2950 Niles Road, St. Joseph, MI 49085-9659, USA
269.429.0300 fax 269.429.3852 hq@asabe.org www.asabe.org

An ASABE Meeting Presentation

Paper Number: 1008704

Slope Stability Analysis Tools for Stream Bank Stabilization

Benjamin C. Doerge, P.E., G.E.

USDA-Natural Resources Conservation Service
501 W. Felix St., Bldg. 23, Fort Worth, TX 76115
ben.doerge@ftw.usda.gov

Written for presentation at the
2010 ASABE Annual International Meeting
Sponsored by ASABE
David L. Lawrence Convention Center
Pittsburgh, Pennsylvania
June 20 – June 23, 2010

Abstract. Slope stability is a key concern in stream bank stabilization projects. However, stream bank stability analyses are often limited to the application of "rule-of-thumb" guidelines, such as: 1) "Replicate the stable slopes from upstream and/or downstream of the treatment area;" or 2) "A 2(H):1(V) slope is satisfactory for slopes less than about 6 meters (20 feet) in height." Such guidelines, while useful in many cases, may overlook critical factors affecting slope stability in both the conservative and non-conservative directions, including spatial variation of bank materials, seepage conditions, and the stabilizing effect of riprap or other revetments. For banks with negligible cohesion, simple infinite slope equations can be applied. But this method of analysis can be overly conservative if the slope is lined with riprap and the effect of the riprap is neglected. On the other hand, if the infinite slope analysis is applied simply to the riprap lining, the result is non-conservative. Stream bank stability should be analyzed for all seepage conditions that may reasonably be expected to occur over the life of the project. The most critical condition is typically the rapid drawdown case, which occurs during the recession portion of the flood hydrograph.

This paper presents slope stability analysis tools which are of particular relevance to stream bank stabilization applications, including infinite slope equations, quantifying the stabilizing effects of partial and complete bank riprap, and consideration of seepage effects in stream bank stability analysis. A parametric analysis is presented showing the relative effect of soil permeability on seepage conditions and slope stability during rapid drawdown.

Keywords. Slope stability, stream bank stabilization, infinite slope analysis, seepage, rock riprap, rapid drawdown.

The authors are solely responsible for the content of this technical presentation. The technical presentation does not necessarily reflect the official position of the American Society of Agricultural and Biological Engineers (ASABE), and its printing and distribution does not constitute an endorsement of views which may be expressed. Technical presentations are not subject to the formal peer review process by ASABE editorial committees; therefore, they are not to be presented as refereed publications. Citation of this work should state that it is from an ASABE meeting paper. EXAMPLE: Author's Last Name, Initials. 2010. Title of Presentation. ASABE Paper No. 10----. St. Joseph, Mich.: ASABE. For information about securing permission to reprint or reproduce a technical presentation, please contact ASABE at rutter@asabe.org or 269-429-0300 (2950 Niles Road, St. Joseph, MI 49085-9659 USA).

Introduction

Stream bank stability consists of two separate components: 1) hydraulic stability (i.e., stability against erosion); and 2) geotechnical stability (i.e., stability against slope failure). The two components are analyzed independently, although they are integrally related. For example, toe erosion or undercutting (from hydraulic instability) can directly lead to slope failure. At the same time, measures employed to enhance hydraulic stability, such as riprap or other revetments, can improve geotechnical stability.

Since stream bank stabilization often involves slopes of relatively low height, detailed geotechnical analyses are frequently not performed. Instead, rules of thumb may be applied, such as: 1) "Replicate the stable slopes from upstream and/or downstream of the treatment area;" or 2) "A 2(H):1(V) slope is satisfactory for slopes less than about 6 meters (20 feet) in height." Such guidelines, while useful in many cases, may overlook critical factors affecting slope stability in both the conservative and non-conservative directions, including spatial variation of bank materials, critical seepage conditions, and the stabilizing effect of riprap revetments or other bank stabilization measures. Furthermore, such simplified approaches give no information on how close the bank is to failure.

The purpose of this paper is to provide easy-to-apply tools which can be used to understand and analyze geotechnical stability of stream banks. Use of these tools, however, does not eliminate the need for site-specific geologic investigations to characterize bank materials, stratigraphy, and seepage conditions. In fact, such investigations provide the inputs needed to perform these very analyses.

Infinite Slope Analysis

The simplest form of slope stability analysis involves the application of infinite slope equations. This method of analysis applies to relatively long, uniform slopes with certain well defined seepage patterns. For long slopes, end effects such as toe buttressing can be reasonably neglected, and the results are independent of overall slope length. An infinite slope analysis considers a free-body diagram of a typical slice of soil on the slope and formulates a factor of safety as the ratio of all forces resisting downward sliding of the slice (shear strength) to those driving such movement (weight and pore pressures). Figure 1 shows an example of such a free-body diagram for a homogeneous slope of cohesionless soil with no seepage.

Failure in infinite slope situations involves shallow sloughing or sliding of surface materials in the down-slope direction. This failure mechanism applies to all cohesionless materials, such as sand, gravel, riprap, and non-plastic silts. Infinite slope analysis is generally not applicable to slopes of cohesive soils, since such slopes typically fail along a deeper, circular failure surface, rather than by shallow sloughing. However, infinite slope analysis is applicable to cohesive soils in cases where a layer of altered surface material overlies a base of tighter, stronger soil or rock. This condition is typically the result of weathering by such processes as shrink-swell, freeze-thaw, or chemical decomposition of bedrock. Such layering can force the cohesive surface material to fail by shallow sloughing parallel to the slope, rather than by a deeper rotational failure. Failure in such cases is typically triggered by saturation of the more permeable surface material.

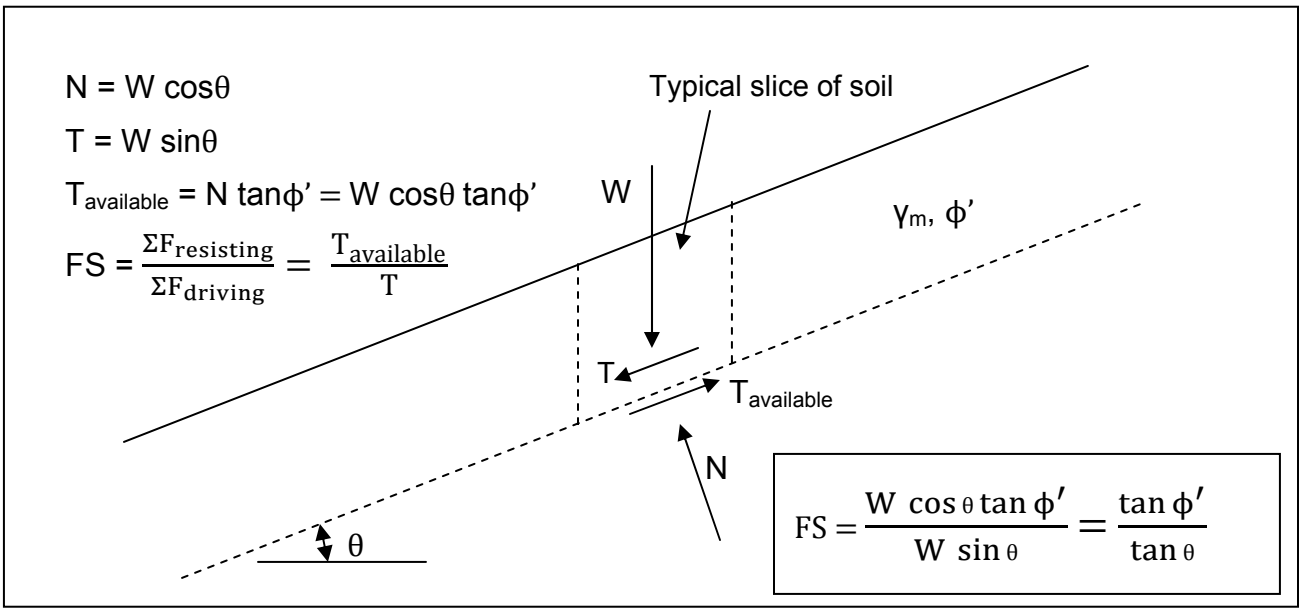

Figure 1. Free-Body Diagram of Infinite Slope of Moist, Cohesionless Soil

Applicable Seepage Conditions

Infinite slope analyses are typically performed for three distinct seepage conditions: 1) no flow; 2) seepage parallel to slope; and 3) horizontal seepage. The no flow condition exists in slopes of moist (i.e., partially saturated) soil or in submerged slopes (see Figure 2). Flow parallel to slope can occur near the base of a slope where the phreatic surface from ground water in the bank coincides with the slope surface (see Figure 3). It can also develop when the slope becomes saturated due to sustained rainfall or snow melt (see Figure 3). Horizontal flow approximates the flow condition that exists immediately following rapid drawdown of the water surface in a stream channel when the saturated bank drains toward the suddenly exposed slope. Horizontal flow can also develop when seepage from a confined layer enters a stream

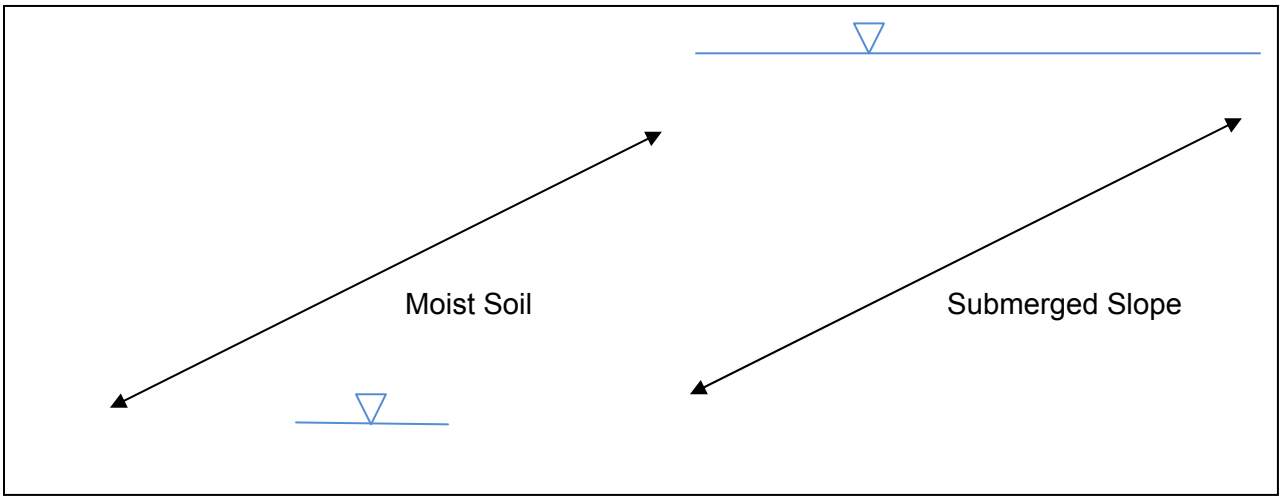

Figure 2. No Flow Seepage Condition

channel or when heavy rainfall infiltrates into a highly permeable soil formation which overlies a horizontal impermeable layer, such as a sand dune over bedrock (see Figure 4).

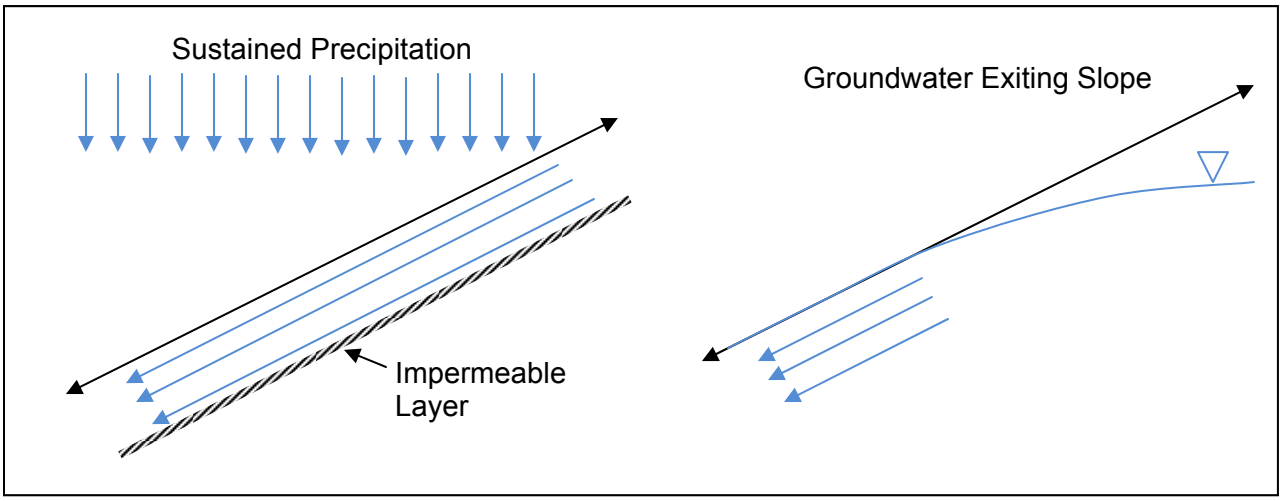

Figure 3. Seepage Parallel to Slope Condition

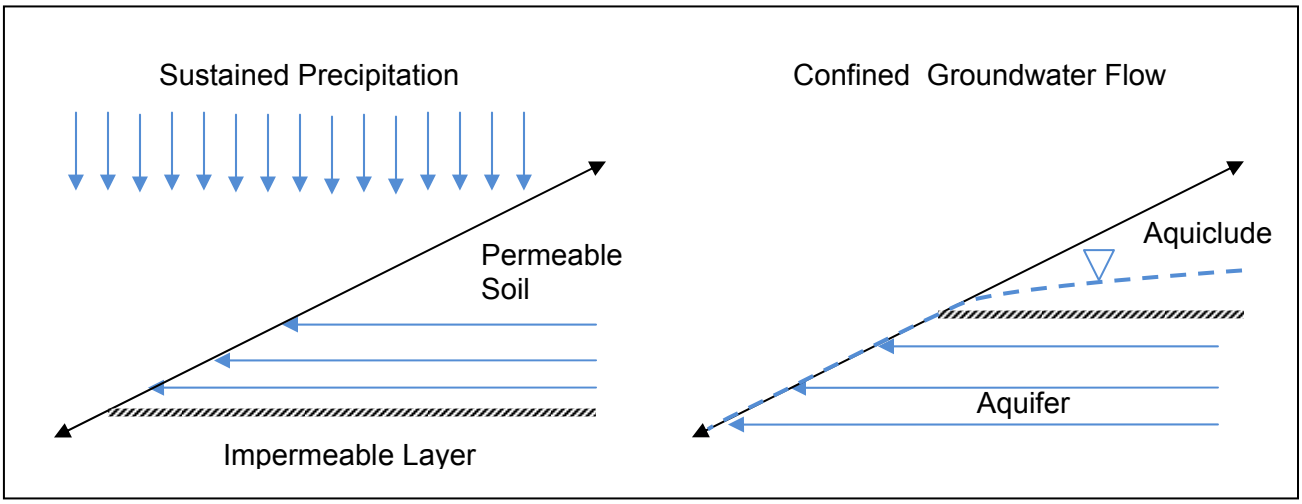

Figure 4. Horizontal Seepage Condition

Infinite Slope Equations for Cohesionless Soils

The general equations for infinite slopes in cohesionless soils are presented below for the three basic seepage conditions (U.S. Army Corps of Engineers, 1970).

No Flow:

$$FS = \frac{\tan \phi'}{\tan \theta} \qquad (\text{Eq. } 1)$$

Seepage Parallel to Slope:

$$FS = \frac{\gamma'}{\gamma_{sat}} \frac{\tan \phi'}{\tan \theta} \qquad \text{(Eq. 2)}$$

Horizontal Seepage:

$$FS = \frac{\gamma' - \gamma_w \tan^2 \theta}{\gamma_{sat}} \frac{\tan \phi'}{\tan \theta} \qquad \text{(Eq. 3)}$$

where:

FS = factor of safety against sliding

γ' = buoyant or submerged unit weight of soil

γ_{sat} = saturated unit weight of soil

γ_w = unit weight of water

θ = angle of inclination of slope with horizontal

ϕ' = effective angle of internal friction of soil

Comparison of the Equations 1-3 shows that the no-flow condition yields the highest safety factor of the three for a given soil and slope angle. For the same soil and slope under the seepage-parallel-to-slope condition, the safety factor drops to approximately half of that for the no-flow condition. The reason for this result is that for normal weight cohesionless soils, the buoyant unit weight is approximately one half of the saturated unit weight. For horizontal seepage, the safety factor is approximately 90 to 95 percent of that for the parallel seepage case. The significant reduction in stability between the no-flow condition and the other two seepage conditions explains why stream banks and other natural slopes can remain stable for many years and then suddenly experience large-scale failure when extreme events of stream flow or precipitation cause a much more severe seepage condition to develop higher up in the slope.

Infinite Slope Equations for Cohesive Soils

Under certain conditions, infinite slope analysis is applicable to cohesive soils. For example, in high plasticity clays with a high potential for shrink-swell weathering, a surface zone can develop which has a blocky, fissured structure. This surface material is of higher permeability than the underlying, less weathered clay, due to the presence of the numerous cracks and fissures within the soil mass. The shrink-swell weathering also reduces the shear strength of the surface material to the “fully softened” shear strength (Terzaghi, et al., 1996). Upon saturation by precipitation and/or inundation, seepage parallel to the slope can develop in the weathered zone. The resulting increase in pore pressure may trigger a slide along the interface between the weathered and unweathered materials.

The infinite slope equation for cohesive soil for seepage parallel to slope is given by Equation 4 (Lambe and Whitman, 1969):

Seepage Parallel to Slope:

$$FS = \frac{c' + \gamma' z \cos^2 \theta \tan \phi'}{\gamma_{sat} z \sin \theta \cos \theta} \qquad \text{(Eq. 4)}$$

where:

FS = safety factor against sliding

γ' = buoyant or submerged unit weight of surface soil

γ_{sat} = saturated unit weight of surface soil

γ_{w} = unit weight of water

θ = angle of inclination of slope with horizontal

ϕ' = effective angle of internal friction of surface soil

c' = effective cohesion of surface soil

z = vertical depth of surface soil

Equation 4 can be modified as shown in Equations 5 and 6 for the moist and submerged slope cases.

Moist Slope:

$$\mathrm{FS} = \frac{c' + \gamma_{\mathrm{m}}\, z\, \cos^2\theta\, \tan\phi'}{\gamma_{\mathrm{m}}\, z\, \sin\theta \cos\theta} \qquad \text{(Eq. 5)}$$

where:

γ_m = moist unit weight of soil

Submerged Slope:

$$\mathrm{FS} = \frac{c' + \gamma'\, z\, \cos^2\theta\, \tan\phi'}{\gamma'\, z\, \sin\theta \cos\theta} \qquad \text{(Eq. 6)}$$

It can be seen from Equation 4 that safety factor is a function of depth of the surface zone, z. This depth must reach a critical value before failure can occur, even with fully developed seepage parallel to the slope. This explains why slopes of such soils can remain stable for many years, until weathering has penetrated to the critical depth and saturation occurs.

Estimating Soil Shear Strength Parameters

Infinite slope analysis requires the input of the appropriate shear strength parameters for the soil, specifically, the angle of internal friction (ϕ or ϕ') and the cohesion (c or c'). For cohesive soil (clays and plastic silts), it is difficult to accurately estimate its shear strength parameters based on just soil classification and other index properties. Therefore, shear tests are required if accurate shear strength parameters are needed. However, for cohesionless soils (clean sands and gravels, non-plastic silts, or sands and gravels with non-plastic silt fines), the angle of internal friction may be estimated with reasonable accuracy from generalized tables and charts. Sources for such charts and tables include: U. S. Army Corps of Engineers (1992), Naval Facilities Engineering Command (1982), Terzaghi et al. (1967), and Fang (1991).

Stability Improvements Due to Riprap

The stability of a slope is improved by placing a layer of free-draining riprap on the slope surface. However, quantifying the improvement is not possible with simple infinite slope equations. Mackey, et al. (2003) adapted infinite slope analysis to multiple-layer systems with varying depths of saturation and non-zero cohesion. However, this analysis is applicable only for systems with geometry that forces the failure surface to be parallel with the slope surface,

such as in landfill cover systems. This is not true of either cohesionless or cohesive slopes with riprap blankets.

With Cohesionless Soils

Computer analysis using the SLOPE/W program reveals that the shape and location of the failure surface for cohesionless slopes changes in response to the addition of the riprap blanket. The failure surface with the lowest safety factor is a shallow circle exiting just beyond the toe of the slope (see Figure 5), instead of a shallow sloughing failure parallel to the slope.

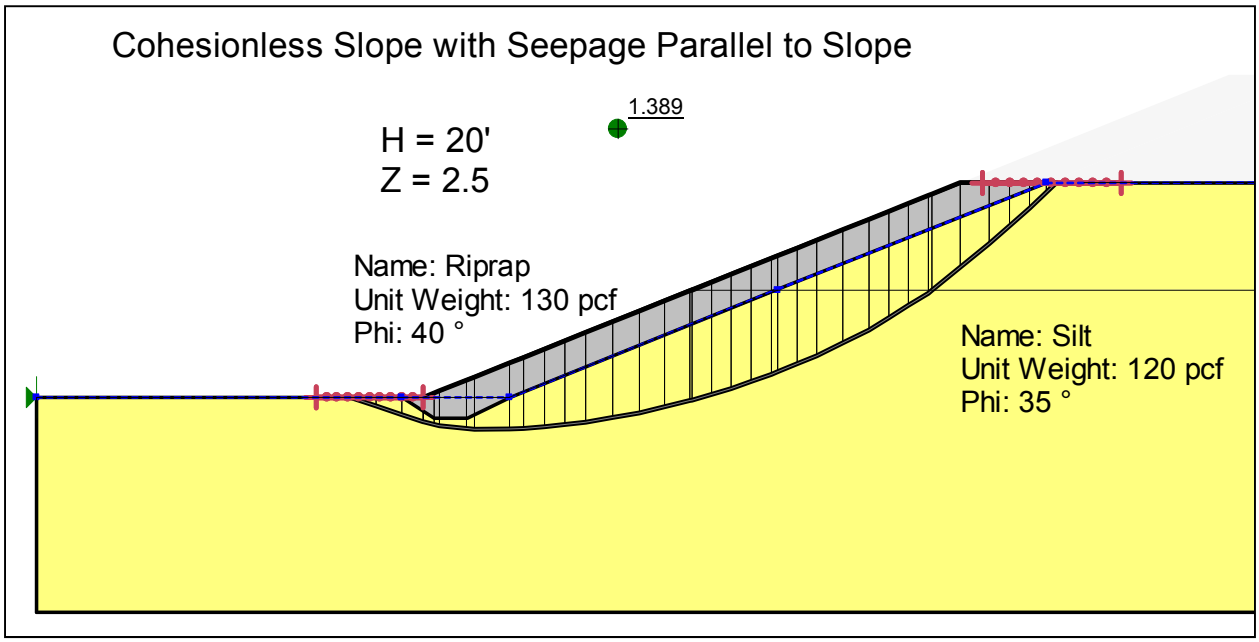

Figure 5. Typical Failure Surface for Cohesionless Slope With Riprap Blanket

The results of multiple computer runs over a range of slope heights and angles, effective friction angles, and riprap thicknesses are summarized in Table 1. The angles of internal friction (ϕ') shown in Table 1 (25^o, 30^o, and 35^o) correspond to loose silty sand (SM), loose clean sand (SP), and medium dense clean sand (SP, SW), respectively.

Table 1. Safety Factor (FS) vs. Slope Height (H), Side Slope, Internal Friction Angle (ϕ'), and Riprap Thickness for Cohesioneless Slopes With Seepage Parallel to Slope

Riprap Thickness = 0 (no riprap)

	FS								
	Side Slope (H):(V)								
	3:1			2.5:1			2:1		
ϕ', deg =	25	30	35	25	30	35	25	30	35
H									
Any ht.	0.67	0.83	1.01	0.56	0.69	0.84	0.45	0.55	0.67

Riprap Thickness = 0.61 m (2 ft)

	FS								
	Side Slope (H):(V)								
	3:1			2.5:1			2:1		
ϕ', deg =	25	30	35	25	30	35	25	30	35
H									
m (ft)									
3.05 (10)	1.18	1.46	1.77	1.04	1.29	1.56	0.90	1.12	1.33
6.10 (20)	0.98	1.21	1.47	---	1.04	1.26	0.70	0.87	1.04
9.14 (30)	---	1.10	1.33	---	0.93	1.14	---	---	0.92

Riprap Thickness = 0.91 m (3 ft)

	FS								
	Side Slope Z(H):1(V)								
	3:1			2.5:1			2:1		
ϕ', deg =	25	30	35	25	30	35	25	30	35
H									
m (ft)									
3.05 (10)	1.31	1.61	1.95	1.15	1.43	1.73	0.99	1.23	1.49
6.10 (20)	1.07	1.33	1.61	0.93	1.15	1.39	---	0.96	1.16
9.14 (30)	0.97	1.20	1.45	---	1.02	1.24	---	---	1.02

From the results in Table 1, it can be seen that slopes of cohesionless soils with seepage parallel to slope are significantly more stable with the addition of a riprap blanket. Furthermore, the stability of the slope increases as the thickness of the blanket increases. Table 2 shows that the addition of a 0.61 to 0.91 m (2 to 3 ft) riprap blanket to a stream bank with seepage parallel to slope allows for the slope cotangent (Z) to be approximately half of that for the same slope with no riprap.

Table 2. Slope Cotangent (Z) Required to Produce a Safety Factor = 1.0 for Seepage Parallel to Slope.

	Side Slope, (Z(H):1(V)					
ϕ', deg =	25°		30°		35°	
H, m (ft) =	3.05 (10)	6.10 (20)	3.05 (10)	6.10 (20)	3.05 (10)	6.10 (20)
Riprap Thickness, m (ft)						
0 (0)	4.5	4.5	3.6	3.6	3.0	3.0
0.61 (2.0)	2.4	3.1	1.7	2.4	1.3	1.9
0.91 (3.0)	2.0	2.8	1.3	2.1	1.2	1.6
>> 0.91 (3.0)	1.2	1.2	1.2	1.2	1.2	1.2

For slopes of cohesionless soil with seepage parallel to slope, the riprap protection should extend to the upper limit of the seepage, since any unblanketed portion of the seepage zone is subject to instability by the typical infinite slope failure mechanism (i.e., shallow sloughing). Whenever seepage is expected to exit the slope through the riprap blanket, then filter compatibility should be provided between the base soil and the riprap by means of graded sand and gravel filter layers and/or geotextiles.

With Cohesive Soils

For a slope of cohesive soil, the failure surface is typically circular, and the most severe seepage condition is the rapid drawdown case. For illustration purposes, a 6.10 m (20 ft) high slope of a hypothetical cohesive soil was analyzed using the SLOPE/W program to determine the effect of varying the height of riprap placed on the slope. The rapid drawdown condition was assumed, with the phreatic surface location varying from half-bank to full-bank height. A typical example of the model used is shown in Figure 6.

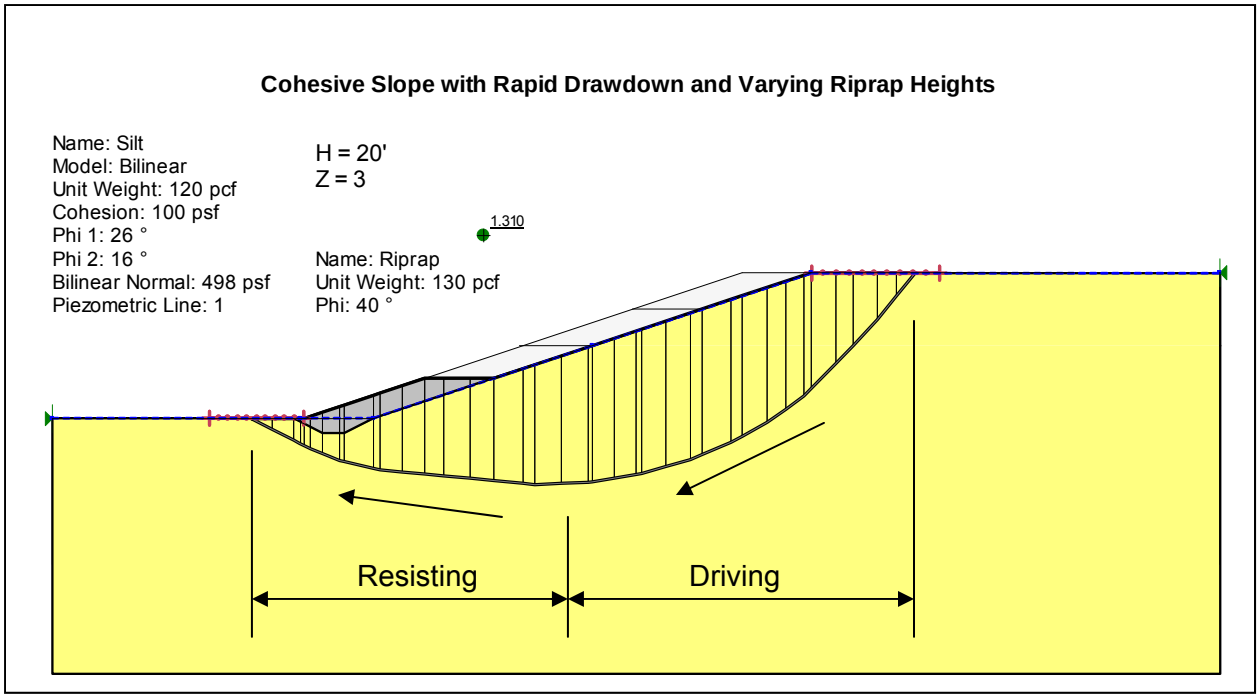

Figure 6. Typical Failure Surface for Cohesive Slope With Varying Riprap Height

In Figure 7, the relationship between riprap height and safety factor is shown. It is seen that the safety factor reaches a maximum when the riprap is placed to the mid-height of the slope. If riprap is placed above this level, the additional weight adds to the driving (downhill) forces acting on the failure mass. In contrast, the riprap on the lower half of the slope adds to the resisting (uphill) forces (see Figure 6).

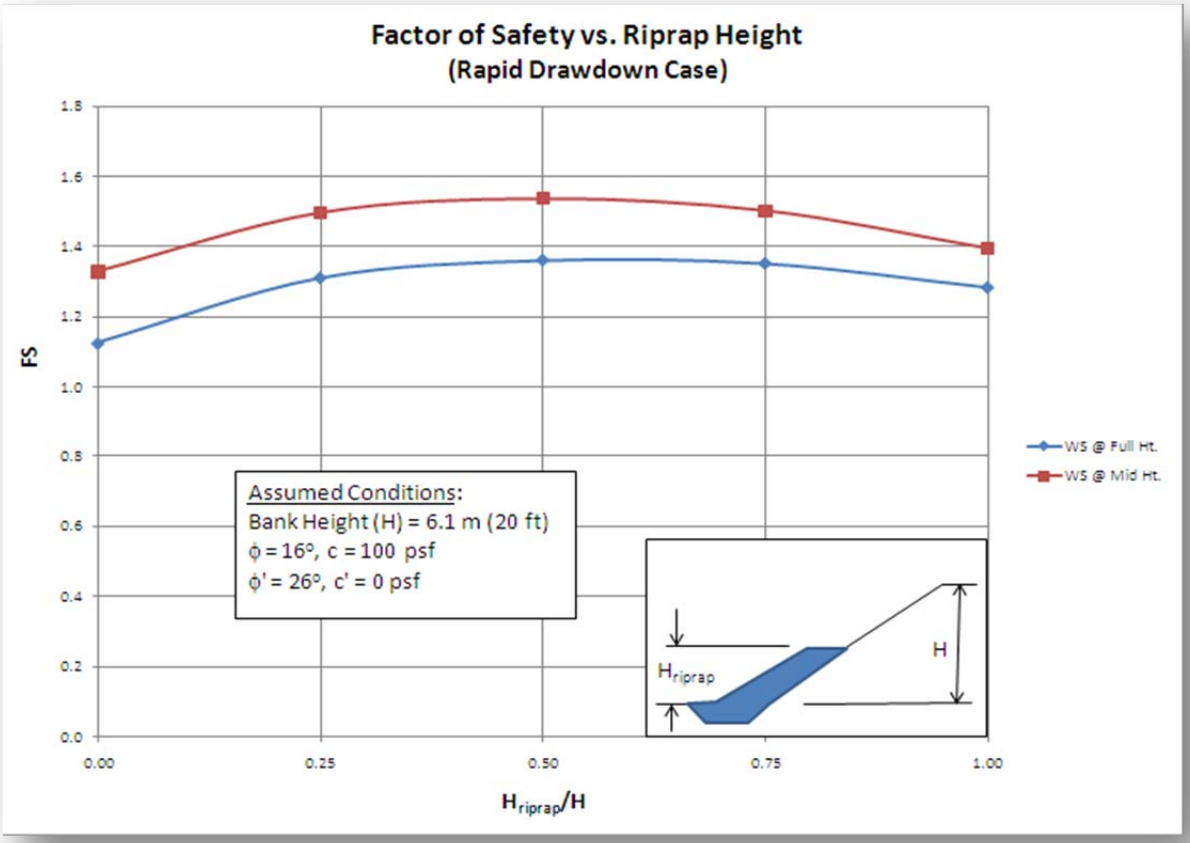

Figure 7. Safety Factor vs. Riprap Height for a Cohesive Slope With Rapid Drawdown Seepage Condition in Stream Banks

Critical Seepage Conditions for Stream Bank Stability

As demonstrated in the section above on infinite slope analysis, the seepage condition in the slope has a profound influence on the stability of the slope. Stream bank slopes can remain stable for years until an extreme hydrologic event produces destabilizing seepage conditions. For infinite slope-type situations, the most critical seepage conditions are seepage parallel to slope and horizontal flow. Which of these two conditions will develop in a given slope depends on factors such as stratigraphy and groundwater conditions in the slope, relative permeabilities of the various layers, and the relative speed of inundation and drawdown as the flood hydrograph passes through the reach in question.

In general, the most critical seepage condition is that in which the groundwater level in the slope is the highest relative to the water level in the channel. Typically, this corresponds to the rapid drawdown condition, which takes place during the recession phase of the flood hydrograph. To demonstrate the effect of soil permeability on slope stability during rapid drawdown, a parametric analysis was performed in which the position of the groundwater surface in a slope is determined during the rise and fall of the flood hydrograph, and the only parameter that is changed is the permeability of the soil. This analysis was performed using the transient analysis feature in the SEEP/W computer program. The stream bank is assumed to have a

height of 6.10 m (20 ft), and the side slope is 2(H):1(V). The flood hydrograph is as shown in Figure 8.

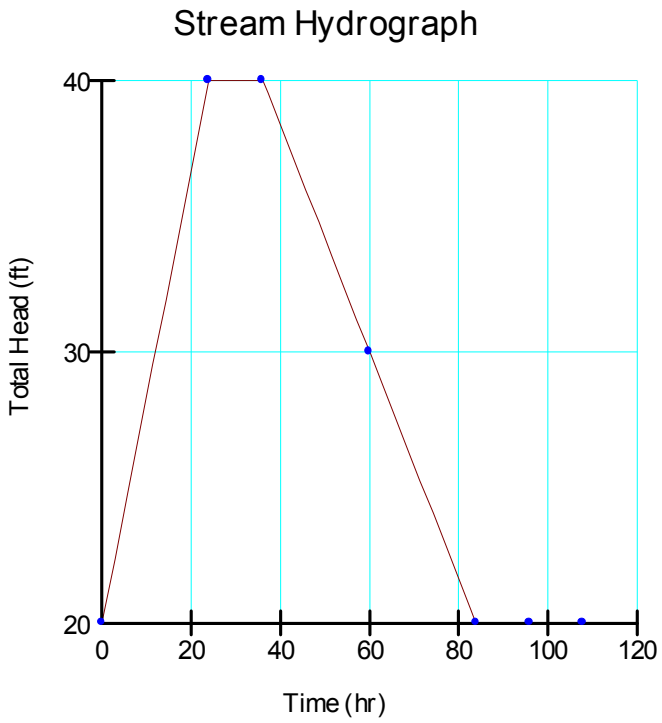

Figure 8. Assumed Flood Hydrograph for Parametric Analysis

Four different soil types are analyzed, ranging from a permeable sand to a low permeability clay. The soil classifications and permeability values are shown in Table 3. Graphical results for each of the four trials are shown in Figures 9 through 12, respectively.

Table 3. Soils Used in Parametric Analysis.

Soil Description	Unified Soil Classification Symbol	Saturated Permeability, cm/sec
Uniform Sand	SP	2×10^{-3}
Sandy Loam	SM	6×10^{-4}
Silty Gravel	GM	1×10^{-5}
Clay/Silt	CL	2.5×10^{-6}

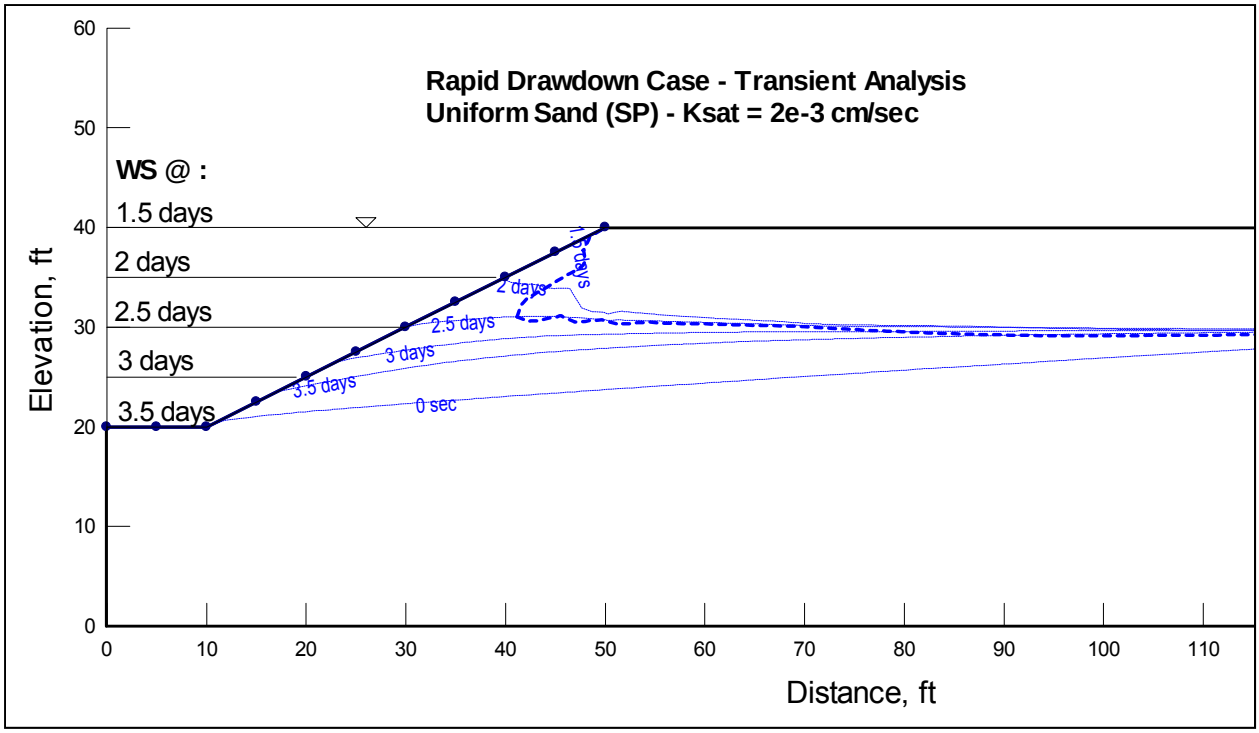

Figure 9. Phreatic Surface Location vs. Time for Uniform Sand

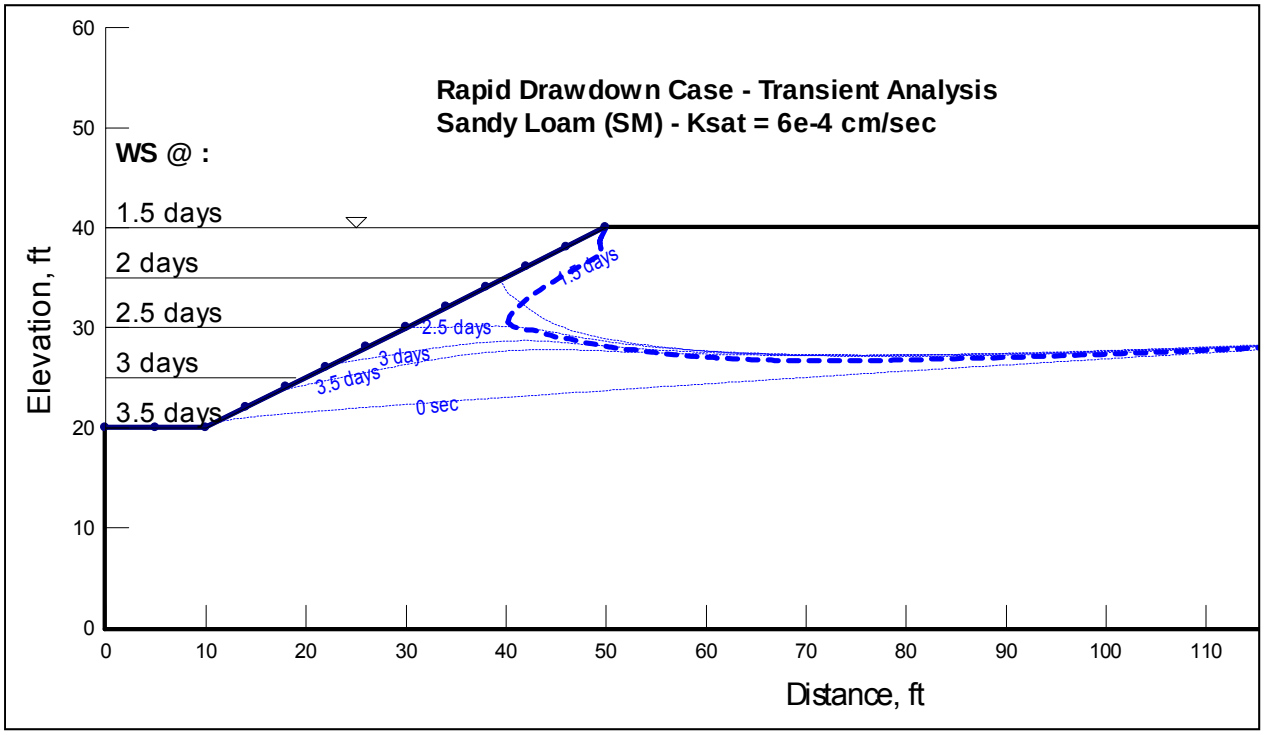

Figure 10. Phreatic Surface Location vs. Time for Sandy Loam

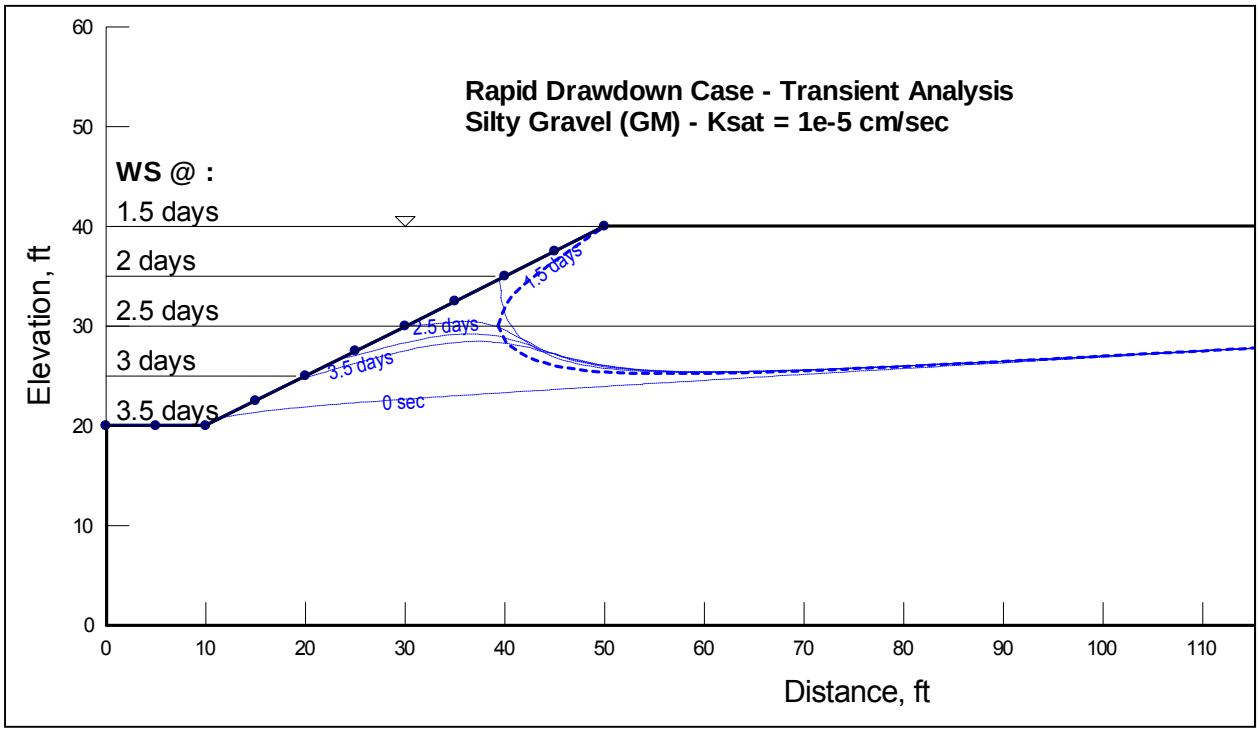

Figure 11. Phreatic Surface Location vs. Time for Silty Gravel

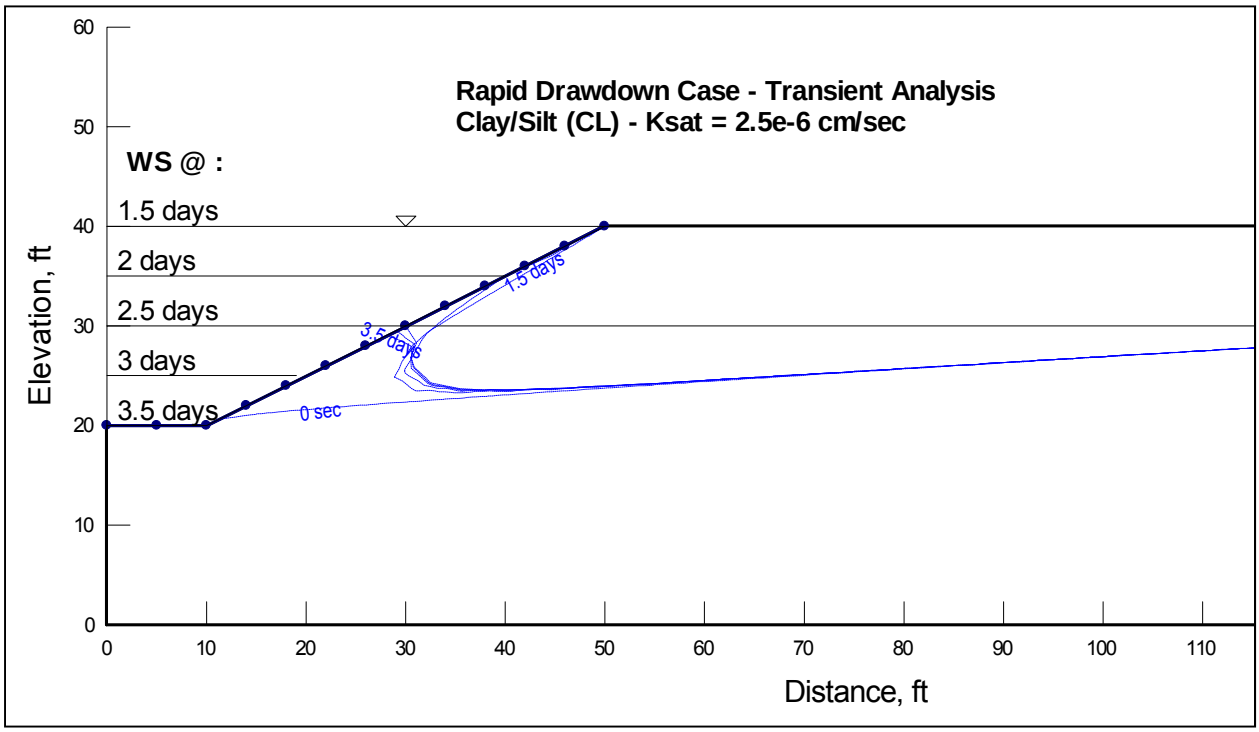

Figure 12. Phreatic Surface Location vs. Time for Clay/Silt

In Figures 9 and 10, it can be seen that the SP and SM soils are permeable enough to allow the phreatic surface in the bank to rise and fall with little or no lag behind the water surface in the channel. In both cases, when the water surface reaches the bottom of the channel at 84 hours (3.5 days), the phreatic surface is about 0.9 m (3 ft) above the channel bottom. At this point in time, the seepage flowing out of the bank may be sufficient to cause instability at the toe, unless it is sufficiently reinforced with plant roots, soil bioengineering measures, or riprap. Higher on the slope, the no-flow seepage condition will exist throughout the entire duration of the flood hydrograph.

In Figures 11 and 12, it can be seen that as the permeability of the soil decreases, the lag between the position of the phreatic surface in the bank and the water level in the soil increases. For the GM soil, the phreatic surface is about 1.52 m (5 ft) above the channel bottom at the end of drawdown. For the CL soil, the phreatic surface is about 2.74 m (9 ft) above the channel bottom at the same time. Since the CL soil will have some cohesion, the elevated phreatic water surface will not trigger an infinite slope-type failure (i.e., shallow sloughing), but, instead, the potential failure surface will have a deeper, circular orientation. In this case, surface or shallow revetments will have little or no value, unless they add stabilizing weight to the resisting portion of the failure mass (see Figure 6).

Conclusion

Analyzing stability of stream banks requires careful consideration of the geotechnical behavior of the soils and the impact of critical seepage conditions. Specific conclusions include:

1. Full height riprap blankets dramatically improve stability on slopes of cohesionless soil with seepage parallel to slope.
2. For riprap revetments on partially or fully saturated cohesive slopes, maximum stability is achieved by extending riprap only to the approximate mid-height of the slope.
3. Stream bank stabilization measures should be consistent with the anticipated geotechnical failure mode, particularly the depth and shape of the failure surface.

References

Fang, H. Y. 1991. *Foundation Engineering Handbook*. 2nd ed. Norwell, Mass.: Von Nostrand Reinhold/Kluwer Academic Publishers.

Lambe, T. W. and R. V. Whitman. 1969. *Soil Mechanics*. New York, N.Y.: John Wiley and Sons.

Mackey, R. and R. Thiel. 2003. Design of Waste Containment Liner and Final Closure Systems, In *Educating for Engineering Solutions, Geosynthetics 2003 Conf*. Atlanta, Ga.

Naval Facilities Engineering Command. 1982. *NAVFAC DM-7.1, Soil Mechanics*. Alexandria, Va.: Department of the Navy.

Terzaghi, K. and R. B. Peck. 1967. *Soil Mechanics in Engineering Practice*. 2nd ed. New York, N.Y.: John Wiley and Sons.

Terzaghi, K., R. B. Peck, and G. Mesri. 1996. *Soil Mechanics in Engineering Practice*. 3rd ed. New York, N.Y.: John Wiley and Sons.

U. S. Army Corps of Engineers. 1992. *EM 1110-1-1905, Bearing Capacity of Soils*. Washington, D. C.: Department of the Army.

U. S. Army Corps of Engineers. 1970. *EM 1110-2-1902, Stability of Earth- and Rock-Fill Dams*. Washington, D. C.: Department of the Army.