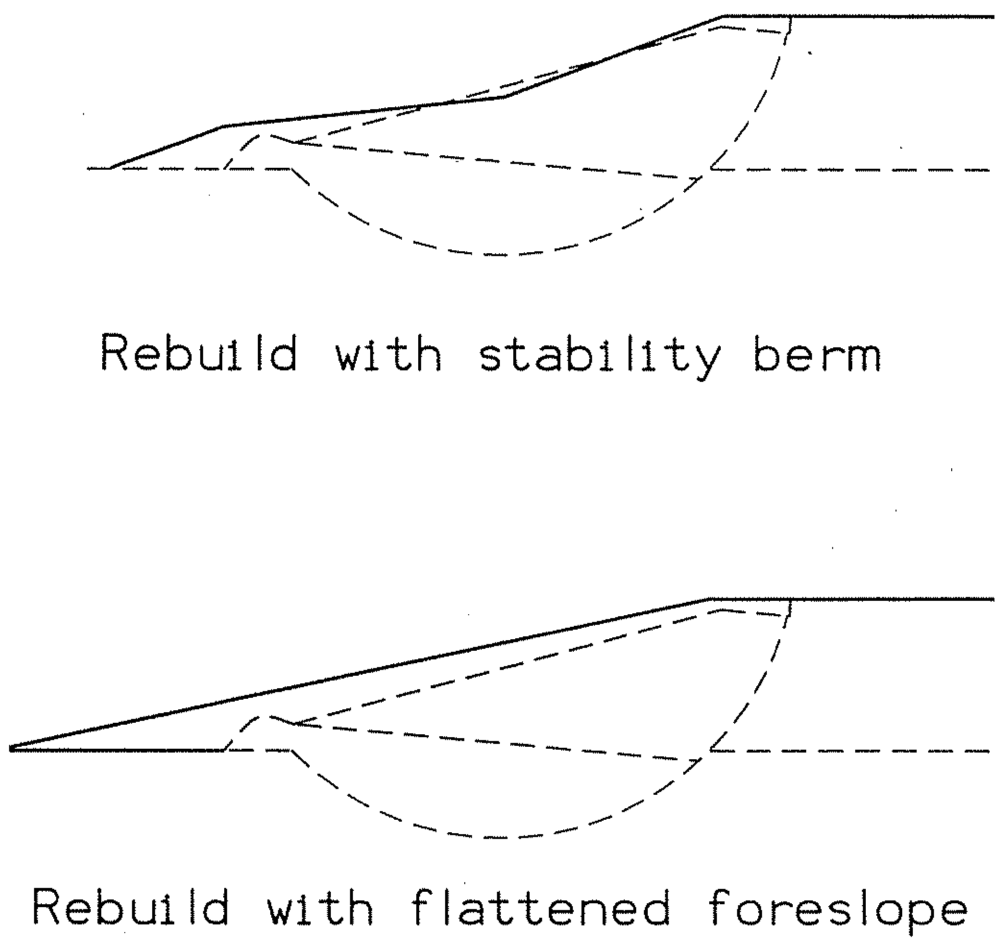
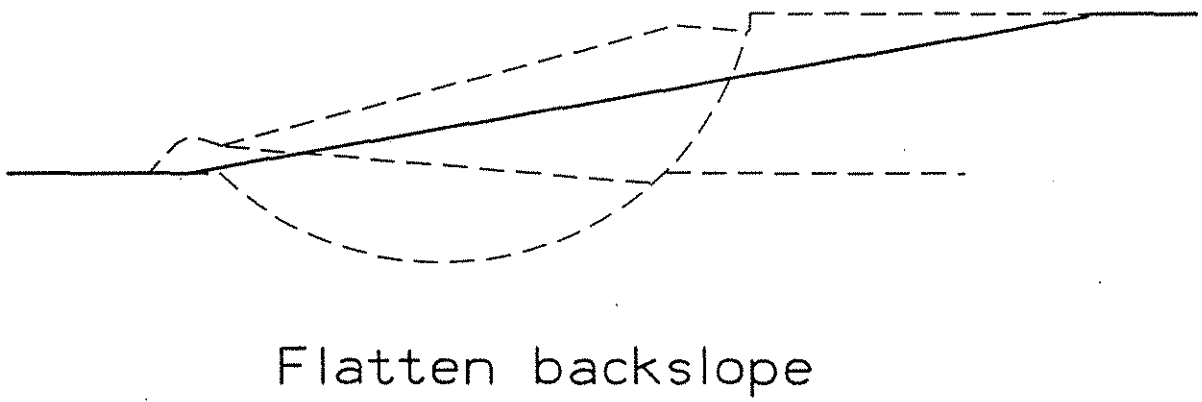


Figure 8.1 Slope repair with stability berm and flattened foreslope



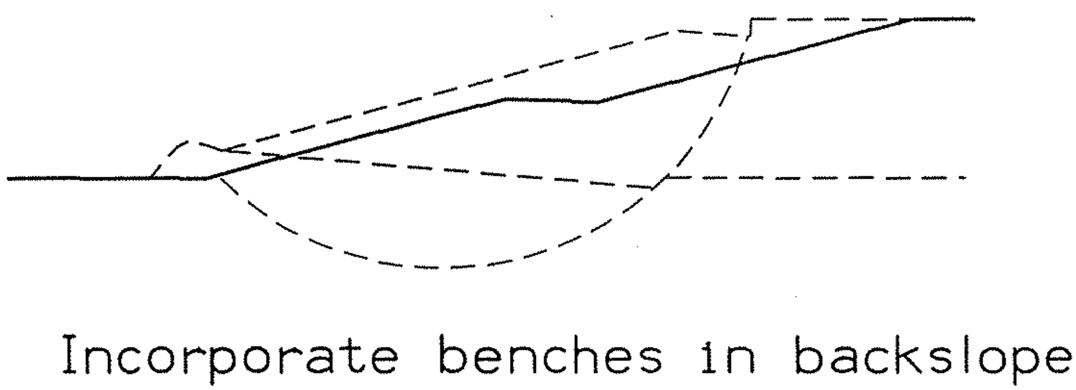


Figure 8.2 Slope repair with flattened backslope and benches

Perforated Drainpipe is typically 4 inch (102 mm) diameter and composed of PVC, polyethylene, or corrugated metal. Engineering (Filter) Fabric is placed between the interface of relatively fine-grained soils with coarse, open-graded material to prevent migration and infiltration of fines. Typically filter fabric is placed beneath riprap and erosion stone.

8.3.3 Earthwork

Excavation is usually associated with rebuilding a slope, especially in benching before fill placement. Extreme care should be exercised when performing excavation on a failed slope so as not to further de-stabilize the slope. Excavation can also be part of a solution, as in re-channeling a stream that is eroding a slope.

Benches should be constructed before placing new fill at a failed slope. This is accomplished by excavating a flat bench until a vertical dimension of about 5-6 feet (1.6 – 2.0 m) is achieved. The width of the bench is usually determined by the existing slope geometry. The exposed face is then cut back to an angle of about 1:1, and then another bench is constructed. This procedure interlocks the new fill with the existing slope and reduces the chance for a shallow failure at the interface of these two materials as shown in Figure 8.3. Benching need not and probably should not be performed all at once, but rather just in advance of new fill placement. This benching operation should be performed for any fill placed on ground sloping steeper than about 4:1. This applies to new construction as well.

All suitable earth materials used to rebuild a slope should be appropriately compacted. Larger materials like riprap are not compacted. When rebuilding a slope, the placement of new fill should begin at the toe and progress upward.

Heavy equipment should be kept on the toe side of the repair whenever possible. Equipment should not be placed at the top of the slope, as this may further exacerbate the situation.

8.3.4 Drainage

Internal longitudinal drains are typically incorporated in rebuilt foreslopes with a known or suspected water problem. The type and number of drains will depend on the extent of the water problem and the resources and equipment available. The flat benches formed during reconstruction are the preferred platform for drain installation as shown in Figure 8.4. If a water problem is suspected, it is a lot easier to install drains during reconstruction than to retrofit them at a later date.

Backslope drains are longitudinal drains used for one of two purposes as shown in Figure 8.5. The more common of the two is to collect water that moves along the contact between two different geologic units, such as loess/till or alluvial clay/shale. Another use is to draw down a high water table through a backslope cut. Backslope drain trenches should be backfilled with porous backfill but the top 1-2 feet (0.3 to 0.6 m) should be capped with a cohesive, relatively impervious material to prevent infiltration of surface water.

Transverse drains (Yugoslavian Drains) are constructed from the toe up to some point mid-slope. They are usually constructed using a backhoe with a bucket width of about 2-4 feet (0.6 to 1.3 m). The trench can vary in depth (4-8 feet (1.3 to 3.2 m)) from mid-slope to toe, but the bottom of the trench should allow gravity flow of water. They are typically spaced about 20 feet (6 m) on center. The trench is then backfilled with riprap or erosion stone, depending on availability and trench size. Engineering fabric should be placed beneath and adjacent to the stone. There is typically no collector pipe in this type of drain. See Figure 8.6.

8.3.5 Buttressing

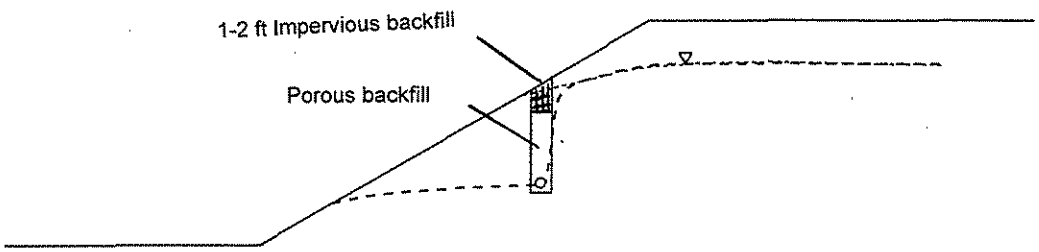
A rock buttress is a reconstructed slope using only large stone material, such as riprap or erosion stone as fill material as shown in Figure 8.7. Fabric to prevent loss of fines should underlie the stone.

Granular (Shear) Keyways are constructed by excavating a keyway through weaker material into stronger material, and backfilling the keyway with high-friction, granular material. This longitudinal feature provides high shear resistance at the toe of the slope. This can be accomplished with extensive excavation shown as option 1 Figure 8.8 or with less excavation shown as option 2 in Figure 8.8.

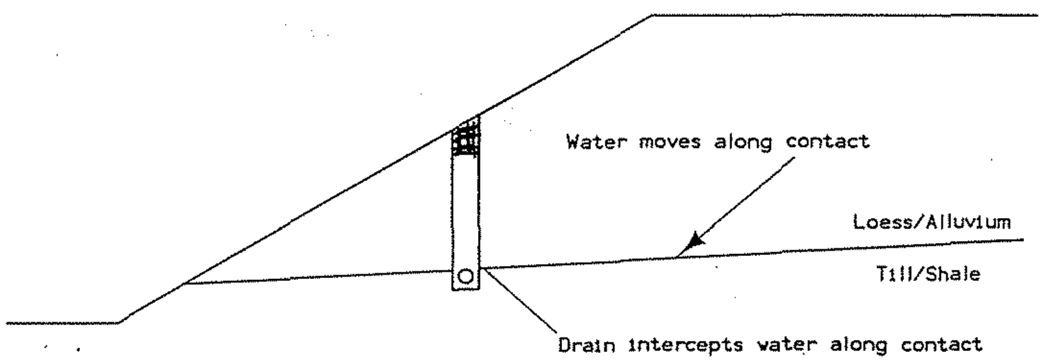
8.3.6 Erosion protection

Shoulder fillets consist of constructing small rock buttresses beneath the shoulder of the road with erosion stone. Fabric is placed between the earth fill and the stone. This buttress helps prevent localized erosion due to surface runoff as illustrated in Figure 8.9.

A revetment protects the toe of a slope from erosion by open channel flow due to stream or even ditch flow. The size of revetment material, from erosion stone to riprap, depends on the quantity and velocity of water flow. Fabric should underlie the revetment.



Option 1. Drawdown high water table



Option 2. Intercept water moving along contact layer

Figure 8.5 Two types of backslope drains

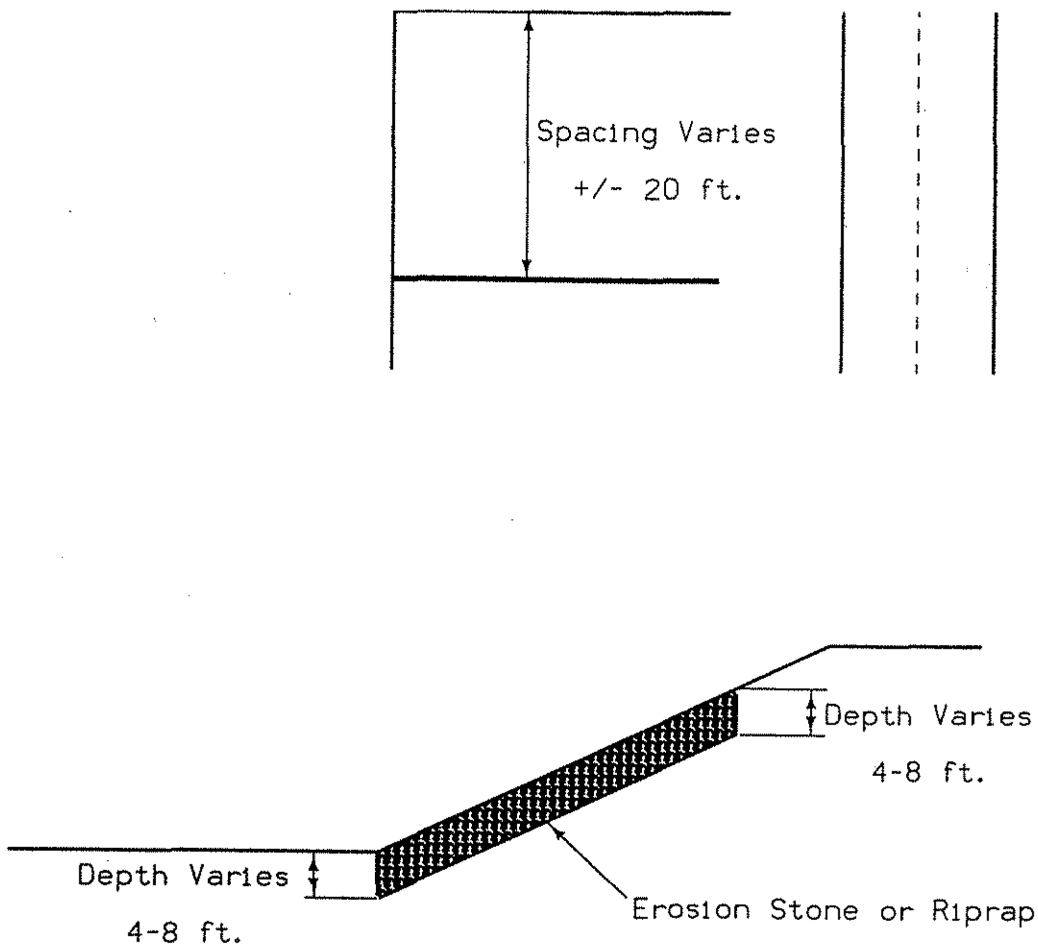


Figure 8.6 Transverse slope drains plan view at top section view below

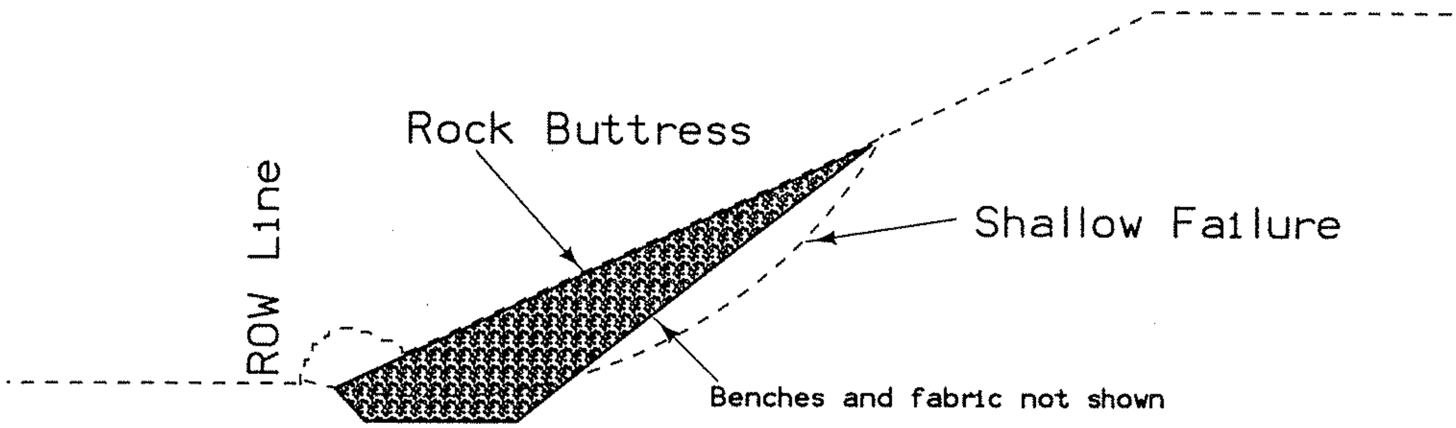
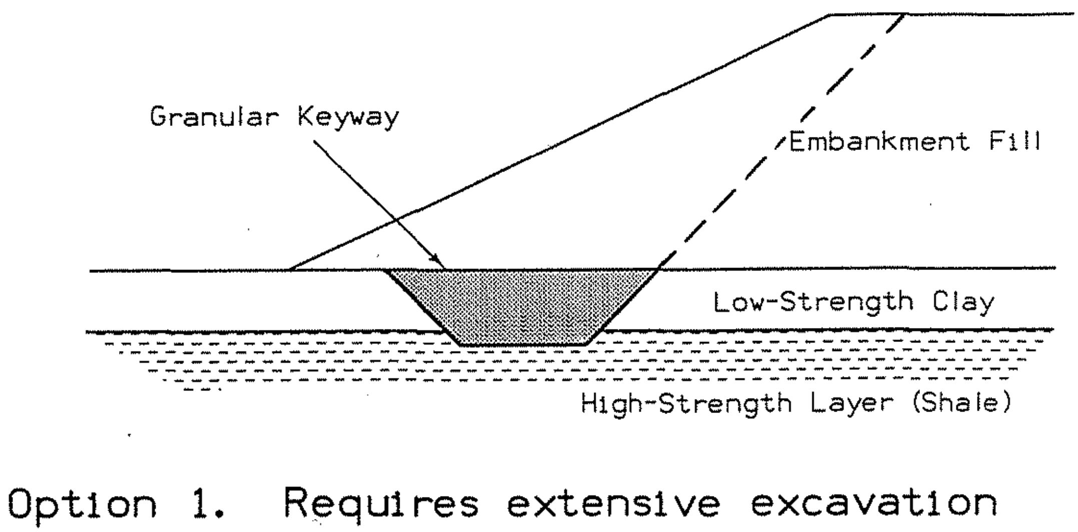


Figure 8.7 Slope repair with rock buttress



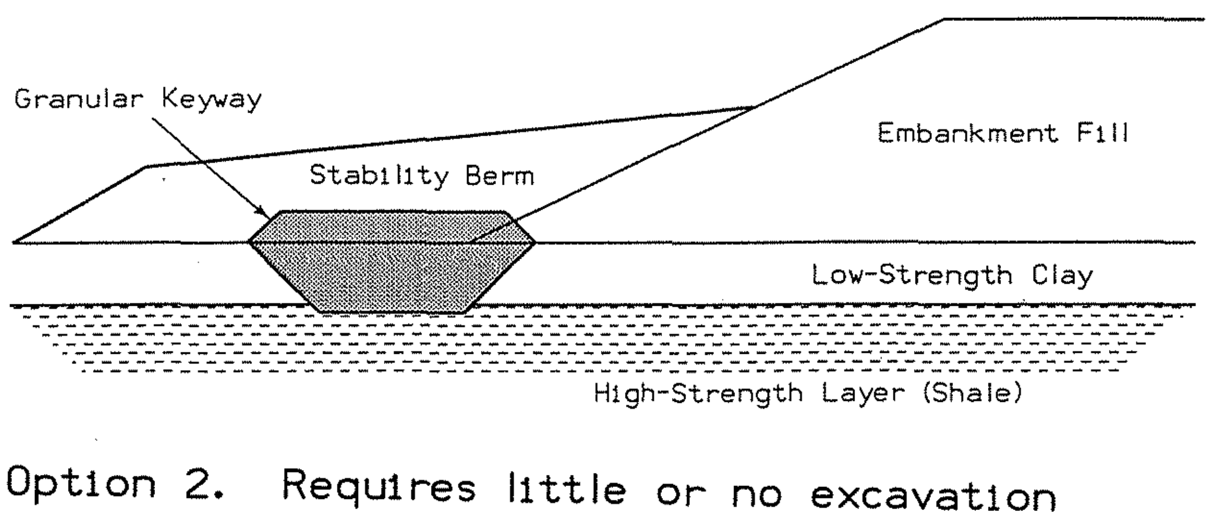


Figure 8.8 Slope repair with shear keys

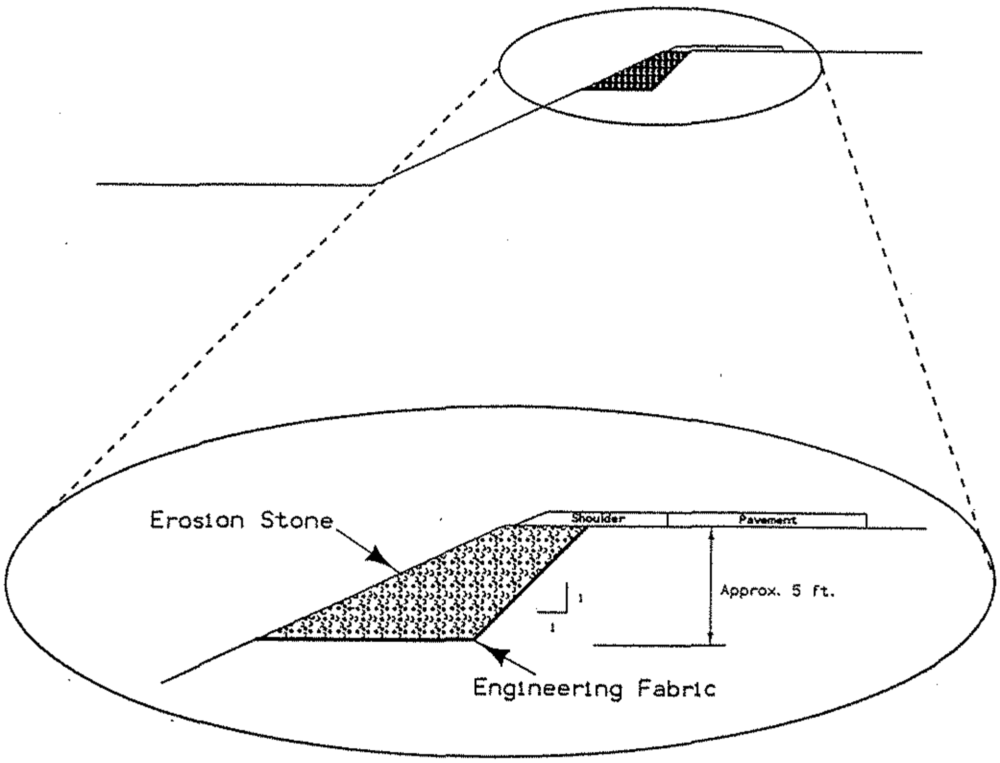


Figure 8.9 Protection from shoulder erosion

Rock Flumes are used when channelized water must be let down the slope. A flume is shaped and lined with erosion stone or riprap, which is of course underlain by fabric. An alternate to a rock flume is a paved concrete flume.

All exposed earth should be seeded to establish vegetation at the end of construction. Often, temporary slope protection will be required to help the vegetation become established.

8.3.7 General repair procedure

The first step in designing a slope repair is to determine the cause of the failure. The cause may be a single item, or it may be a combination of several. The cause of a failure is often not apparent, so the goal is then to determine the most likely causes and address those with the repair solution. The engineer should visually inspect the site to check for potential causes. Local maintenance people and even nearby homeowners can be valuable sources of information.

A good start is to determine if water was a possible contributor. If so, was it most likely groundwater, or was it surface flow that was able to infiltrate the subsurface? If surface water was involved, the design must correct the surface flow of water to prevent this. If a broken utility or similar source is responsible, this must be corrected. If the culprit is groundwater, then drains must be installed to remove the water from problem areas. The only other alternative is to perform the geometric design to provide adequate stability under wet conditions. If it is feasible to do so, drains should be installed to reduce the influence of water on the slope. Water is at least associated with, if not the main cause of, most observed slope failures.

The engineer should note the height and angle of the slope before and after the failure. Was the failed area higher or steeper than adjacent unfailed sections? Has material recently been removed from the toe, either by human activities or natural processes? Has the road been widened at the top of the slope? The most common geometric cause is probably unloading the toe of the slope.

It should be determined if any detrimental activity has recently occurred. Has heavy equipment been placed at the top of slope, or has it operated along the slope? Road widening, a potential geometric cause, also brings traffic loadings closer to the top of the slope. Has any material been stockpiled at the top of the slope? The repair of the slope should compensate for any loading conditions that could be repeated in the future.

The selected repair method should adequately address the causes (or suspected causes) of the failure. Since the cause of slope failure may be somewhat unclear, a certain amount of conservatism is usually warranted in the design. The repair can be selected from the repair components listed earlier, addressing as necessary geometric, drainage, buttressing, and erosional issues. The next section outlines the most common types of repair for different types of failures.

8.3.8 Specific issues by failure type

Shallow Slides

Shallow slides, for this discussion, are slides in which the failure surface is no deeper than about 6-8 feet (1.8-2.4 m) below the surface at the deepest point. If water was the cause of a shallow slide, and the problem may be chronic, internal longitudinal drains should be incorporated. Longitudinal internal drains should be placed at benches. The number of drains depends on the extent of the water problem and the height of the slope. Drains might be installed every third or fourth bench for a low to moderate problem, while a more severe problem might require drains on every other bench or even every bench. There really is no hard and fast rule. The slope may be rebuilt with suitable earth material to the original geometry, or the slope may be flattened. If the ROW allows, it is typically a good idea to rebuild at a less steep angle.

Another option worth trying in some circumstances is transverse drains. These are sometimes used for a very minor slide where rebuilding the foreslope might not be warranted. Installing transverse drains could be considered an attempt at a quick fix, with the understanding that if conditions worsen rebuilding the foreslope will be necessary.

When dealing with a shallow slide where a little additional stability is needed but it is impossible to flatten or berm the slope due to ROW limitations, a rock buttress might be considered. A rock buttress can be an effective repair for a shallow slide with limited ROW.

Deep Seated Slides

Deep slides, for this discussion, are slides in which the failure surface is deeper than about 6-8 feet (1.8-2.4 m) below the surface at any point. Deep slides are typically more complex and more expensive to repair. Almost without exception, deep slides require a geometric component to the repair, either flattening the foreslope or berming the toe.

When water is a contributing factor in a deep slide, which it often is, it is imperative to incorporate internal drainage with the slope repair. The problem, though, is that sometimes the failure is deep enough that it is difficult to install effective drains. The most prudent course of action in this situation is to install drains; although they may not drain the deep slide, they will help prevent future shallow slides. Drains are relatively inexpensive, and much easier to install when rebuilding the slope.

Granular keys can also be used to effectively stabilize a deep slide. They can be installed through a relatively weak foundation layer into a stronger one. Granular keyways work by interrupting the failure plane or potential failure plane. They are constructed by excavating a keyway and backfilling with high-friction granular material, such as sand or roadstone. The penetration of the underlying strong layer is usually about 2-3 feet (0.6 – 1 m). The effect of overburden on the key is to increase shear resistance through the granular material. For this reason, the keyway must either be beneath the embankment, or beneath a stability berm on the toe side of the embankment. Because installing a keyway beneath the embankment requires extensive excavation, the berm option is preferable for foreslope applications.

8.3.9 Backslopes

Most of the previous section dealt with issues for foreslope slides. The major difference between foreslope and backslope slides is that backslope slides typically occur within natural deposits (cut slope) and foreslope slides occur within embankments (fill slope) over natural deposits. Foreslope slides are often considered more critical in nature because they can directly imperil traffic. Backslope slides are considered by many people to be more of a nuisance (depending on what is at the top of the slope!), but they should be repaired.

The main options for backslope repair are flattening or benching and backslope drains. Both geometric options affect ROW, but benching less so. Backslope benches might be incorporated about every 10-20 feet (3-6 m) of height, depending on the severity of the slide and ROW. Benches are typically about 10 feet (3m) wide.

The goal of backslope drains is to intercept water moving along a geologic contact, which is very often the cause of the slide. The material at the contact is weakened by this flow of water, and a translational type of failure usually occurs. This is common at the interface of loess over till, and is very common with materials overlying shale.

There are a few other options to consider for potential backslope repairs. If the roadway ditch is not being blocked by slide debris, the easiest repair is to simply buy additional ROW above the slide and allow nature to take its course. The reasoning here is that in some instances, acquiring additional ROW is less expensive than repairing the slide. This is applicable in situations where the failure is not progressive. Backslope slides involving shale should not be left to "fix" themselves. Shale is notorious for progressive strength loss after exposure to weathering.

Another option might be applicable when the backslope extends upward into cultivated ground or pasture. With a cooperative landowner, it may be feasible to flatten the backslope without buying additional ROW. The backslope is benched at the existing fence location, and flattened beyond the ROW. The fence is rebuilt through the benched area, topsoil is replaced on the cut slope, and the landowner can still use the land for crops or grazing. This falls in the category of "mutual benefit" (a variation of temporary easement) where permanent ROW acquisition is not utilized.

A special case sometimes encountered in backslopes is localized "blow outs" at sand lenses within cohesive material. This is common in cuts through glacial till. Water collects in the lens and exits on the slope face to cause a localized failure. A simple solution to this problem is to trench up the backslope to the blowout and backfill with porous material perhaps with a perforated pipe. This drain should outlet to the ditch. Once the area is stabilized, topsoil should be added if needed and the area re-vegetated.

8.3.10 Shale

Shale deserves special consideration in a discussion of landslides. Shale is a common bedrock in Iowa, but has very different characteristics from other Iowa bedrock. Shale is probably most associated with backslope slides, although many foreslope slides are caused by shale. Foreslope problems can occur in situations where shale or shale derived soils are incorporated in the embankment, or when a high fill is constructed above shale that is at or near the surface.

Shale is problematic because of its tendency to weather or disintegrate and lose strength in the presence of excess water. Water moves through the overburden material and collects at the shale surface, which is essentially impervious. Water flows along this contact, softens the top of the shale, and brings it to a very weak state. The underlying, unaffected shale remains strong and a translational, or block, slide can occur. Shale is also weakened by the removal of overburden, which is why problems in backslopes are even more likely.

The most effective repair for shale problems are backslope drains, as discussed in the previous section. Granular keyways, also discussed above, may be utilized to repair slides involving shale. It should be noted that slides on shale can occur on low, and even very flat slopes.

8.3.11 Erosion

Sometimes slope disturbance, even severe slope disturbance, is not the result of mass movement (landslide) but rather the effect of erosion. Erosion disturbance is usually obvious by visual inspection. Severe erosion of the slope can lead to landslides. Slopes that have been severely eroded should be rebuilt in the same way that slope failures are repaired. The water source for the erosion should either be diverted, or the slope armored to prevent erosion. Slope toes should be protected with revetment.

A special case of toe erosion that sometimes occurs is the parallel stream condition. This occurs when a stream crosses under an embankment through a culvert, but flows parallel to the embankment toe for some distance on one or both sides of the culvert. If the stream continues to meander towards the toe, a slope failure is inevitable. This situation is illustrated in Figure 8.10. The optimum solution is to straighten the channel and incorporate enough revetment to protect the slope toe. If straightening the channel is not an option, the entire bank along the endangered toe may be protected with revetment. Depending on the actual configuration, this could be very expensive.

Sometimes erosion occurs from water traveling down from the top of the slope. When this is concentrated, as at the sag of a vertical curve, severe erosion is possible. An excellent solution to this problem is the shoulder fillet. The shoulder fillet is a small buttress constructed at the top of a slope to prevent erosion. Not only is the rock less susceptible to erosion, it has the effect of slowing down water, dissipating energy that could cause erosion below the buttress.

All repairs involving rebuilding slopes should include seeding to establish vegetative cover. The slope should also be protected as necessary until the vegetation is established. Failure to protect the slope from erosion will usually lead to a failed repair.

8.3.12 A note regarding loess and drainage

It should be noted that drainage issues in areas of friable loess are more complicated. It is Iowa DOT standard practice not to use porous backfill drains in areas of friable loess. Experience shows that porous backfill will become clogged by the intrusion of fines over time. It is therefore preferable to use “fin” drains, which are composed of a narrow core of

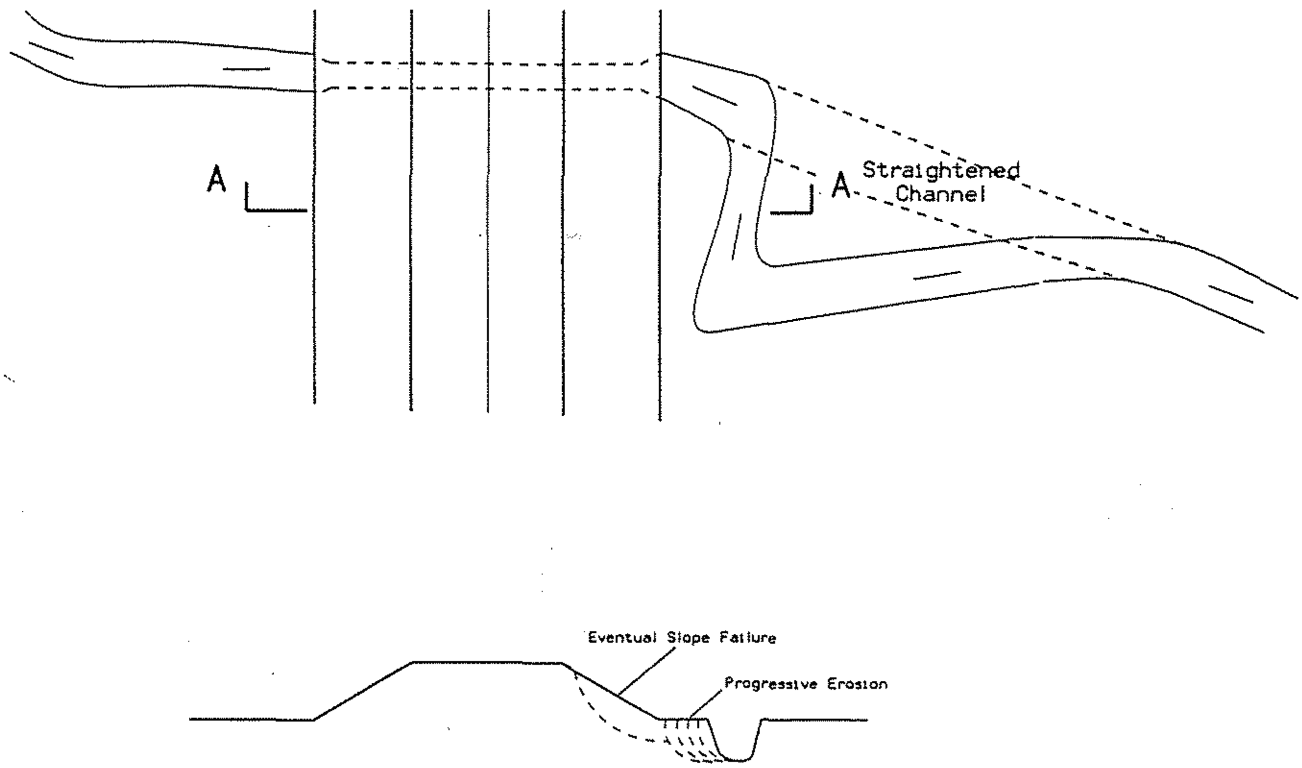


Figure 8.10 Stream or ditch realignment to avoid erosion at toe of slope

polyethylene surrounded by filter fabric. Alternately, the porous backfill could be wrapped in filter fabric. Local experience should govern in these types of matters.

8.3.13 Extent of Repair

The question always arises. How far should the repair be extended? Too little will not solve the problem and too much repair will be too expensive. Virtually all of the features discussed here affect a transverse section, so they can be extended to fit any length of slope. Drainage features can be outlet beyond the area of concern. Geometric features can be transitioned out beyond the area of failure, ultimately it is up to the good judgment of the designer.

There are obvious cases where the repair should be extended. For example, if a deep-seated failure occurs adjacent to a higher or steeper section, it would be prudent to extend the repair through the apparent high risk section. This is true if a localized explanation for the slide cannot be determined. Many new slides have been "just beyond" a previous slide that was repaired.

8.3.14 Should You Go It Alone?

Another question that arises is whether the county engineer should design the repair alone or get help. The answer to this depends on the individual's understanding of the subject. The general practitioner could certainly address erosional features, and probably a lot of shallow slides as well. The repair of deep-seated slides, however, is not for the novice. These types of repairs can entail extensive collection of subsurface information and complex geotechnical analysis. Even if you must go shopping for services, a better understanding of the subject makes one a better consumer.

8.3.15 What Not To Do

Sometimes concepts are more easily understood by identifying things that should not be done. A few examples of bad practice are listed below.

When a slide occurs, do not simply push the bulge at the toe back to the top. This unloads the toe and loads the top, and the failure will recur. Another issue is that once a slide occurs, the slip plane is weakened and simply returning the slope to its original geometry will result in a lower factor of safety than before.

If a slide blocks a drainage ditch, do not continue to remove material that slides into the ditch to keep the ditch clean. Continued soil removal unloads the toe and provokes further sliding.

Do not mistreat a problem. If the slide is deep-seated, do not simply rebuild to the original geometry, because the cause has not been addressed and subsequent failure occur again.

Do not forget to treat all causes. If you have a slide that occurred due to erosion of the toe, be sure to address that in your repair. Besides rebuilding the slope, make sure to divert water flow or armor the toe to prevent recurrence. Nobody wants to rebuild a slope twice (and absolutely nobody wants to pay for it).

8.3.16 Additional Methods of Repair

The methods of repair outlined in previous sections are based on IDOT experience with standard, and time-tested methods. The following list includes other technologies that have been used for slope repair, but have not been used by Iowa DOT to date: soil nailing, horizontal wick drains, minipiles, stone columns, reinforced steepened slope (RSS), and chemical treatment.

8.3.17 Maintenance Issues for Slope Stability

General

The theory of maintaining stable slopes is to be aware of causes of instability, and then prevent that from happening. Maintain records of problem areas and repairs so that a historical database can be established. Routinely inspect areas of known instability, especially during the spring thaw and significant rainfall events. Records of repairs and observations inform all involved staff of these areas; and if a slide does occur, this information is invaluable in designing a repair.

Specific Issues

The following are the most common issues associated with slope stability maintenance. Figure 8.11 illustrates the key issues discussed here.

Identify erosion problems early and repair with approved methods. Watch for potential problematic erosion, as in the case of the parallel stream discussed earlier. Inspect foreslope toes at stream crossings regularly, and especially during and after times of high flow.

The maintenance of vegetation is very important for slope stability, as it is directly related to erosion. This applies not only to ground cover, but to larger trees and shrubs as well. The root systems of these larger plants help hold the soil mass together, which is typically

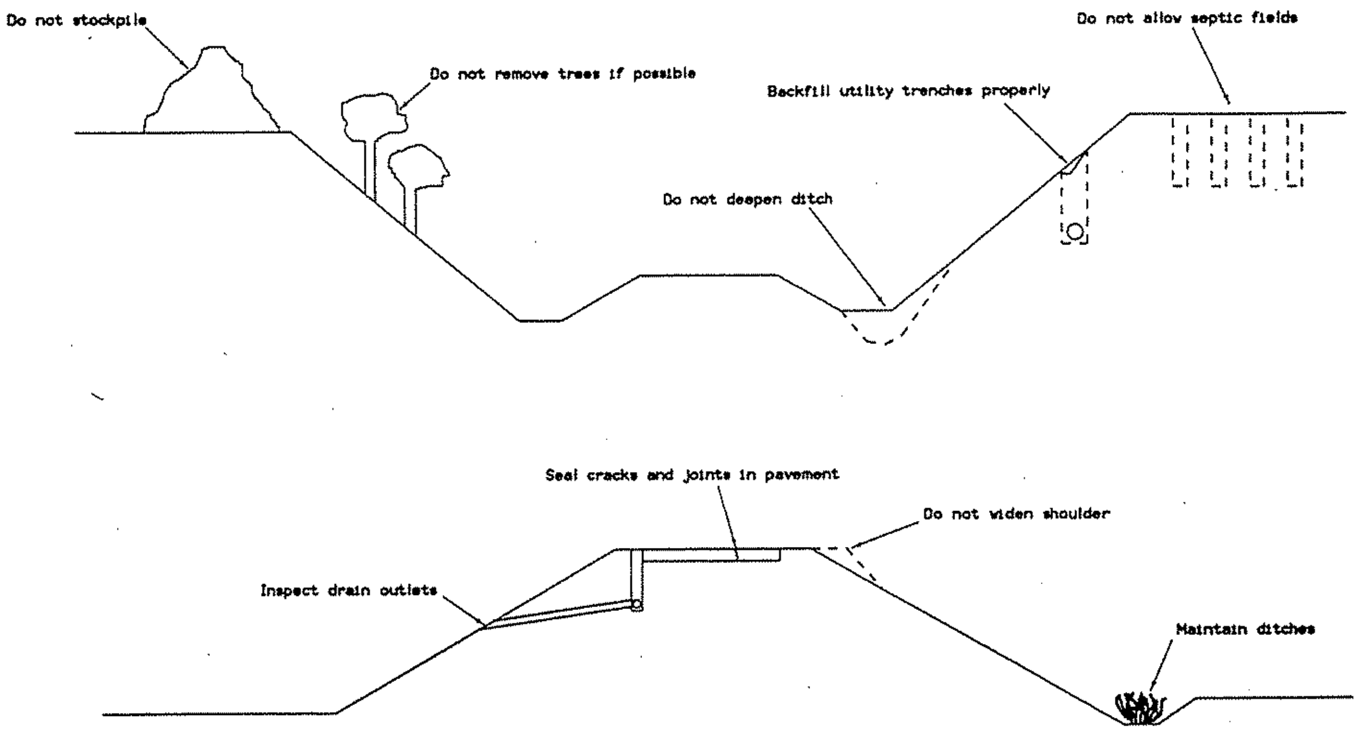


Figure 8.11 Summary of maintenance issues

beneficial for slope stability. That is why it is a good idea to leave established trees and shrubs on backslopes if possible.

Part of vegetation maintenance involves mowing operations. This is especially important in ditches, where overgrown vegetation can restrict ditch flow, resulting in higher flow velocities and potential erosion. Mowing operations on slopes should be limited to times when the slopes are relatively dry and stable. Heavy equipment on saturated slopes can cause rutting and shoving of the slope face. This in turn provides areas for water ponding and infiltration, and a potential progressive failure.

Existing subdrains should be periodically inspected to ensure proper functioning. Over time, drains can be clogged with silt, vegetation, or nesting animals and cease to function. This applies to drain outlets for previous slope repairs as well as longitudinal pavement subdrains and median drains. Culverts should also be inspected periodically.

Careless earthwork operations associated with utility installation can be very detrimental to slope stability. Inspection during backfilling will ensure proper compaction. Utility trenches should be backfilled in such a way as to prevent infiltration of surface water, and to minimize subsidence, which can lead to water ponding problems. This is especially important if the utility trenches penetrate shale. Trench backfill should also not significantly alter existing subsurface water flow. Septic drain fields should not be allowed at the top of backslopes where they could introduce excess water into the subsurface.

Proper pavement maintenance includes sealing cracks and joints to minimize water infiltration. This is especially important if the pavement does not have a porous base and subdrain system. When vehicles leave the pavement and rut the shoulder, they create a depression that will fill with water. Over time this ponded water will soften the surrounding subgrade and de-stabilize the slope. When edge ruts occur, repair them as soon as possible.

Another feature that can develop adjacent to the roadway is a small curb that inhibits surface drainage. This curb might be the grader ridge on a gravel road, or it might be a mixture of soil, vegetation, and garbage that accumulates along a paved road by snow plowing and other activities. The effect of this curb is similar to the edge rut, with standing water softening the shoulder and leading to further problems.

There are a few issues that do not necessarily fall under the previous headings, but nonetheless merit discussion. One very common practice involves widening the pavement or shoulder of a road without widening the embankment. This in effect is loading the top of the slope. Additionally, vehicle loads are brought closer to the edge, where they can be more detrimental.

Any action that loads the top of the slope or unloads the toe of the slope should be avoided if possible. For example, stockpiles should not be allowed at the top of a slope. Another action sometimes performed by maintenance personnel involves deepening ditches to improve ditch flow. This action unloads the slope toe, and creates the opportunity for additional toe erosion.

The task of slope stability maintenance is not complicated. It involves understanding what actions can lead to slope failures, being able to identify these in the field, and then acting to correct the problem before it leads to a large slope failure. Even with proper maintenance techniques, slope failures are inevitable. One of the keys is to prevent the slides that are preventable to preserve resources to deal with the slope failures that aren't. Maintaining records of activities and observances will ease the determination of future repairs.

9.0 IOWA CASE HISTORIES

Four sites representing both embankments (foreslopes) and cut or natural slopes (backslopes) were selected for further study. The objectives of this part of the research are to gain insight into slope stability problems that occur in Iowa and illustrate how slope stability analyses can be used to identify the cause of the instability. After the cause has been identified, the most effective remediation can implemented. More detail on these case histories can be found in Karnik (2001) and Chu (2001). The interpretations and recommendations in this chapter presume that the geotechnical data are accurate and that the assumptions of the stability analyses are appropriate for the various field conditions.

9.1 Foreslope Along Highway 34, East Albia, Monroe County

9.1.1 Location

The landslide is on the foreslope of an emabankment located on Highway 34 about one mile (1.6 km) southeast of Albia, Monroe County, Iowa. The landslide proximity to the highway shoulder poses a risk to public safety.

9.1.2 Geologic setting

The embankment is underlain by the Gosport soil series, which consists predominantly of silty loam and clay shale. The surrounding area has gentle to steep slopes and is moderately to poorly drained. The Gosport soil series was the borrow material for the embankment and is weathered from loess, glacial till and a residuum of acid shales. The soil is subject to severe shrink-swell behavior, and in general, has a low strength. (Oelmann, 1984)

9.1.3 Description of the slide

The foreslope, shown in Figure 9.1, is 24 ft (8 m) high at a 2:1 slope. The slide is classified as a translational slide in which a few blocks of fine grained soil have failed along a shallow failure plane about 2.5 feet (0.75m) deep. The failure plane is approximately parallel to the slope of the existing embankment. During fieldwork in April 1999, it was observed that the soil in the failure zone appeared wetter and with an aggregated structure that differed from the soil samples collected from the borings and the soil on the surface adjacent to the slide.

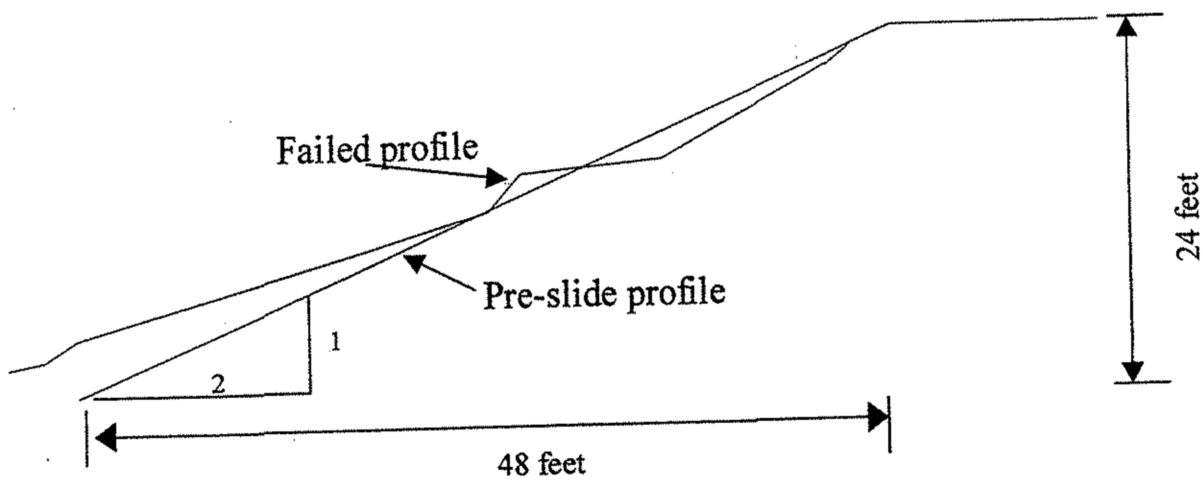


Figure 9.1 Profiles of original slope and landslide surface of Highway 34 slide.

9.1.4 Soil sampling

Because visual inspection indicated a wetter and aggregated soil in the slide, disturbed grab samples were collected along with undisturbed Shelby tube samples from two boreholes. The grab samples and Shelby tube samples were subjected to engineering index tests and mineralogic characterization by x-ray diffraction. The Shelby tube samples were tested to determine the shear strength parameters and unit weight of the soil. Subsurface logs for the borings can be found in Karnik (2001).

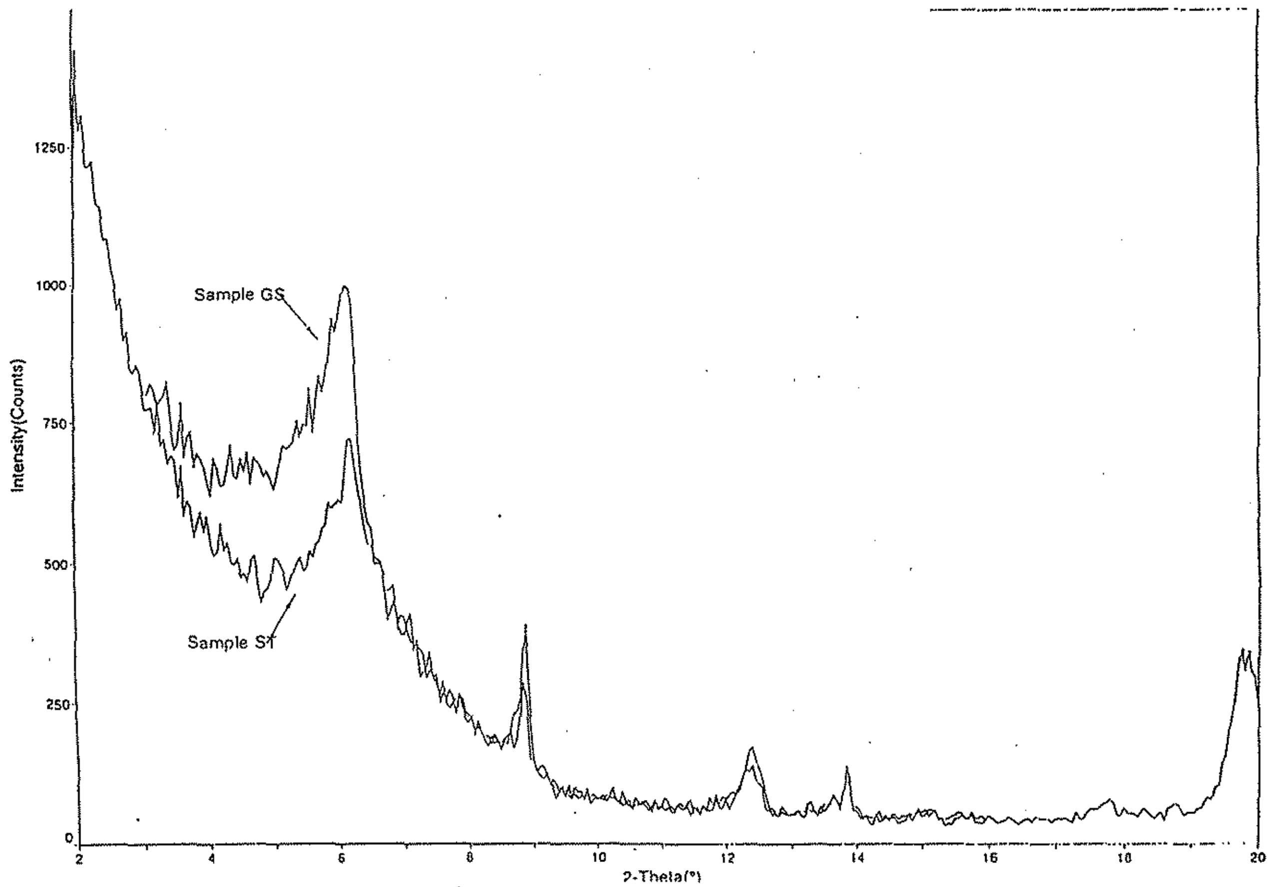
9.1.5 Geotechnical properties

Engineering Index Properties

Atterberg limit tests and mechanical analyses were run on grab samples and the Shelby tube samples. The mean values of liquid limit and plastic limit for the grab samples are 43.7% and 22.4% respectively while the mean liquid limit and plastic limit of the Shelby tube samples are 41.6% and 20.6% respectively. Both sets of samples have more than 82% of the soil particles passing the 0.075mm (No. 200). Based on the particle size distribution and the Atterberg limits, the soil samples classify as CL or CH by the Unified Classification System. According to AASHTO, these soils classify as A-6 and A-7-6 and fall into the category of cohesive embankment soils according to Bergeson et al. (1998). Engineering index properties and engineering classification of the surface grab samples and samples from the interior of the embankment are essentially the same. This was verified by statistical analysis that used the "t test".

Characterization by X Ray Diffraction

X-ray diffraction tests were carried out on both the grab and Shelby tube samples to determine their mineral composition. Figure 9.2 shows an overlay of the X-ray diffractograms from the grab and Shelby tube samples. The composite diffractograms show that both the soils are similar in mineralogy and composed of clay minerals, quartz and feldspar. The clay minerals were dominated by montmorillonite (Schlorholtz, 1999). This supports the description from the soil survey report (Oelmann, 1984) of the Gosport soil series having high shrink-swell potential on drying or wetting. The diffractogram from the grab sample shows a slightly broader montmorillonite peak, which may indicate the presence of intracrystalline moisture and perhaps a different exchangeable cation in the failure zone soil. This subtle difference could account for a lower shear strength in the soil at the surface of the embankment. Other studies have shown



X-ray diffractograms of Shelby and Grab samples (Schlorholtz, 1999)

Figure 9.2 X-ray diffractograms of tube samples and grab samples from Highway 34 slide.

that shear strength of clays and shales varies with adsorbed cations (Steward and Cripps, 1983; Moore, 1991, and Mitchell, 1993)

Compaction test data

Standard Proctor tests were conducted on composite samples to determine dry unit weight and the optimum moisture content so that the unit weights of the compacted soil in the foreslope, determined from the Shelby tube samples, could be compared with specifications. Standard Proctor test results are shown in Figure 9.3. The optimum moisture content is 19% and the corresponding maximum dry unit weight is 105pcf (16.5kN/m^3). The Shelby tube sample moisture contents and dry unit weights are plotted with the Standard Proctor test results in Figure 9.3. A "compaction acceptance zone" of 95% maximum unit weight was marked on the Standard Proctor curve to demarcate the acceptable range of dry unit weights and corresponding moisture contents (Bergeson et al, 1998). Nearly 54% of the samples from the interior of the embankment are below 95% compaction. The moisture contents of most of the samples are higher than the optimum, which suggests that the soil became saturated subsequent to construction.

Shear Strength Characteristics

Consolidated undrained (CU) triaxial tests with pore pressure measurements were carried out on eight Shelby tube samples obtained from the interior of the embankment. The samples were backpressure saturated before testing and pore pressures measured. The results were used to plot the deviator stress versus strain curves. Maximum values from the stress-strain curves were interpreted as the shear strength and used to plot the effective stress K_f line and the "a" and "α" values were obtained by linear regression as shown in Figure 9.4 (a). The plot shows the soil to have a cohesion intercept "c" of 313psf (15kPa) and an angle of internal friction "ϕ" of 19 degrees with a R^2 value of 0.74.

It has been observed that if embankment compaction is less than 95% the shear strength is likely to be lower (Bergesson et al, 1998). Of the eight samples tested, four samples had dry unit weights less than 95% of the maximum dry unit weight. The data from these samples were combined to obtain the shear strength parameters. The regression (Figure 9.4 (b)) shows the low density soil to have a "c" of 25psf (1.20kPa) and a "ϕ" of 25 degrees with a correlation coefficient (R^2) of 0.86.

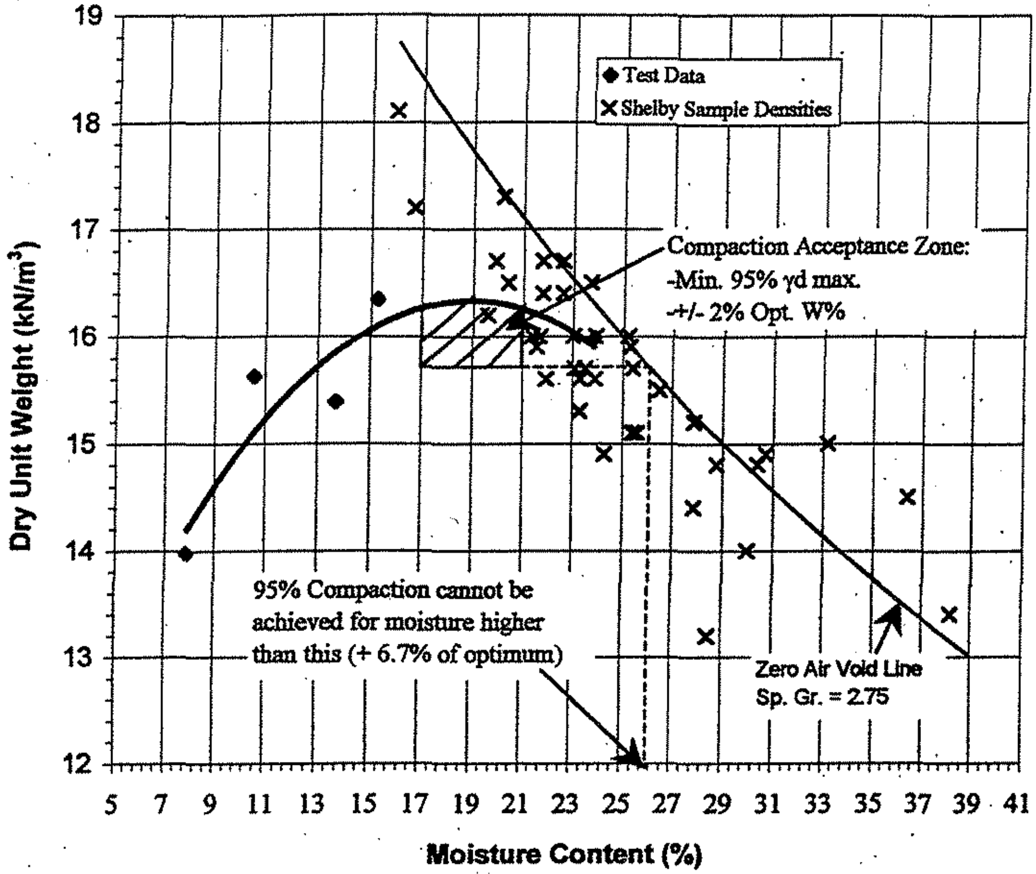
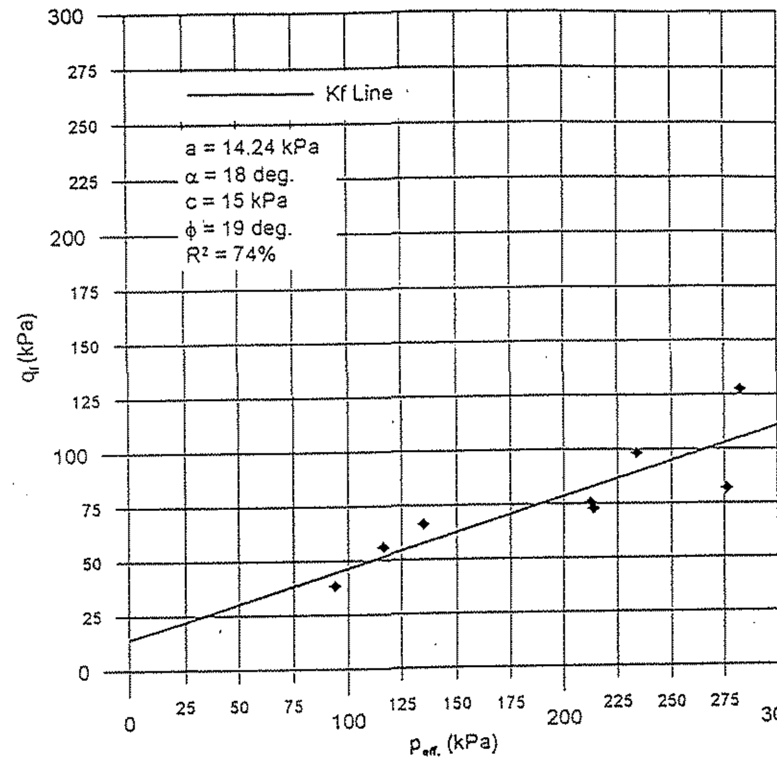


Figure 9.3 Compaction curve and field densities of embankment soil at Highway 34 slide.

(a)



(b)

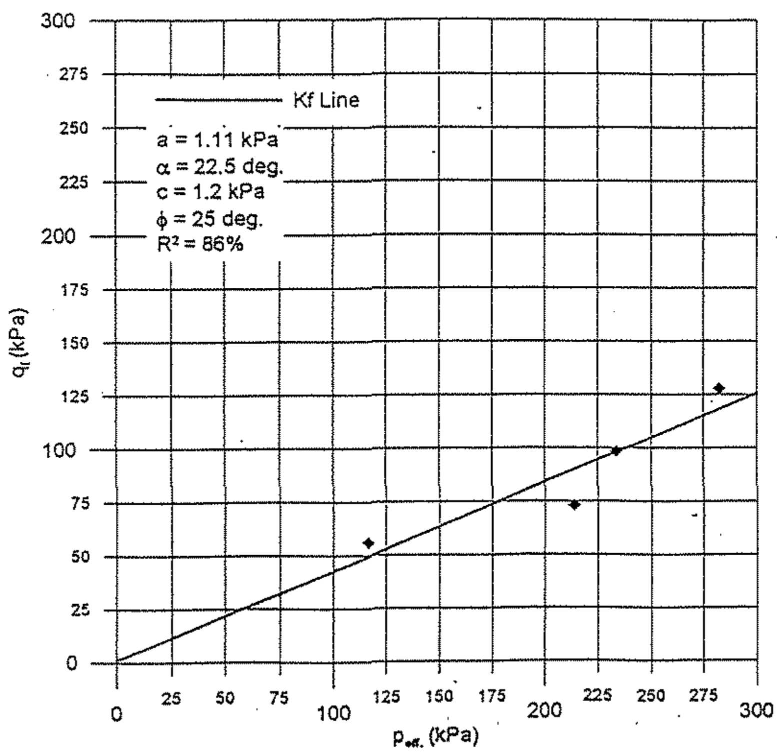


Figure 9.4 a) Strength (effective p-q) graph for all tube samples. b)Strength (effective p-q) graph for soil samples less than 95% maximum dry density

Another regression was carried out on the remaining four samples with dry unit weights greater than 95% of maximum dry unit weight. The regression showed the soil to have a "c" of 587psf (28kPa) and a "ϕ" of 12 degrees with a R^2 value of 0.80. Stability analyses for long-term stability were conducted for each set of strength parameters. It is not surprising that the soil strength varies with degree of compaction with higher relative compaction associated with higher strength.

Unconsolidated undrained tests (UU) with backpressure saturation were also carried out on other undisturbed samples. The values of shear strength parameters obtained were correlated with the initial void ratios of the samples. However, no correlation was observed between the values. It is therefore concluded that the wide range of undrained strength relates to the mineralogy and characterizes the inherent variability of the shear strength of the material in the embankment. For end of construction analysis, an average value of 835psf (40kN/m^2) was used as the undrained shear strength value.

9.1.6 Analysis

To determine the probable cause of slope failure, a series of calculations were carried out considering the slope as infinite with a failure plane parallel to the surface and 2.5 feet (0.75m) deep. The analyses were conducted for a moist slope and a fully submerged slope with seepage parallel to the failure plane. The total unit weight was used for the analysis without seepage, while the saturated unit weight was used for the analysis of infinite slope with seepage. XSTABL analyses were also done using the Modified Janbu method for non-circular failure surfaces because the slide is translational. The XSTABL analysis was carried out for a slope at natural moisture conditions, a slope with the water table elevation at the toe and at the mid height of the slope and a fully submerged slope. The analyses were conducted using the effective stress parameters as obtained from the three sets of strength parameters. The failure surfaces for the various XSTABL analyses are in Karnik (2001).

Case 1 in Table 9.1 shows the results of the infinite slope analysis done using the effective strength parameters obtained from the composite data. The analyses with and without seepage give factors of safety of 2.6 and 2.9, respectively.

	Factor of safety		
Type of analysis	Case1	Case 2	Case 3
	c=15kPa	c=28kPa	c=1.2kPa
	$\phi = 19^{\circ}$	$\phi = 12^{\circ}$	$\phi = 25^{\circ}$
Infinite slope analysis	2.9	4.7	1.0
Infinite slope with seepage	2.6	4.5	0.6
Taylor's analysis, $s_u = 40$kPa		6.3	

Table 9.1 Factors of safety for infinite slope analysis and Taylor's analysis for undrained strength

The minimum factors of safety obtained for XSTABL analysis done using the composite shear strength values are in Table 9.2 (Case 1) and factors of safety range from 1.6 to 2.2 for the fully submerged to natural moisture cases. The failure surfaces obtained are also very deep and do not approximate the failure in-situ.

	Factor of safety		
In-situ conditions Assumed	Case 1	Case 2	Case 3
	c=15kPa	c=28kPa	c=1.2kPa
	$\phi = 19^{\circ}$	$\phi = 12^{\circ}$	$\phi = 25^{\circ}$
Natural moisture	2.2	2.7	1.4
Water table at toe of slope	2.2	2.7	1.4
Water table at mid height	1.8	2.5	1.1
Fully submerged slope	1.6	2.3	0.8

Table 9.2 results of XSTABL analyses for different soil characteristics and groundwater conditions

Table 9.1 (Case 2) shows the results of the infinite slope analysis done using the effective stress strength parameters obtained from the data having dry unit weights greater than 95% of maximum dry unit weight. The analyses with and without seepage give factors of safety of 4.5 and 4.7, respectively. The minimum factors of safety obtained for XSTABL analysis using these shear strength values are in Table 9.2 (Case 2) where the analyses give factors of safety in the range of 2.3 to 2.7 for the fully submerged to natural moisture cases. The failure surfaces obtained from the analyses are very deep and do not approximate the observed failure conditions.

A third set of analyses was carried out using the effective stress strength parameters obtained from the samples having dry unit weights less than 95% of maximum dry unit weight. Table 9.1 (Case 3) shows the results of the infinite slope analysis. The analyses with and without seepage give factors of safety of 0.6 and 1, respectively. The factors of safety obtained for XSTABL analysis using these shear strength values are in Table 9.2 (Case 3). XSTABL analysis gives minimum factors of safety in the range of 0.8 to 1.4 for the fully submerged to natural moisture cases. The failure surfaces obtained are shallow and approximate the failure in-situ. These analyses indicate that the slope would have failed due to the combined effects of low shear strength from improper compaction if the slope is fully submerged.

9.1.7 Observations and conclusion

Of the 20 stability analyses, only 2 with seepage parallel to the slope or a fully submerged slope give minimum factors of safety of 1.0 or less. These groundwater conditions are unlikely in this embankment. The analyses clearly indicate that the slope is less stable if the soil is compacted less than 95% relative density; however those low strengths must be combined with unrealistic soil groundwater conditions to produce failure conditions.

It is interpreted that the most likely cause of failure is the reduction of cohesion near the embankment surface after construction. Weathering of the soil or some other mechanism such as freeze-thaw near the surface, coupled with improper compaction could have lead to the reduction in cohesion. The engineering index properties as well as the x-ray diffraction tests indicate no major difference in the soil near the surface and the soil within the embankment; however, x-ray diffraction tests indicate montmorillonite as the major clay mineral. This clay will swell on coming in contact with water and have a strength reduction. The broadening of the montmorillonite peak observed in the X-ray diffractogram of the grab samples supports this interpretation.

If the reduction of strength is due to weathering and saturation leading to the subsequent swelling of the montmorillonite is the cause of failure, then remediation such as flattening the slope would be ineffective in the long term. The use of other soil strengthening by geosynthetics, minipiles, or chemical stabilization would provide better, long term stability.

9.2 Foreslope Along 160th Street, Page County

9.2.1 Location

An investigation was conducted of an embankment failure located on 160th Street between Bethesda and Clarinda, about one and a half miles (3 km) east of the County Trunk Road M63, Page County. The road has a gravel surface and light traffic; however, the scarp of the slide has cut nearly 3.5 feet (1.2 m) into the road, thereby decreasing the available lane width. The objective of this analysis is to investigate the probable causes of failure of this embankment.

9.2.2 Geologic setting

According to Prior (1976), the site is situated in the Southern Iowa drift plain. This area consists of glacial till deposited during the Kansan stage. Subsequent to the end of the Kansan glaciation, the till was overlain with a thick deposit of loess. In places this loess mantle is thick enough to provide additional relief and to alter slope angles, particularly on leeward hill slopes and along stream valleys.

9.2.3 Description of the slide

The landslide occurred in an embankment about 35 feet (11 m) high. The embankment is over a culvert, which shows signs of degradation at the outlet. It was also observed that the creek downstream of the culvert is incised in glacial till. The original foreslope was 1.7:1, and visual inspection of the site indicated a curved failure surface about 21 feet (6.5 m) in length with a scarp about 5 feet (1.6 m) high. This slide is a shallow rotational slide and a typical profile of the slope is shown in Figure 9.5. The failure plane appears to pass through the toe of the slope.

A wet area approximately 10 feet (3 m) wide parallel to the road and 21 feet (6.5 m) in length, perpendicular to the roadway, was observed over the failure zone about 56 feet (17 m) down the slope. This was anomalous because of the low amounts of precipitation in 1999 and 2000. The source of the moisture is not known.

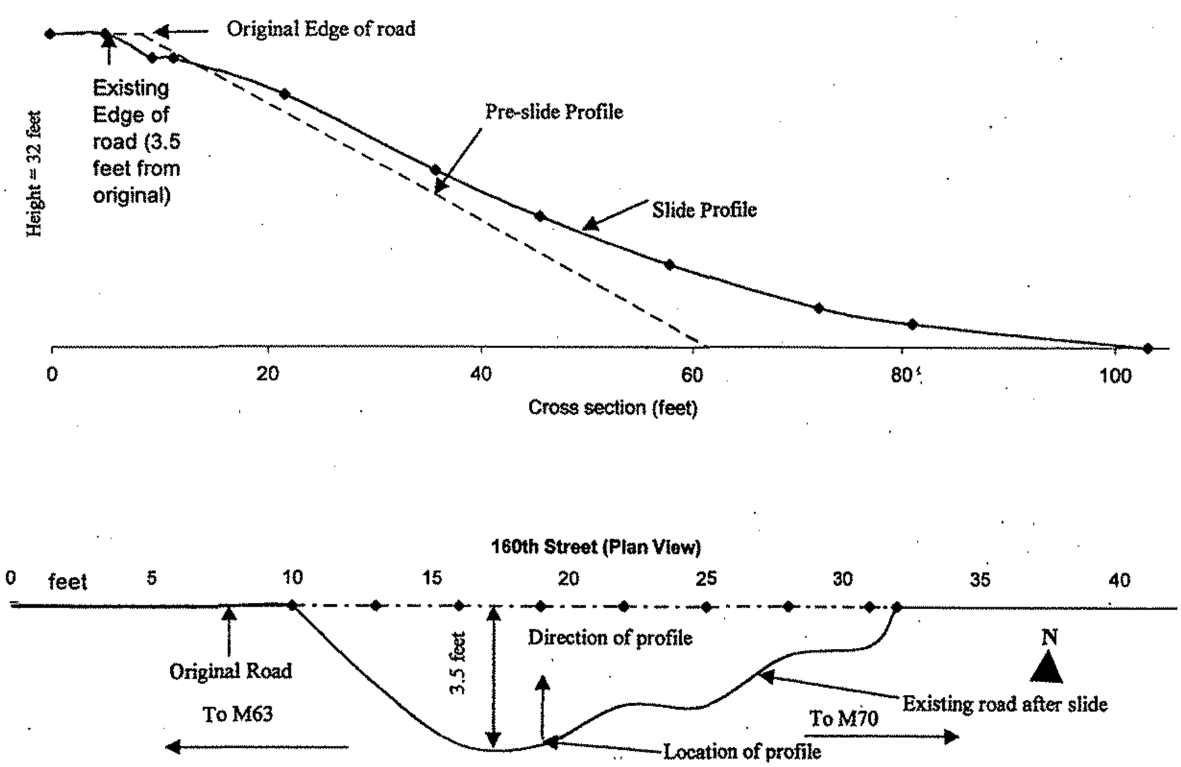


Figure 9.5 Profiles of original slope and post slide failure surface at 160th St. slide.

A corrugated metal pipe about 12 inches (0.3 m) in diameter has its outlet at the toe of the slope. This pipe was observed to be dry during four site visits in the summers and falls of 1999 and 2000. The location of the inlet of the pipe is not known. According to Mr. Jim Christensen, County Engineer, Page County, the slope had failed previously, and the pipe was installed parallel to the slope to drain the slope. The time of the installation is not known as it was prior to Mr. Christensen's tenure as county engineer, but it can be safely assumed to be at least ten years old. The water in the embankment at the base of the slide and the dry pipe indicated that the drain is not functioning properly.

9.2.4 Soil sampling

Undisturbed Shelby tube samples were obtained for the purpose of determining the unit weights and shear strength parameters of the soil as well as engineering index properties. A subsurface log of the boring is presented in Karnik (2001). The embankment is constructed of a mixture of silty clay and glacial clay. During the boring, which extended to a depth of over 20 feet (6 m), no water was observed in the borehole.

9.2.5 Geotechnical properties

Engineering Index Properties

Atterberg limit tests and mechanical analyses were run on three soil samples from each Shelby tube. The mean values of liquid limit and plastic limit are 39.3% and 22.7%, respectively. All the samples had greater than 80% of the soil particles passing the 0.075mm (No. 200). The soil classified as CL by the Unified Classification System. According to AASHTO, the soil classified as A-6 to A-7-6. The raw data and the particle size distribution curves can be seen in Karnik (2001).

Compaction test data:

Standard Proctor tests were conducted on composite soil samples to determine the dry unit weight versus moisture content relationship, so that the density and moisture characteristics of the soil in the embankment could be compared with specifications. Standard Proctor Test results are in Figure 9.6. The optimum moisture content was about 21% and the maximum dry unit weight was 100pcf (15.75kN/m^3). The moisture contents and dry unit weights determined from Shelby tube samples are also plotted with the Standard Proctor results in Figure 9.6.

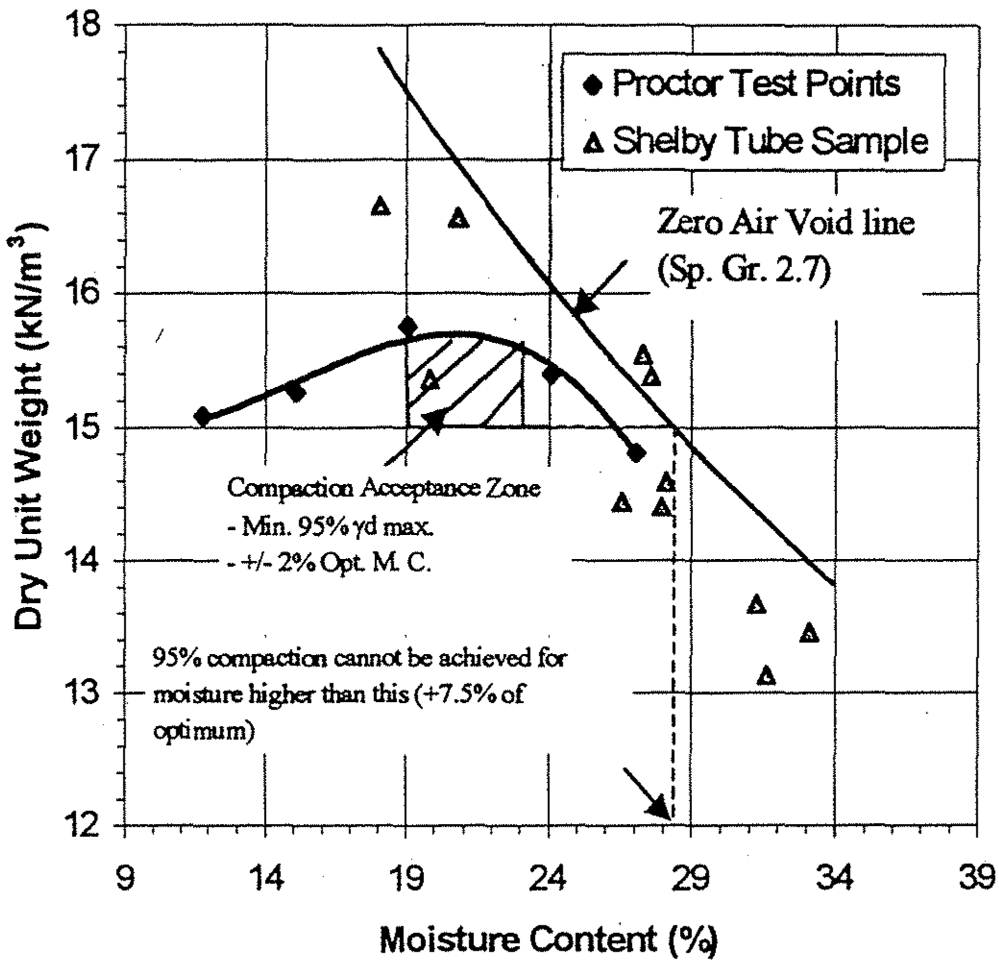


Figure 9.6 Compaction curve and field densities of embankment soils at 160th St. slide.

A "compaction acceptance zone" of 95% maximum unit weight was marked on the Standard Proctor curve to demarcate the acceptable range of dry unit weights and corresponding moisture contents (Bergeson et al, 1998). Figure 9.6 shows that nearly 50% of the samples lie below the zone of 95% compaction. It is likely that the high moisture contents obtained are greater than those at the time of compaction.

Effective stress shear strength

Consolidated undrained (CU) triaxial tests with pore pressure measurements were carried out on backpressure saturated Shelby tube samples. The results were used to plot the deviator stress-axial strain curves on an effective stress basis. The test data, stress-strain curves and the stress paths can be seen in Karnik (2001). Four CU triaxial tests were performed. Because the failure is shallow, two strengths measured at low normal stress values corresponding to the normal stresses along the failure surface were selected to determine the shear strength parameters. If all four data points which had a large scatter were used, the cohesion would have been about 626 psf (30 kN/m^2) and the friction angle less than 10°. These pooled results appeared unreasonable for effective stress parameters and give further rationale for using the two lower normal stress test results to generate "c" and "ϕ" values. For the two low normal stress tests, a "c" value of 19psf (0.91kN/m^2) and a "ϕ" of 35 degrees were obtained. These values were used for the effective stress stability analysis.

9.2.6 Analysis

The slope was analyzed by XSTABL software using the Simplified Bishop Method which generates a circular failure surface. A shallow failure surface based on field observations was used in the analyses. The analyses were conducted for a moist slope and a slope with the elevation of the water table at mid-height of pre-slide slope. Total unit weight was used for the analysis of the moist slope, whereas saturated unit weight was used for the analysis with water table at mid-height. The minimum factors of safety from the XSTABL analysis are 1.3 and 1.1 for the natural moisture and water table at mid-height condition, respectively. The entire results of the analysis are in Karnik (2001). Figure 9.7 shows the analysis result for water table at mid-height.

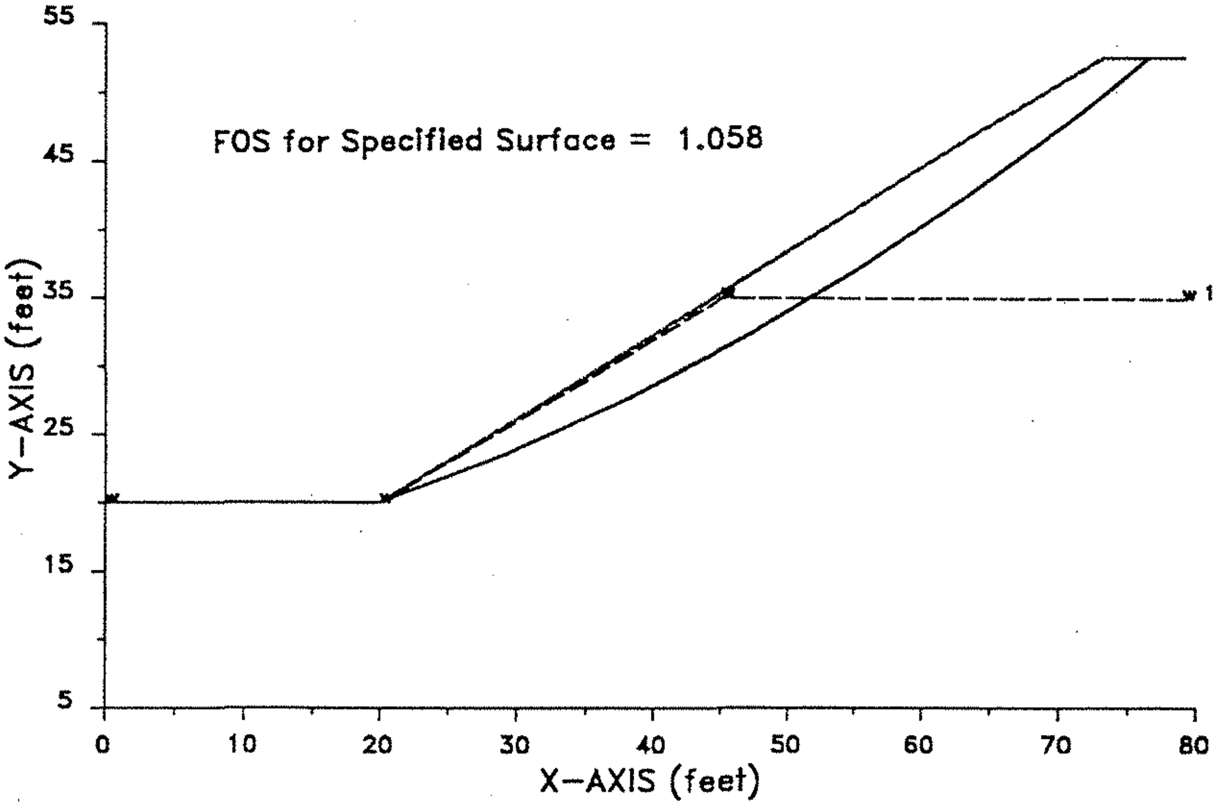


Figure 9.7 XSTABL results for 160th St. assuming watertable at midheight in the embankment

9.2.7 Interpretation and conclusions

The assumption of the water table at the mid-height in the slope is consistent with the field observation; although the factor of safety is slightly above one, it is below the minimum 1.3 specified by Iowa DOT. It is interpreted that groundwater is the major cause of instability in this situation, and it is likely that the failure is associated with accumulation of moisture along the loess-glacial till contact.

Based on the discussion above, improving drainage of the slope should restore stability. The previous attempt at removing the water from the slope was ineffective so installation of wick drains (Santi and Elifrits, 2000) might be an economical and effective remediation.

Because the strength data and subsurface information are limited at this site, further borings and tests may be warranted to determine the source and the path of the water through the foreslope.

9.3 Backslope Along K Street, Page County

9.3.1 Introduction

An investigation was conducted of a natural slope failure located on K Street about four miles (6.5 km) north of Yorktown and one mile (1.6 km) east of County highway M56 in Page County. The failure occurred in a backslope along the shoulder of the road. The failure surface extends laterally nearly 2 feet (0.6 m) into the road, thereby decreasing the available lane width. The objective of this analysis is to determine the probable causes of failure of this slope.

9.3.2 Geologic Setting

Loess mantled glacial till is the regional geology of this site similar to the 160th Street site. The K Street slope is situated in the Mayberry soil series, which consists predominantly of silty clay loam having a depth of about five feet (1.5 m). The area has a gentle to medium slope and is poorly drained. The parent material is weathered glacial till. The soil is subject to severe shrink-swell behavior, and in general, has a low strength. (Clark & McWilliams, 1978). The above geologic description is consistent with the soil observed during the subsurface investigation. The boring log can be found in Karnik (2001) and shows 14 feet (4.2 m) of glacial till beneath 1 foot (0.3 m) of road metal and overlying weathered limestone.

9.3.3 Description of the slide

The site is a natural slope about 15 feet (5 m) high. The original slope angle varied from about 1:1 at the north end to about 1.5:1 at the south end. The slope is parallel to a small creek that is tributary to the Middle Tarkio River. This creek impinges on the toe of the slope at the north end of the slide. Visual inspection of the site indicated an arcuate failure surface about 110 feet (33.5 m) in length parallel to the road with a scarp of about 4 feet (1.2 m) high. A wet spot was observed at the base of the south end of the slope, measuring approximately 17 feet (5.15 m) wide parallel to the road. The source of the water is not known.

The slide is rotational with the failure plane passing through the toe of the slope. During inspection of the slide in November 1999, it was observed that except for the wet spot, the soil in the failure zone was dry.

9.3.4 Soil sampling

Four relatively undisturbed Shelby tube samples were obtained for the purpose of determining the unit weights and the shear strength parameters of the soil. During boring, no water was observed in the borehole. Subsequent testing of the soil samples in the laboratory showed the soil to have a high degree of saturation, varying from 75% at a depth of 8 feet (2.4 m) to 102 % at a depth of 14 feet (4.3 m).

9.3.5 Geotechnical properties

Engineering Index Properties

Mechanical analyses and Atterberg limit tests were carried out on three Shelby tube samples. The mean values of liquid limit and plastic limit are 36.6% and 20.6%, respectively, for samples from a depth of 10 feet (3 m). Mean values of liquid limit and plastic limit for samples from a depth of 14 feet (4.2 m) are 51.0% and 30.2%, respectively. The soil from a depth of 10 feet (3 m) had greater than 75% of the soil particles passing the 0.075mm (No. 200) sieve, whereas the soil from a depth of 14 feet (4.2 m) had more than 80% of the soil particles passing the 0.075mm (No. 200) sieve. The Unified Classification System classifies the upper soil as CL and the lower soil as MH. According to AASHTO, the soils are classified as A-6 and A-7-5 respectively. The raw data and the particle size distribution curves can be seen in Karnik (2001).

Effective stress shear strength

Three consolidated undrained (CU) triaxial tests with pore pressure measurements were carried out on backpressure saturated Shelby tube samples. The results were used to plot the deviator stress versus axial strain curves. The stress-strain curves of high normal stress tests peaked at strains of greater than 15 % or exhibited no peaks while the low normal stress curve peaked at about 5% strain. Stresses corresponding to axial strains of about 5% were selected as the failure values for all three tests. Using these values, the effective stress K_f line was plotted and the "a" and "α" values were obtained by linear regression as shown in Figure 9.8. The effective stress strength plot shows the soil to have a "c" value of 201psf (9.66 kN/m^2) and a "ϕ" of 20 degrees. These values were used for the stability analysis. The triaxial test stress-strain curves and the stress paths are in Karnik (2001).

9.3.6 Analysis

The slope was analyzed using XSTABL with the Simplified Bishop method of stability analysis that employs a circular failure surface. The analysis was carried out considering three different pre-slide slope profiles: K1 south of the slide, K2 through the center of the slide, and K3 north of the slide. The analyses were conducted assuming a slope at natural moisture content and a slope with a water table elevation at 0.25 times the height of slope.

Profile	Slope (H:V)	Assumed in-situ conditions	Factor of safety
South	1.5 : 1	Soil is moist	1.9
South	1.5 : 1	Water table at 0.25 H	1.7
Center	1.25 : 1	Soil is moist	1.5
Center	1.25 : 1	Water table at 0.25 H	1.4
North	1 : 1	Soil is moist	1.5
North	1 : 1	Water table at 0.25 H	1.4
North	1 : 1	Toe erosion & soil is moist	1.2
North	1 : 1	Toe erosion & water table at 0.25 H	1.2

Table 9.3 Safety factors for different profiles and assumed environmental conditions

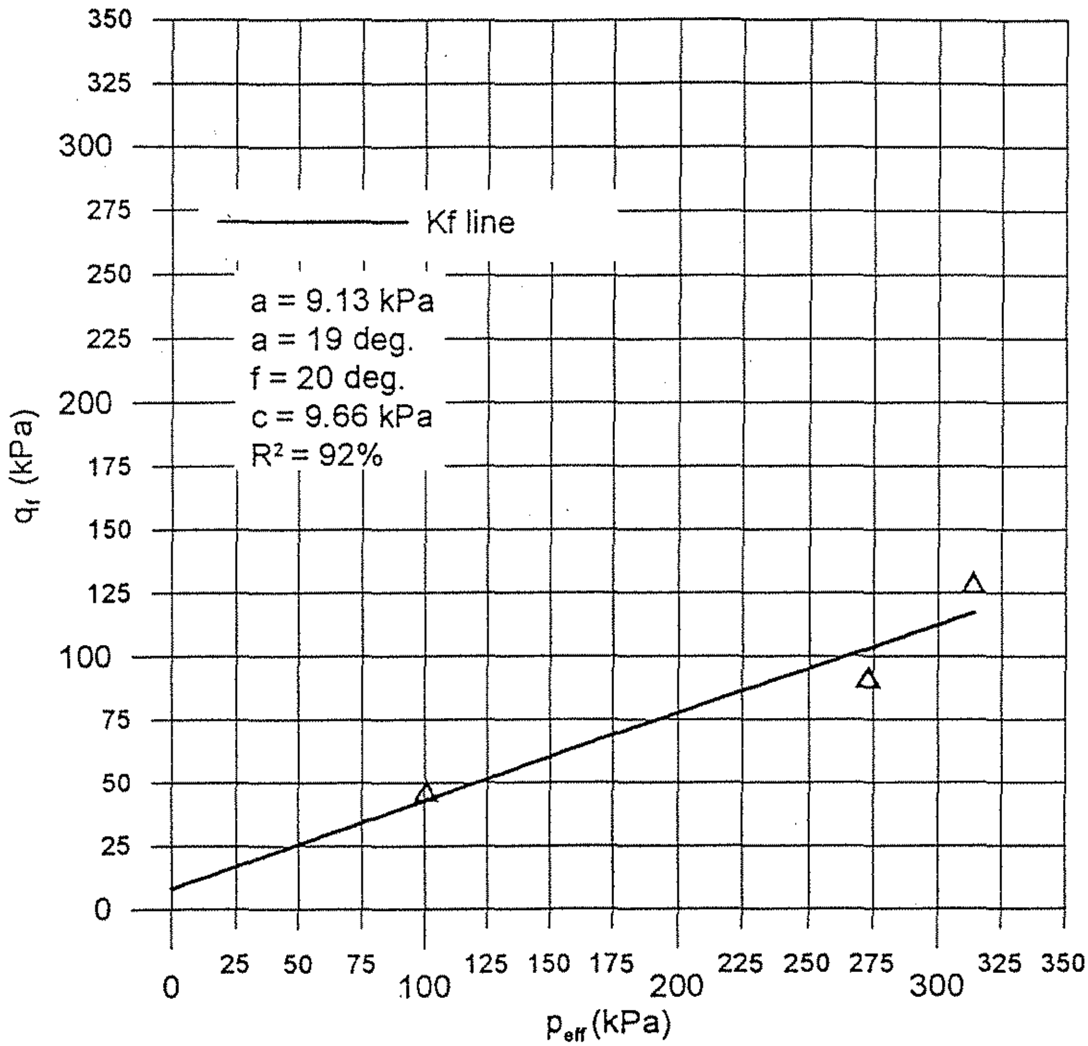


Figure 9.8 Strength (effective p-q) diagram for Shelby tube samples from K Street slide.

Table 9.3 shows the various minimum factors of safety obtained for different profiles. A typical analysis is shown in Figure 9.9.

It can be seen from Table 9.3 that the factor of safety of 1.9 for the south end of the slide is the highest value. Other minimum safety factors range from 1.7 to 1.4 with lower values from the high water table condition at the center and north end of slide. None of these values falls below the DOT recommended minimum. These safety factors suggest that groundwater alone is insufficient to create a failure.

Because the previously determined safety factors are so high, undercutting of the toe of the slope by the stream at the north end of the slide was hypothesized as another possible cause. To assess this hypothesis, the north profile of the slope was analyzed using XSTABL, with the toe eroded laterally by 10 feet (3 m) as shown in Figure 9.10. This seems reasonable based on the field observations. Analyses conducted for both the slope in a natural moisture condition and the slope with a high water table resulted in factors of safety rounded off to 1.2. Clearly, toe erosion has more influence on the stability of this slope than has raising the groundwater level. Although the safety factors are above 1, they are less than the minimum safety factor of 1.3 recommended by Iowa DOT.

As with the slide on 160th Street, the wet spot at the toe of the slope was observed after a period of protracted drought conditions, and the source of this moisture could not be determined. The samples from the borings also showed a surprisingly high value of degree of saturation. It is possible that this water played a role in the slope failure as a result of lateral seepage along the contact between limestone and till.

9.3.7 Conclusions

It is interpreted that the landslide occurred as a progressive failure starting at the north end of the slide as a result of a combination of undercutting of toe at the north end and seepage along the glacial till/limestone contact, and then progressed southward. Although the south end of the slide had the highest factors of safety in the two dimensional analysis, the failure of the adjacent soil mass may have overstressed this mass of soil.

If this interpretation is correct, riprap or lateral erosion control at the north end of the slide combined with drains could prevent this slide from recurring.

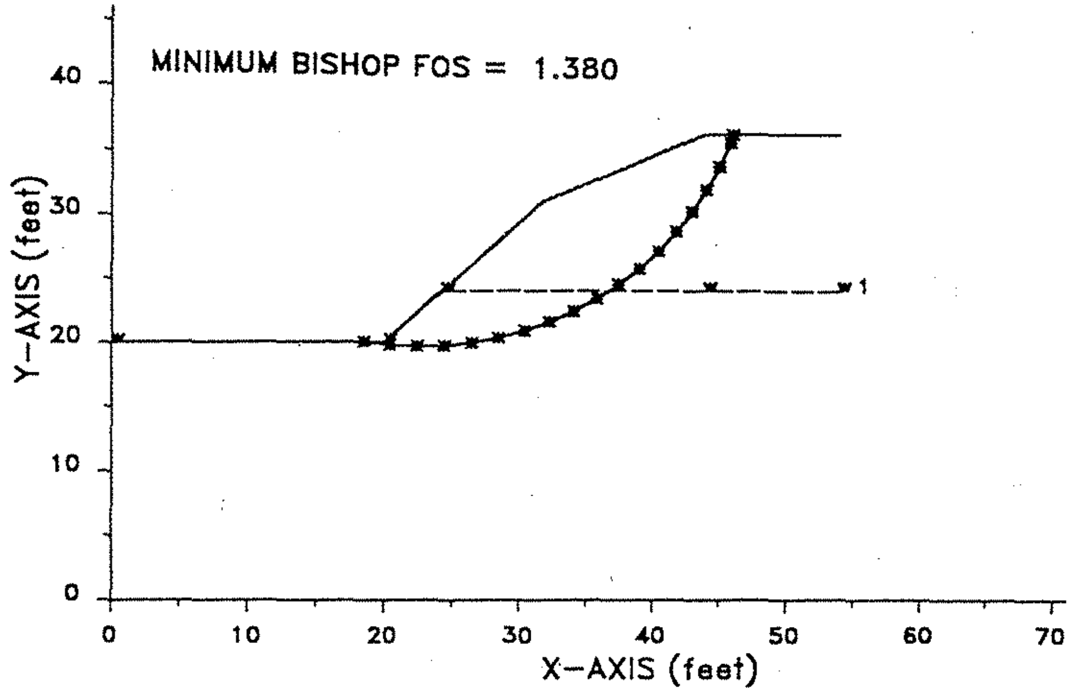


Figure 9.9 XSTABL results for K Street slide with watertable at 25% slope height.

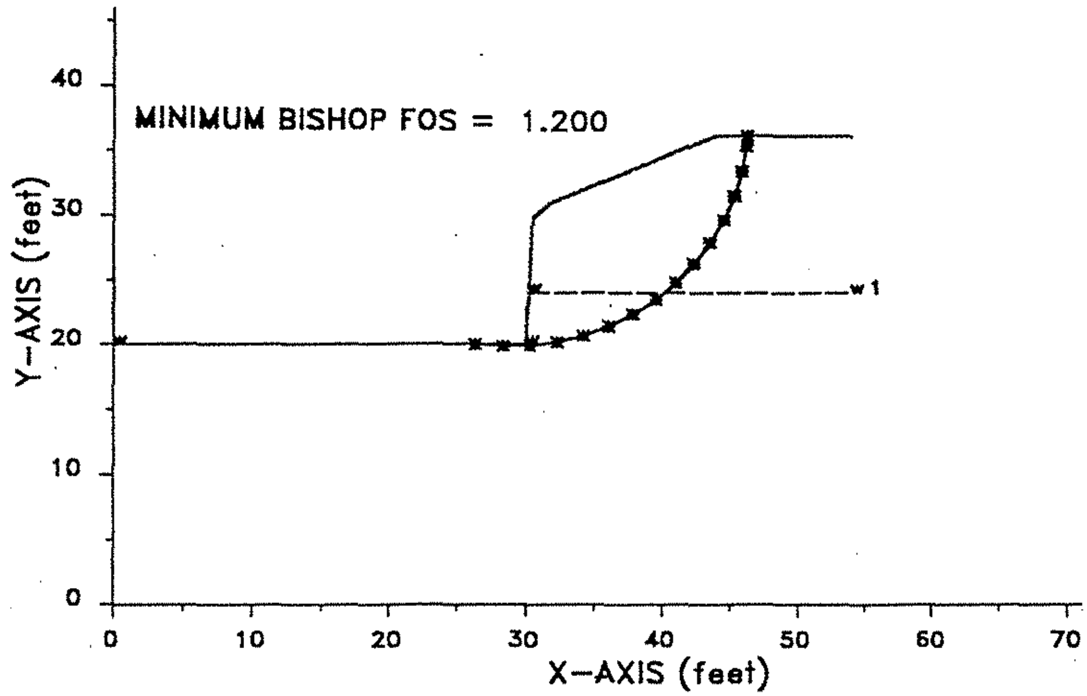


Figure 9.10 XSTABL results for K Street assuming erosion at the toe.

9.4 Backslope at Murray Hill , Harrison County

9.4.1 Introduction

The site for this study is a backslope located approximately 2 miles (3.2 km) northeast of Little Sioux, on the paved county road, F20, in northwest Harrison County. The landslide is near a scenic overlook of the Missouri River known as Murray Hill. The date of the slide is not known but occurred prior to 1998.

9.4.2 Geologic Setting

The site is situated in the steep sloped loess hills where the subsoil profile typically consists of a thick layer of aeolian loess silt underlain by glacial till over bedrock. The upper layer is Peorian age loess, up to 50 feet (15 m) or more in thickness, and is often underlain by Loveland age loess. This material is called friable loess. The soil in the upland drainageways and footslopes often consists of alluvial-colluvial soils that have eroded or migrated from the higher elevated hills (Jury and Fisher, 1976).

The soil series at this location is the Hamburg silt loam with 40° to 75° slope angles. Erosion and gullying are serious hazards. Slump blocks about 1 foot (0.3 m) high, called "catsteps", are common features on this soil series. Hamburg silt loam contains very low organic-matter, and has moderate alkaline and calcareous contents.

The permeability of the soil is moderately rapid. The available water capacity is high, but the runoff is so rapid that the soil seldom absorbs enough moisture to reach capacity (Jury and Fisher, 1976). This suggests that soil saturation in upland positions such as Murray Hill is unlikely.

9.4.3 Description of the Slide

The site is 170 feet (52 m) above the Missouri River floodplain, near the upper part of the upland bluffs. The slopes adjacent to the roadway are vegetated with grasses and trees. During the site investigation carried out by ISU in June 1999, a longitudinal crack was observed along the edge of the existing asphalt pavement, and a minor settlement of 3 inches (8cm) was observed. A horizontal ledge interpreted as an old trail was observed at the bottom of the slope failure, about 25 feet (7.5 m) below the roadway. Slope profiles were measured along the slide and adjacent to it. The original slope profile is 33 feet (10 m) high with angles of 41° in the upper portion and 51° in the lower portion (Figure 9.11). The slope profile in the failed region is shown in Figure 9.12.

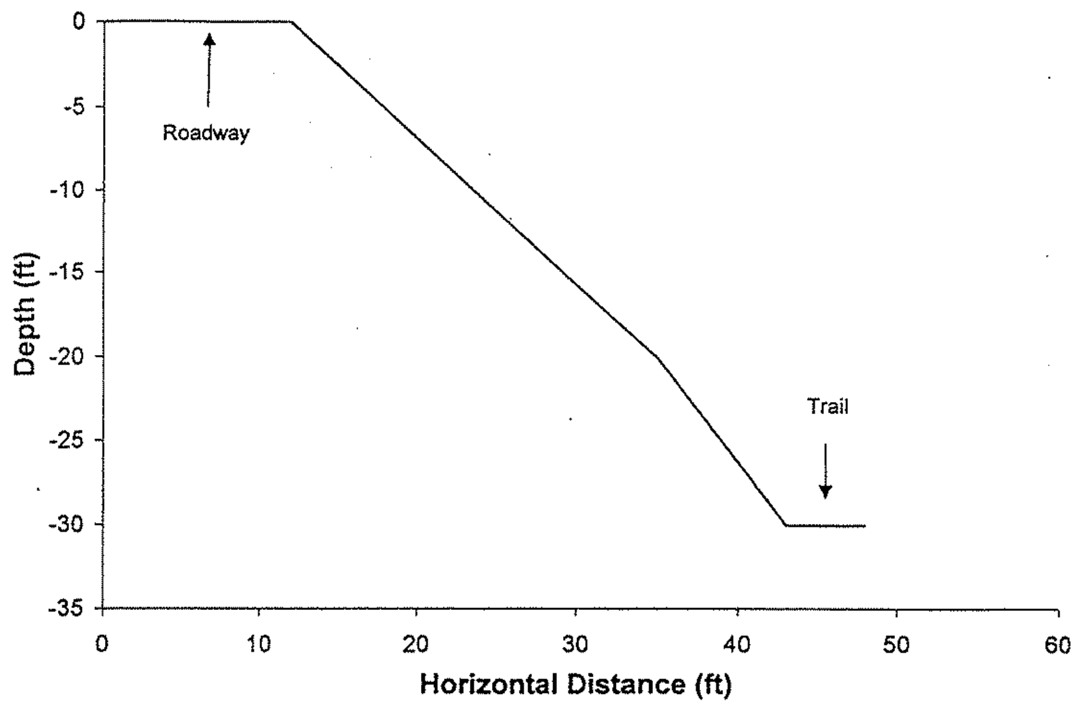
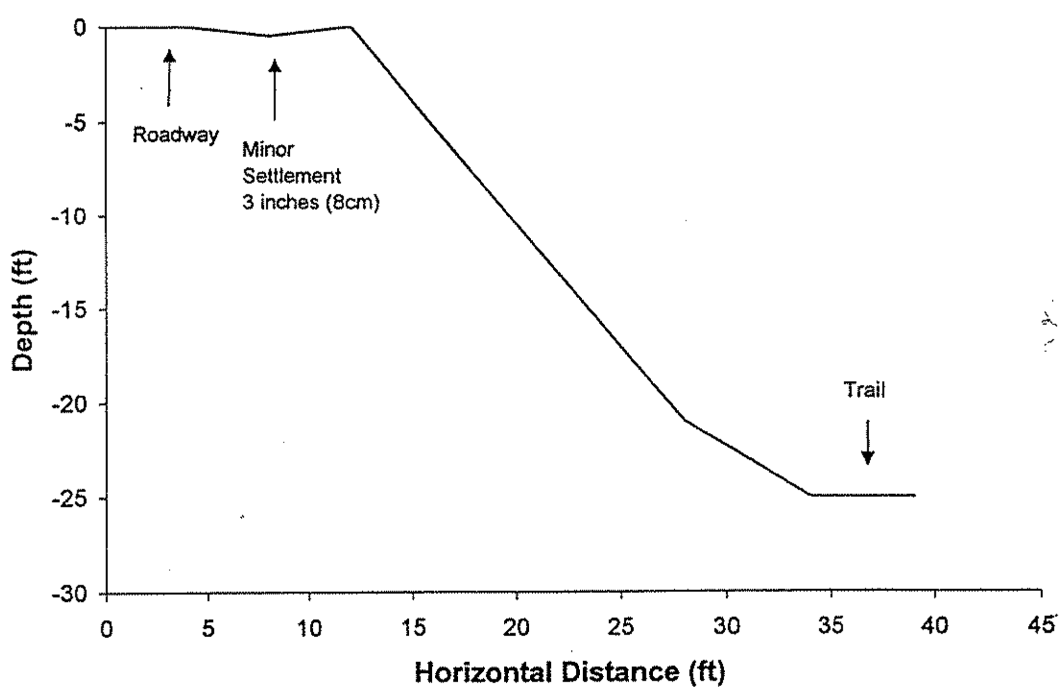


Figure 9.11 Original slope profile at Murray Hill slide



9.12 Post failure slope profile at Murray Hill

9.4.4 Soil sampling

For this study, seven Shelby tube samples were obtained from one boring. The samples were tested to determine the unit weight and effective stress shear strength parameters of the soil. A subsurface log of the boring can be found in Chu (2001). The stratigraphy consists of a layer of gray brown silt to a depth of 26 feet (8 meters) underlain by a 9-foot (2.7 meters) thick layer of brown gray silt. No water was observed in the borehole.

9.4.5 Geotechnical properties

Engineering Index Properties

Atterberg limit tests and mechanical analyses were performed on two Shelby tube samples. The liquid limit for both samples is 32%. The plastic limits are 26% and 25%, and the plasticity indices are 6% and 7%. The soil at a depth of 10 feet (3 m) had 71% of the soil particles passing through the 0.075mm (No. 200) sieve whereas the soil at a depth of 30 feet (9 m) had 53% of the soil particles passing through 0.075mm (No. 200) sieve. The soil is classified as ML by the Unified Classification System, and A-4 under the AASHTO classification System.

Effective stress shear strength

Triaxial tests were performed on the Shelby tube samples to obtain the shear strength parameters of the soil. Consolidated drained (CD) triaxial tests were carried out on samples at natural moisture content. Deviator stress versus axial strain curves were plotted by using the test results. These stress-strain curves and stress paths are shown in Chu (2001). By using the maximum values from the stress-strain curves, the effective stress K_f line was plotted and the "a" and "α" values were obtained by linear regression as shown in Figure 9.13. These values were converted to "c" and "ϕ" values where the cohesion "c" is 98.79 psf (4.73 kPa) and the internal friction angle "ϕ" is 30.1°. These strengths are lower than similar studies of loess at natural moisture contents where cohesion intercepts averaged 259 psf (12.4 kPa) and ϕ averaged 26° and are nearly equal to values for saturated loess (Akiyama, 1964, Olson, 1958). Table 9.4 shows the average shear strength parameters that were reported by Olson (1958), Akiyama (1964), and Benak (1967) for the Hamburg soil along with shear strength parameters obtained as part of this study.

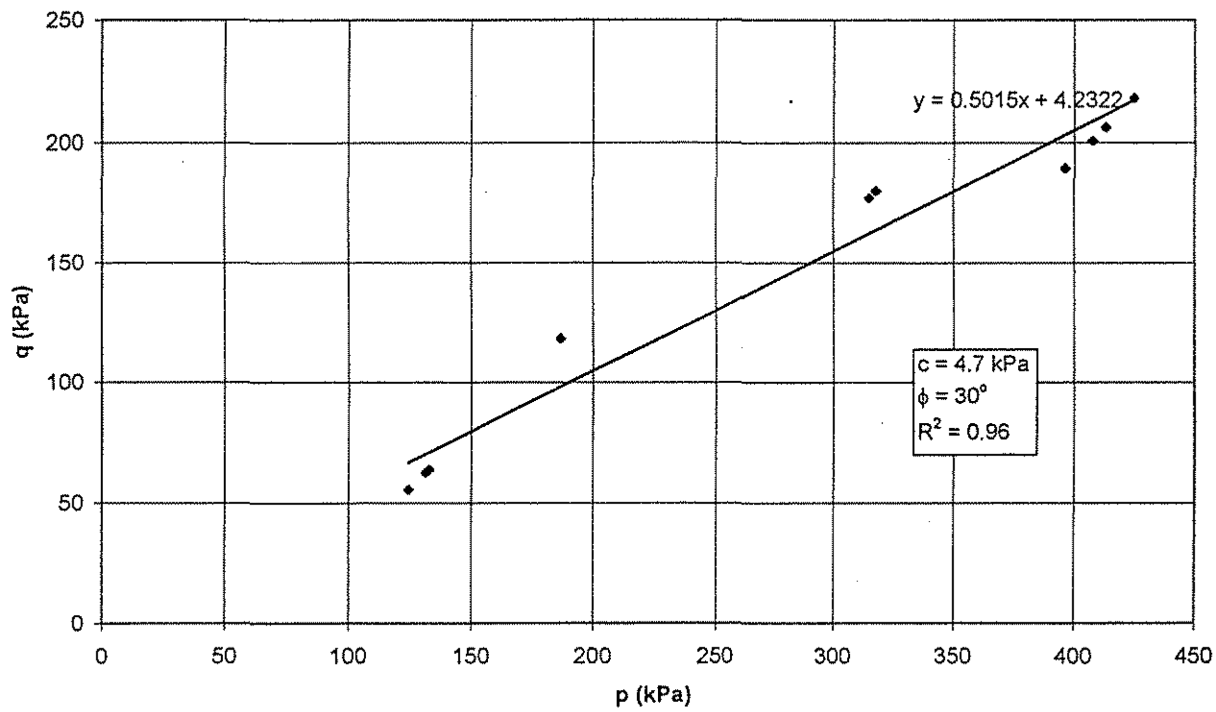


Figure 9.13 Strength diagram for loess at Murray Hill slide

Sources	Saturation %	Cohesion kPa	ϕ degrees	Unit wt. kN/m³	Surcharge KPa	Safety factor
This study	45	4.7	30	15.5	0	1.1
This study	45	4.7	30	15.5	814	0.8
Akiyama, 1964; Benak, 1967	33	12.4	29	14.9	0	1.5
Akyiama, 1964; Benak, 1967	33	12.4	29	14.9	814	0.9
Olson, 1958: Akiyama, 1964	100	5.1	26	15.1	0	1.0

Table 9.4 Strength parameters, unit weights, surcharge stresses, and safety factors from XSTABL

From these data, it is apparent that the cohesion intercept "c" value will be lower if the tests are performed under saturated conditions. These results are similar to conclusions of Badger (1972) who observed that cohesion of friable loess is inversely proportional to the degree of saturation:

9.4.6 Analysis for cause of failure

The slope was analyzed by using XSTABL computer software with the Simplified Bishop method using shear strength values obtained in this study and shear strength values reported in other studies. A typical analysis is shown in Figure 9.14. When the average strengths are used, the minimum safety factor is 1.5 for the soil at natural moisture content. For the shear strength values obtained in this study, the safety factor is 1.1 with a theoretical failure surface close to that observed in the field. Although the latter stability analysis indicates a marginal safety factor at Murray Hill, additional analyses were conducted to determine the influence of other possible factors.

During fieldwork, trucks were observed to pass close to the edge of the road with their tires tracking through the depression in the pavement. Therefore, XSTABL analyses with a surcharge load from the trucks were performed. The vehicle surcharge loading represented a tandem truck; a stress of 17000 psf (814 kPa) was applied on each tire in the location shown in Figure 9.15. The minimum factor of safety decreases to 0.8 when the surcharge is included with the load at the shoulder; the failure surface shown by the analysis, however, extends further back below the roadway than suggested by the cracks observed on the pavement. It is interpreted that the slope without surcharge is at a metastable condition; and instability results from the surcharge load of passing trucks.

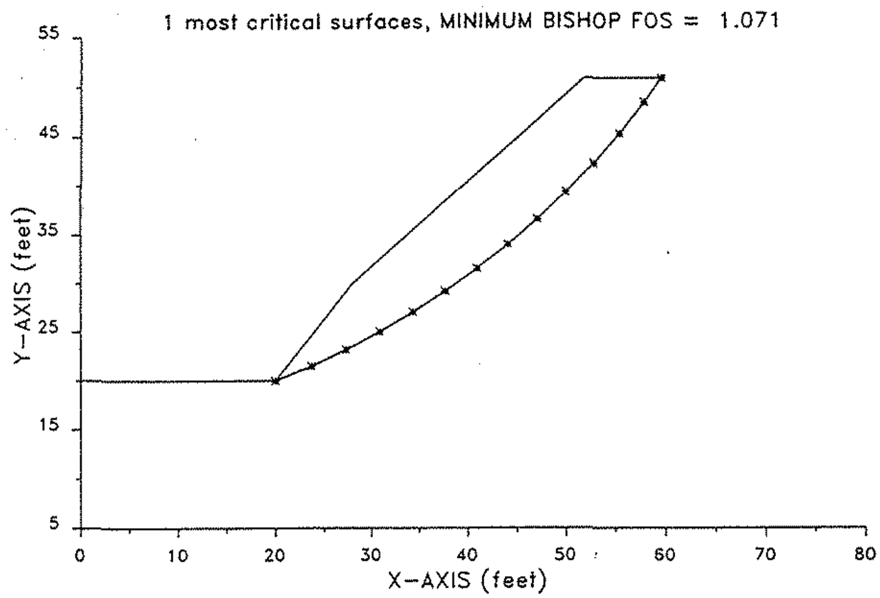


Figure 9.14 XSTABL results for Murray Hill slide

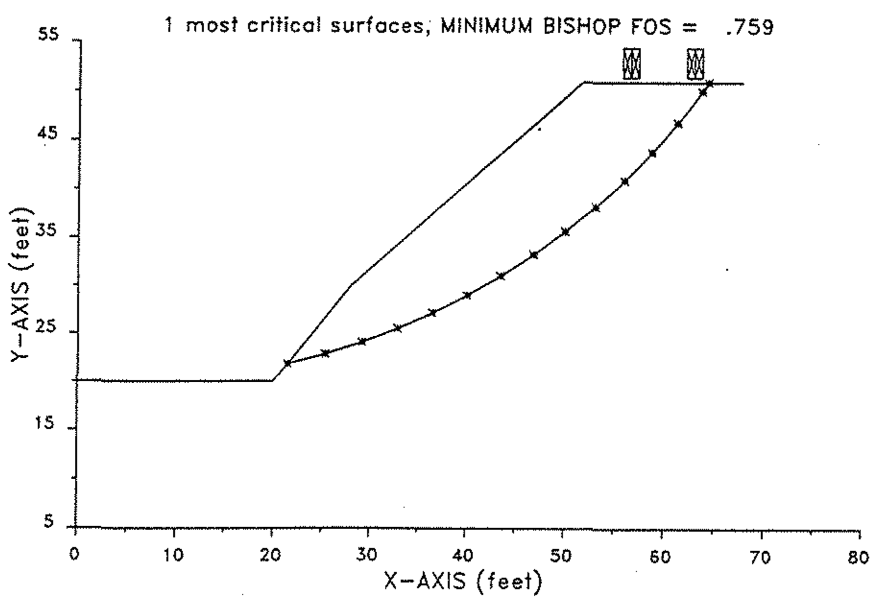


Figure 9.15 XSTABL results for Murray Hill slide with truck surcharge.

9.4.7 Suggested repair method

As truck weight is a likely cause of failure, stability analyses for the surcharge at various locations were performed. Minimum safety factors under vehicle surcharge loading at different distances from the shoulder were conducted and show that if the truck loading is 20 feet (6.1 m) from the shoulder, the factor of safety increases to 1.1. Therefore, moving the road from its present location provides a marginal factor of safety and a partial solution to this slope stability problem. Because of the metastable condition of the natural slope, relocation of the roadway should be combined with a buttress for a more comfortable safety factor. Further analysis of the slope downslope of the proposed buttress should be carried out to insure that the buttress loading will not create stability problems downslope of this slide.

9.4.8 Summary

Stability analyses indicate that the Murray Hill slope is metastable, with a safety factor of 1.1. Additional analyses suggest that a surcharge truck load at the edge of the road reduces the minimum safety factor to 0.8. By shifting the load to about 20 feet (6 m) from the edge of the road, the metastable condition is restored, but an acceptable safety factor requires a buttress.

10.0 CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

A survey of county engineers provided data for a slope stability risk map for the state of Iowa. Areas of high risk are along the western border and southeastern portion of the state. These regions contain deep to moderately deep loess. The central portion of the state is a low risk area where the surficial soils are glacial till or thin loess over till. In this region the landslides appear to occur predominantly in foreslopes or in backslopes along deeply incised major rivers, such as the Des Moines River. The south-central portion of the state is an area of medium risk where failures are associated with steep backslopes and improperly compacted foreslopes.

Of the 60 counties that responded to the survey on landslides, 80% reported landslide activity since 1993 and 31% of the counties had more than 11 landslides during that period. Statewide, failures in foreslopes and backslopes are about equal at 37% and 32%, respectively, while slides along streams are third most important at 26%. Nearly half of the slope failures occur on slopes between 1:1 and 2:1 with both steeper and gentler slopes having fewer slides. Both curvilinear and planar failure surfaces were observed throughout the state. All of the landslides occurred during spring and summer with 50% of the failures caused by ground water. Twenty one percent of the failures are associated with design issues. Most of the slides occurred in slopes between 11 and 20 ft high. The most common and successful repair procedures have employed drainage and slope flattening. Structural support and geosynthetic stabilization are only used occasionally. Chemical stabilization, engineering fabrics, plastic pin piles and wick drains haves not been employed in the state. These repair techniques merit further investigation in Iowa.

Reconnaissance trips to over fifty active and repaired landslides in Iowa suggest that slides in Iowa are frequently either translational or rotational with failure surfaces generally less than 6 feet (2 m) deep. All of the slides observed appear to have the failure surface passing through the toe of the slope.

A database of soil shear strength parameters, classified according to geologic parent material and triaxial sample drainage conditions, was developed with data from the Iowa DOT and consulting engineers' files. Mean values of shear strength parameters and unit weights were computed for glacial till, friable loess, plastic loess and local alluvium. Statistical tests demonstrate that friction angles and unit weights differ significantly but in some cases effective stress cohesion intercept and undrained shear strength data do not. Moreover, effective stress cohesion intercept and undrained shear strength data show a high degree of variability. Some of the variability may be attributed to the variety of data sources and small differences in testing technique. An expanded database on soil shear strength with data generated from uniform testing methods would provide a valuable resource for slope stability assessment and other applications such as foundation design.

Two foreslope and two backslope failure case histories provide additional insights into slope stability problems and repair in Iowa. These include the observation that embankment soils compacted to less than 95% relative density show a marked strength decrease from soils at or above that density. Foreslopes constructed of soils derived from shale exhibit loss of strength as a result of weathering. Moreover, embankment soils showed high degrees of water saturation, even after periods of prolonged dryness. In some situations multiple causes of instability can be discerned from back analyses with the slope stability program XSTABL. In areas where the stratigraphy consists of loess over till or till over bedrock, the geologic contacts act as surfaces of groundwater accumulation that contributes to slope instability.

10.2 Recommendations

10.2.1 Alternate slope repair methods

To date, neither counties nor the state have used plastic pin piles or chemical treatment to repair landslides and geosynthetics have been used sparingly. Recent literature (Loehr et al., 2000 and Santi and Elifrits 2000) suggest that these techniques provide economical alternatives to the more traditional methods of slope remediation. It is suggested that these techniques be investigated with full scale demonstration research projects

10.2.2 Weathering of shale and shale derived soils

This study has indicated that shale and shale-derived soils lose strength when exposed to weathering. Research focusing on fundamental physical and physico-chemical causes of strength loss resulting from weathering could lead to more effective prevention and remediation of slope failures in shale and/or shale-derived soils.

10.2.3 Systematic and controlled database on soil strength.

This research suggests that a database on soil shear strength classified according to geologic parent material is possible. Such a database will facilitate planning and, in some cases, design of stable slopes and other geotechnical applications that require soil strength. A systematic study conducted by one organization with a consistent protocol for soil classification and testing is recommended.

10.2.4 Shallow slope failure remediation

It was observed in this study that the majority of slope failures in Iowa have failure surfaces less than 3 feet (1 m) deep. A project to focus on causes and repair of this type of landslide should result in more economical approaches and is suggested.

10.2.5 Embankment soil moisture

This and previous research (Bergeson et al, 1998) have noted that soil moisture contents of the soils within compacted embankments are at or near saturation. The cause of this condition is not clear. The previous study interpreted this condition to result from overcompaction of the soil during construction. It is speculated here that the cause is uptake of moisture after construction. This study clearly illustrates that these high moisture contents contribute to foreslope instability. If the causes of these high moisture contents can be identified, then remediation of landslides on embankments can be improved. Research on moisture and groundwater/pore pressure conditions in compacted fills is recommended.

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