LANDSLIDES OF THE CINCINNATI, OHIO, AREA

Contribution of Artesian Water to Progressive Failure of the Upper Part of the Delhi Pike Landslide Complex, Cincinnati, Ohio

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Contribution of Artesian Water to Progressive Failure of the Upper Part of the Delhi Pike Landslide Complex, Cincinnati, Ohio

By REX L. BAUM

LANDSLIDES OF THE CINCINNATI, OHIO, AREA

U.S. GEOLOGICAL SURVEY BULLETIN 2059–D

Analysis of saturated ground-water flow and slope stability of a landslide in thin colluvium to determine where failures probably began

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LIST OF SYMBOLS

c'	Residual cohesion for effective stress
D	Vertical distance from ground surface to slip surface
E	Resultant normal force acting on a vertical plane in a landslide
F	Overall factor of safety
F'	Initial local factor of safety
h	Static head of water
h_ε	Amount of error in a computed solution to the ground-water flow equation due to an error in the boundary conditions
H	Average distance from the water table to the failure surface measured normal to the slope
K_1, K_2	Hydraulic conductivities of isotropic layers of material shown in figure 4
K_x, K_z	Hydraulic conductivity of materials in directions parallel to x and z Cartesian coordinates, respectively
p	Pore pressure
Q_{xl}, Q_{zb}	Total flux of water at the left-hand and basal boundaries of model
r	Ratio of Q_{xl} to Q_{zb}
T	Resultant shear force acting on a vertical plane in a landslide
t	Average thickness of landslide debris, measured normal to the slope
x, z	Cartesian coordinates, horizontal and vertical, respectively
β	Average slope angle, measured clockwise from the horizontal
γ_t	Unit weight of wet soil
γ_w	Unit weight of water
ε	An error in boundary conditions
θ	Local slope of slip surface
λ_1, λ_2	Angles measured from a line normal to the contact between materials to flow lines in figure 4
σ_{nn}	Normal stress acting on the slip surface
σ_{ns}	Shear stress acting on the slip surface
ϕ'	Angle of residual friction for effective stress
ψ	Pressure head, p/γ_w

CONTRIBUTION OF ARTESIAN WATER TO PROGRESSIVE FAILURE OF THE UPPER PART OF THE DELHI PIKE LANDSLIDE COMPLEX, CINCINNATI, OHIO

By Rex L. Baum[1]

ABSTRACT

Artesian water in bedrock beneath a colluvium-mantled slope can influence the stability of the slope, either by causing local failure that leads to progressive failure of the colluvium or by reducing the overall strength of the colluvium and causing failure. Artesian water under at least 1.5 m of pressure head occurs in limestone layers of weathered shale and limestone bedrock at the Delhi Pike landslide complex in Cincinnati, Ohio. Colluvium, 1–2 m thick on the upper part of the slope, mantles the weathered bedrock. The colluvium slips on an undulating failure surface that coincides approximately with the top of the weathered bedrock. Several limestone layers are in contact with the colluvium near the failure surface.

A model of ground-water flow in ground beneath the slope, for saturated flow assumed to occur when the landslide is active, indicates that seepage in the colluvium is nearly horizontal near the contacts with the limestone beds, but seepage may have a strong downward component away from the contacts. Consequently, relative maxima in the distribution of pore pressure at the slip surface are near the limestone beds and relative minima occur along other parts of the failure surface.

Analysis indicates that artesian water flowing from the limestone beds has no significant effect on the overall stability of the thin colluvium at Delhi Pike, and plays only a minor role in determining the positions of initial local failures that lead to progressive failure. Stability analysis indicates that the overall factor of safety due to the pattern of ground-water flow determined by the model is approximately equal to the factor of safety due to slope-parallel seepage with the water table at the ground surface. The role of the artesian water in determining the positions where local failures start was assessed by computing local factors of safety at many points along the failure surface. Positions of relative minima in the graph of local factors of safety coincide with locally steep parts of the slip surface. A few relative maxima of the pore pressure also coincide with positions of relative minima of the local factors of safety. However, the local slope of the developing slip surface appears to have an overriding influence on the positions of initial failures in slopes undergoing progressive failure.

INTRODUCTION

Artesian water in bedrock beneath colluvium-covered slopes is thought to be a major cause of landsliding in the colluvium (Zaruba and Mencl, 1982; Deere and Patton, 1971). In humid climates, artesian water commonly occurs in horizontally layered shale alternating with permeable rock, such as sandstone or fractured limestone (Deere and Patton, 1971). Clay-rich colluvium that covers the layered bedrock has low permeability and impedes the flow of water out of the permeable bedrock layers. Consequently, high pore pressures can develop in the colluvium near the contact with the permeable bedrock layers and can contribute to sliding of the colluvium.

Artesian water beneath colluvium can contribute to failure of the colluvium by reducing the overall factor of safety; furthermore, it can contribute to progressive failure, the gradual growth in area of a slip surface from an initial, localized failure (Bishop, 1968), by reducing local factors of safety. The local factor of safety is the factor of safety on a small part of the potential failure surface; the overall factor of safety is an average factor of safety taken over the entire potential failure surface. The factor of safety against sliding is commonly defined as the ratio of available strength to the

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strength mobilized along the slip surface (Morgenstern, 1974). Analysis of the influence of artesian water on the local and overall factors of safety requires a detailed knowledge of the distribution of pore pressures along the slip surface.

An opportunity to investigate the role of artesian water in the failure of colluvium arose when artesian water was discovered in limestone layers of weathered shale and limestone bedrock beneath the Delhi Pike landslide complex near Cincinnati, Ohio (Fleming and others, 1981). During exploratory drilling, water flooded previously dry boreholes as the auger penetrated the first layer of limestone in the weathered shale and limestone bedrock beneath the landslide. A small-diameter standpipe was sealed into one of these layers, and measurements through the following year showed that water levels in the standpipe rose as much as 1.5 m above the top of the limestone bed.

Several workers have modeled ground-water flow beneath slopes and analyzed its effect on slope stability. Hodge and Freeze (1977) analyzed some ground-water models that included confined permeable layers within weathering profiles. They determined that high pore pressures can occur near the toe of a slope in the confined layers, and that the high pore pressures can significantly reduce stability of a slope. Rulon and Freeze (1985) modeled steady, saturated and unsaturated seepage in horizontally layered, unconfined media beneath a hillside and showed that the distribution of pore pressure in their model produces a significantly higher factor of safety than the distributions commonly used in stability analyses. However, they did not analyze flow in layered media beneath slopes mantled by low-permeability colluvium. Wilson and Dietrich (1987) monitored ground water in a colluvium-filled hollow; they found that topography and heterogeneities in the hydraulic conductivity of bedrock beneath the colluvium determined the pattern of ground-water flow in bedrock, which, in turn, determined pore pressures in the colluvium. Iverson and Major (1986) clarified the roles of seepage direction and magnitude of the hydraulic gradient in the failure of long, uniform slopes. They found that, other things being equal, horizontal seepage is the most unstable condition if the friction angle of the soil is equal to the slope angle of the ground surface.

Progressive failure of slopes was an active field of research from the late 1960's through the 1970's; however, the influence of seepage on progressive failure has received little attention. The idea of progressive failure, meaning the gradual growth in area of a slip surface from an initial, localized failure, has existed for at least 40 years (Terzaghi and Peck, 1948, p. 91; Taylor, 1948, p. 342–343). Bjerrum (1967) proposed a conceptual model for progressive failure from a cut at the base of a long, uniform slope. Christian and Whitman (1969), Palmer and Rice (1973), and others subsequently analyzed Bjerrum's model and determined conditions that can cause a slip surface to propagate. Romani and others (1972) analyzed progressive formation of rotational failures in slopes. Burland and others (1977) studied a field example of progressive failure due to excavation in a clay pit and observed evidence that a slip surface was propagating into a slope at the base of a cut. Much of the work on progressive failure has focused on progressive failure due to excavation, while neglecting any effects due to seepage.

I have modeled ground-water flow within the colluvium at part of the Delhi Pike landslide complex near Cincinnati, Ohio, and analyzed the role of artesian ground water in progressive failure of the colluvium. In sections that follow, I describe and analyze the physical characteristics of the landslide and materials beneath it that seem relevant to ground-water seepage and slope stability. These data are taken largely from Fleming and others (1981), supplemented by my own observations. Based on these observations, I formulate and use a model of ground-water flow to analyze possible seepage patterns. I then use pressure-head data from the ground-water model to compute local factors of safety, in order to study the influence of seepage on the locations of initial failures that lead to progressive failure of the slope.

ACKNOWLEDGMENTS

Robert Fleming (U.S. Geological Survey) and Arvid Johnson (Purdue University) shared data and observations from the Delhi Pike landslide.

THE DELHI PIKE LANDSLIDE

The Delhi Pike landslide is a thin translational slide that is part of a much larger landslide complex on a steep slope (fig. 1). The landslide complex is on the side of a ridge that is about 110 m high and several kilometers long. The complex extends for about 1 km along the side of the ridge and measures 100–200 m perpendicular to the ridge (Fleming and Johnson, 1994). The landslide complex consists of landslides as much as 2 m thick uphill from Delhi Pike and landslides as much as 5 m thick downhill from Delhi Pike. The Delhi Pike landslide was active during several years in the 1970's and has been studied by Fleming and others (1981). At the conclusion of their studies, they excavated a trench through the lower part of the Delhi Pike landslide (fig. 2). Data from the trench have been used in this analysis. The thickness of the landslide in the trench ranges from 0.3 to 1.8 m and averages 1.2 m. The ground surface undulates gently and has an average slope of 18.2°. The slip surface also undulates; its slope ranges from 0° to 60°, and averages 18.7°.

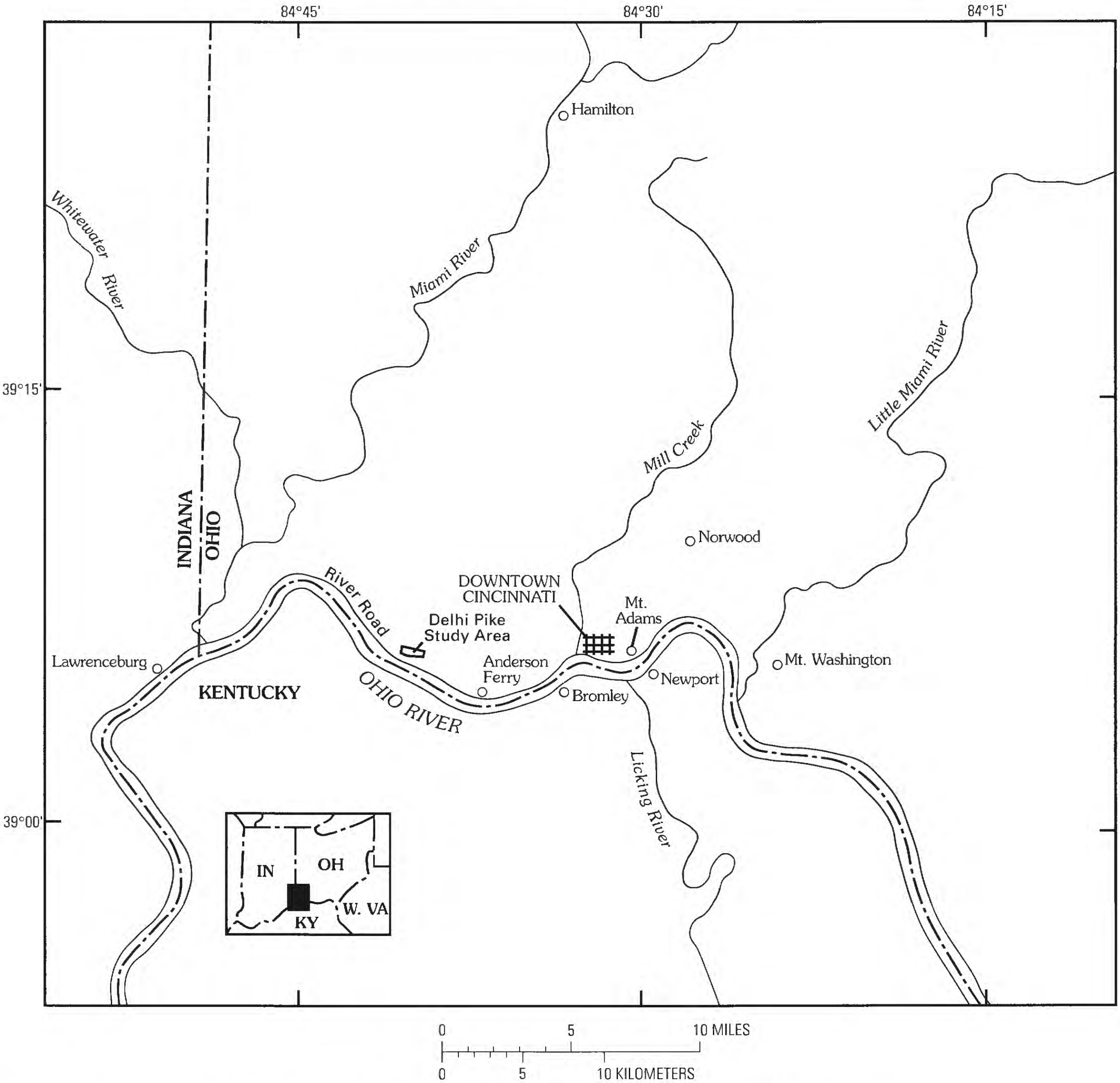


Figure 1. Location of the Delhi Pike landslide complex, Cincinnati, Ohio. (From Fleming and Johnson, 1994.)

Landslide debris at Delhi Pike consists of stony, clayey colluvium derived from weathered limestone and shale bedrock. The surface layer of the colluvium typically is a dark-brown silty clay, as much as 0.6 m thick. The lower layer of colluvium is a yellowish-brown to brown silty clay, 1–2 m thick, containing pebble-, gravel-, and cobble-sized clasts of gray limestone. The upper part (soil B horizon) of the yellow-brown colluvium is formed into blocky peds, 0.5 cm across, and free water fills fractures between the peds in the spring (Fleming and others, 1981). The ped structure diminishes with depth.

The base of the colluvium grades to weathered bedrock within a few centimeters. The slip surface generally occurs at the top of the weathered bedrock, in a paper-thin layer of clay. Colluvium above the failure surface has a higher water content than weathered bedrock immediately below the slip surface. Measurements showed that colluvium immediately above the slip surface contained 27 percent water (based on dry weight) and the material beneath contained 19 percent water (Fleming and others, 1981, p. 550).

The weathered bedrock consists of fractured brown shale and jointed limestone beneath the colluvium. The shale is uncemented and traversed by many hairline fractures; it breaks into small blocky clasts, 2–10 cm across. Several beds of limestone from 0.1 to 0.3 m thick, separating beds of weathered shale from 0.1 to 3.0 m thick, are evident in the

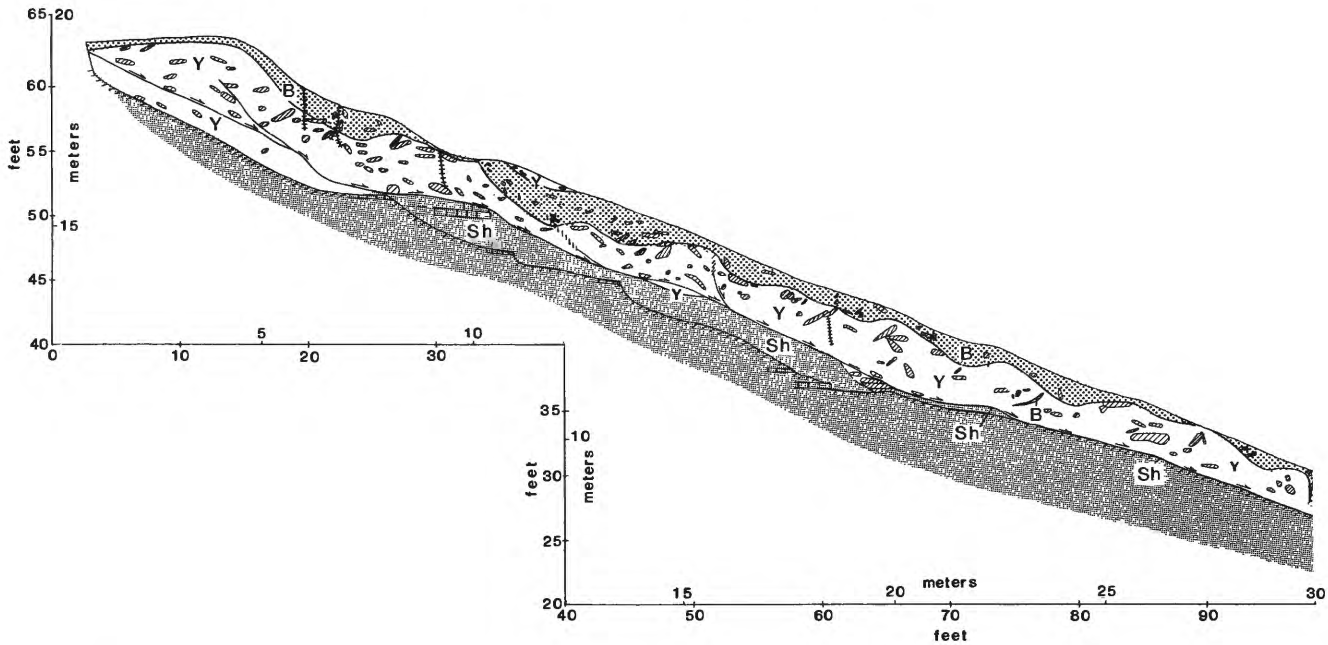


Figure 2 (above and facing page). Cross section of east wall of trench through part of Delhi Pike landslide, Cincinnati, Ohio. Figure reduced from Fleming and Johnson (1994, pl. 1).

cross section (fig. 2). Vertical joints, from 0.1 to 0.4 m apart, break limestone beds into rectangular, triangular, or rhomboidal blocks. The joints are open 0–5 mm (Baum, 1983). About 3–4 m beneath the ground surface (2–3 m below the contact with colluvium), the bedrock is relatively unweathered; fractures and joints are narrower than those near the surface, and ground-water seepage diminishes with depth. Bedding is almost horizontal; the regional dip of bedrock in the Cincinnati area is 1–2 m/km toward the northwest (Fleming and others, 1981, p. 543).

Ground-water flow beneath the slopes tends to concentrate in the colluvium and the upper part of the weathered bedrock (Robert W. Fleming, oral commun., 1988). Water seeping from limestone layers in bedrock exposed at an abandoned quarry in western Cincinnati freezes into icicles on the face of the quarry during the winter (fig. 3). The largest volume of ice forms at the topmost layer of limestone in the weathered bedrock. The volume of ice diminishes rapidly at successively deeper layers so that the combined volume of ice on all lower layers of limestone is only a few percent of the volume on the topmost layer.

The concentration of ground-water flow in the colluvium and upper part of the weathered bedrock is consistent with a model of seepage in a sloping layer of relatively permeable material overlying less permeable material. Figure 4 shows a flow net for saturated ground-water flow beneath an infinite slope in which the water table is a short distance below the ground surface. The sloping layer has hydraulic conductivity K_1 and the underlying material has hydraulic conductivity K_2, such that $K_1=20K_2$. The ground surface and the interface between the materials slope 20°. Seepage is assumed to be vertical below the interface. Vertical seepage is a limiting case for saturated flow in this example, because seepage with a component to the left in figure 4 would indicate that the material became unsaturated at some depth below the interface. The orientation of flow lines above the interface is determined by the tangent law (Freeze and Cherry, 1979, p. 172–173):

$$\tan \lambda_1=(K_1/K_2) \tan \lambda_2 .$$

In applying the formula for the tangent law to figure 4, λ_1 and λ_2 are angles measured counterclockwise from a line normal to the interface to the flow lines in the materials having K_1 and K_2, respectively. In figure 4, the average slope angle, β, is 20°, $K_1/K_2 = 20$, $\lambda_2 = 20°$, and $\lambda_1 = 82.2°$. Increasing K_1/K_2 or λ_2 will make λ_1 approach 90°, so that flow above the interface will become more nearly parallel to the slope. The relatively close spacing of flow lines above the interface in figure 4 indicates that flow is concentrated above the interface.

Landslides in thin colluvium tend to be active in the early spring, from late March through early May (Fleming and others, 1981). Rapid, spectacular response of shallow ground water to spring rainstorms indicates that locally the entire thickness of the colluvium is nearly saturated during this time of year (Haneberg, 1989; Haneberg and Gökce, 1994). For instance, the water level in a shallow boring rose

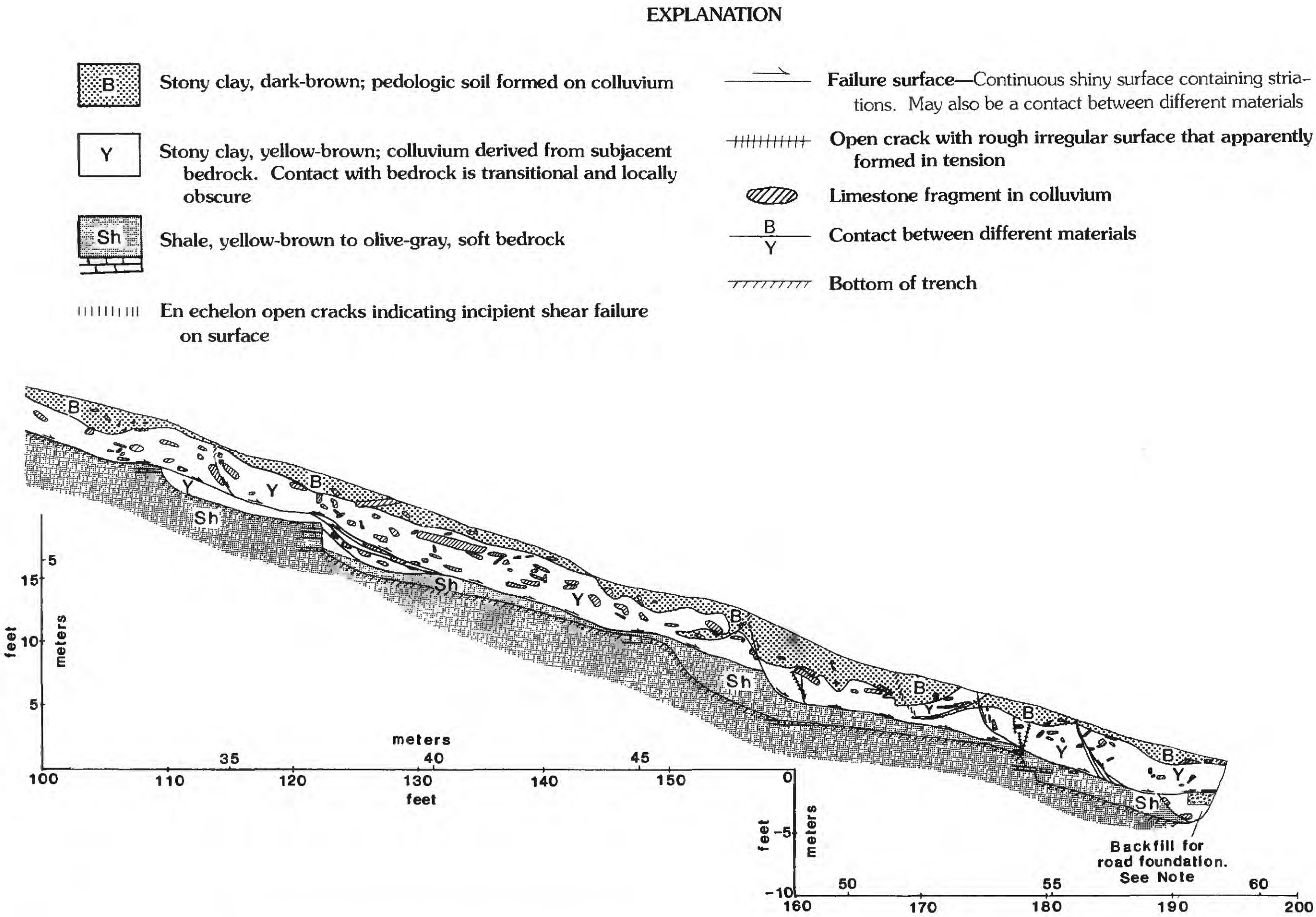


Figure 3. Icicles on the face of a quarry in western Cincinnati, Ohio. The amount of ice diminishes with depth below the topmost limestone layer. (Photograph by R.W. Fleming.)

50 cm in response to 1.7 cm of rainfall (Fleming and others, 1981, fig. 7). Such a large rise in water level is consistent with an unfilled porosity of no more than 3.4 percent (1.7 cm is 3.4 percent of 50 cm); thus, most of the void space must be filled with water. However, some borings remain dry during storms, and tensiometers indicate that some of the landslide debris remains unsaturated during storms (W.C. Haneberg, written commun., 1989).

Ground-water conditions in the Delhi Pike landslide were not observed at the time of failure, nor when the landslide was active, but data exist to constrain possible ground-water conditions at failure. Water levels and movement of the landslide were monitored for 5 yr during the 1980's by R.W. Fleming, A.Ö. Gökce, L. Murdock, and W.C. Haneberg. Their data indicate that the landslide debris did not become fully saturated (W.C. Haneberg, written commun., 1989) and did not move significantly during the time of monitoring. Stability analyses, based on residual strength of the landslide debris, as determined by laboratory tests and the well-known infinite-slope analysis (Lambe and Whitman, 1969, p. 352–356), indicate that the factor of safety[2] ranges from 0.63 to 1.21 (table 1) if the water table is at the ground surface and flow is parallel to the slope; it would be about 8 percent higher for flow like that shown in figure 4. The factor of safety is unity if flow is parallel to the slope and the water table is less than 0.95 m below the ground surface or if slightly artesian conditions exist (table 1). Although the strength parameters vary considerably, these calculations indicate that the water table is probably near the ground surface when the landslide is active. Thus, a reasonable assumption about ground-water conditions in the landslide is that the water table is at the ground surface when the landslide is active.

MODEL OF GROUND-WATER SEEPAGE

I have made several assumptions in modeling ground-water flow at Delhi Pike:

- Flow obeys Darcy's law.
- Flow is two dimensional.
- The porous medium is fully saturated and the top flow line is at the ground surface.
- Flow is steady.
- Flux at the left and lower boundaries of the model is known.
- Flow in fractured rock can be modeled as flow in an equivalent porous medium.

Ground-water flow, subject to these assumptions, in a vertical, two-dimensional cross section of a heterogeneous, anisotropic, porous medium is governed by

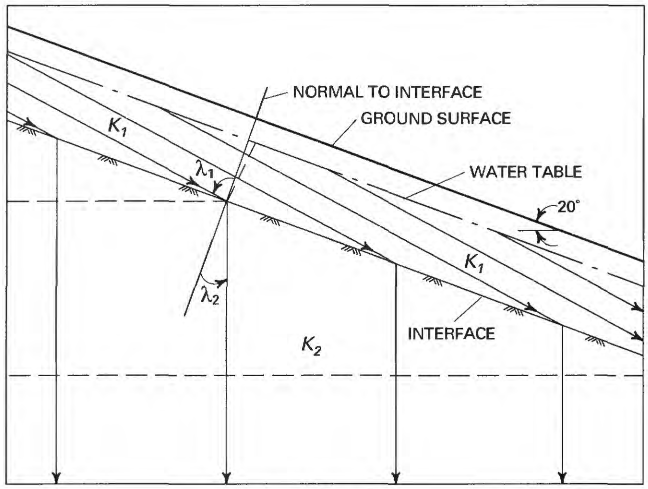
$$\frac{\partial}{\partial x}\left\{-K_x\frac{\partial h}{\partial x}\right\}+\frac{\partial}{\partial z}\left\{-K_z\frac{\partial h}{\partial z}\right\}=0. \qquad (1)$$

In equation 1, x and z are the horizontal and vertical Cartesian coordinate directions, respectively; h is the hydraulic head, and K_x and K_z are principal components of the hydraulic conductivity tensor. The principal axes of the conductivity tensor are assumed parallel to x and z. The head, h, is a measure of the potential of the fluid, and it has two main components, z and ψ,

$$h=z+\psi \qquad (2)$$

where ψ is the pressure head and z is the elevation above an arbitrary datum (the x-axis).

Equation 1 is solved subject to conditions of either specified head or specified flux at the boundaries of the problem domain. Head at the ground surface is set equal to the elevation of the ground surface, and flux across the other boundaries is specified in the model analyzed here. I have used finite differences on a square grid with the method of successive over-relaxation to solve the flow problem. Finite



EXPLANATION

– – – – Contour of constant static head

——► Flowline

Figure 4. Flow net showing directions of seepage in an infinite slope consisting of a permeable layer overlying less permeable material. The permeability, K_1, of the overlying layer is 20 times the permeability of the underlying material, K_2. Flow in the permeable layer is almost parallel to the slope (λ_1=82.2°). Flow in the less permeable material is vertical downward (λ_2=20°), a limiting case for saturated flow in this slope.

Table 1. Factors of safety and height of water table under varying conditions in the Delhi Pike landslide.

[Strength parameters from Fleming and Johnson (1994, table 3). Only those tests having c' less than 5.5 kN/m^2 were used here; higher values of c' are inconsistent with the shallow failure surface of the Delhi Pike landslide]

Residual strength, under effective stress		Factor of safety, F, if water table at ground surface	Height of water table in meters, if F=1	
Cohesion, c' (kN/m^2)	Angle of friction, ϕ'		Above slip surface	Relative to ground surface
0	24°	0.63	0.55	–0.65
3	21°	.96	1.11	–.09
5	20°	1.21	1.64	+.44
5.5	12°	1.06	1.42	+.22
3	12.5°	.73	.25	–.95

[2]The factor of safety, F, is determined by the formula

$$F=\frac{c'+(t\gamma_t-H\gamma_w)\cos\beta\tan\phi'}{t\gamma_t\sin\beta}$$

where c' is the residual cohesion for effective stress; ϕ' is the residual friction angle for effective stress; β is the average slope angle; t is the average thickness of the landslide debris (t=1.2 m for the Delhi Pike landslide); H is the height of the water table above the slip surface, assuming slope-parallel flow; γ_t is the unit weight of saturated soil (γ_t was assumed to be 18.8 kN/m^3); and γ_w is the unit weight of water, 9.8 kN/m^3.

difference formulas approximating equation 1 are found in Freeze and Cherry (1979, p. 181–185), and special formulas for the irregular boundary at the ground surface are in Smith (1985, p. 247–248).

Previously, I justified the assumption of total saturation with the water table at the ground surface; however, some of the other assumptions used in the model require some explanation.

In general, ground-water flow at Delhi Pike is not steady. Data of Fleming and others (1981, p. 551) indicate that shallow landslides are active during the times of rapid water-level rises associated with spring rainfall. I have used the assumption of steady flow to determine the pattern of seepage during times of highest water levels, which is the worst case for slope stability.

Flux across any given vertical or horizontal line in a slope is usually not known; thus, in designing models, regions of flow are commonly chosen that are much larger than the area of interest so that possible errors in the basal and lateral boundary conditions have a minimal effect on the results of the model (Hodge and Freeze, 1977, p. 468). This method of minimizing errors by modeling oversized regions of flow can be justified qualitatively by means of the principal of superposition and the properties of elliptic partial-differential equations. (See, for example, Weinberger, 1965; Kovach, 1984.) Let h represent the correct solution to equation 1 for a given set of boundary conditions, and let h_ε represent the amount of error in a computed solution to equation 1 due to an error in the boundary conditions, ε. The computed solution will be $h+h_\varepsilon$. According to the maximum value theorem (Weinberger, 1965, p. 55–57), $|h_\varepsilon|<|\varepsilon|$. If the error, ε, is small compared to the true boundary values, then $|h_\varepsilon|<<|h|$ and the computed solution will closely approximate the correct solution. Furthermore, the magnitude of h_ε decreases with increasing distance from the boundary where the error exists (Kovach, 1984, figure 4.4–4, p. 177, or 4.4–6, p. 179). Thus, small errors in boundary conditions that are relatively far from the area of interest will have a very small effect on the solution for the area of interest.

Detailed data about the ground upslope and downslope from the trench shown in figure 2 are not available, so I put the left boundary of the model near the upper end of the trench. The lower end of the trench coincides with the toe of the landslide element within the larger landslide complex, and the basal boundary of the model is approximately at the elevation of the bottom-right side of the trench. I expect the model to yield relatively accurate results near the slip surface, except possibly near the head and toe, where the arbitrary left and lower boundaries of the model intersect the ground surface. The Delhi Pike landslide is shallow, so the slip surface is close to the ground surface in the model, and the geometry of the ground surface is well known. For saturated flow with the water table at the ground surface, the boundary conditions at the ground surface are accurately known. Thus, the known boundary conditions at the ground surface should dominate the solution near the slip surface, except near the head and toe of the slope, where arbitrary boundaries of the model are close enough to influence the solution significantly.

The horizontal component of the hydraulic gradient at the left boundary of the model, $\{\partial h(0,z)/\partial x\}$, and the vertical component of the hydraulic gradient at the bottom boundary, $\{\partial h(x,0)/\partial z\}$, were set equal to constants. Thus, the flux across these boundaries varies in proportion to the hydraulic conductivity. Furthermore, the total flux at the left boundary, Q_{xl}, and the total flux at the basal boundary, Q_{zb}, were also constants. I varied the ratio of the fluxes, r,

$$r=(Q_{xl}/Q_{zb}) \ , \tag{3}$$

in successive runs of the model to determine how sensitive the model was to the boundary conditions.

The boundary conditions used in the model would produce straight flow lines at some constant angle to the ground surface of a hill having a planar surface and consisting of an isotropic, homogeneous medium. The slope of the flow lines would depend on the ratio, r, in such a model. Topographic irregularities on the ground surface, and inhomogeneities and anisotropy within the slope, perturb the pattern of straight flow lines (Iverson and Major, 1986; Wilson and Dietrich, 1987).

The degree to which flow in fractured rock can be successfully modeled by an equivalent continuous medium is partly a matter of scale. For instance, rock containing many closely spaced, well-connected fractures can be successfully modeled as an equivalent continuous medium (Freeze and Cherry, 1979, p. 73). Fractures within the weathered bedrock at Delhi Pike have spacings on the order of centimeters to tens of centimeters and seem to be well connected. The slope being modeled has dimensions on the order of tens of meters, and nodal spacings in the model are 0.76 m. Thus, the spacing of fractures is small compared to the overall size of the model and individual units within the model, so I expect the representation of the fractured bedrock as an equivalent porous medium to yield acceptable results.

A possible distribution of hydraulic conductivity for the materials making up the slope is shown in the cross section in figure 5. In modeling steady flow, the ratios of conductivities between different materials are much more important than the actual values of the hydraulic conductivities. Thus, I have assigned conductivities to various materials in the slope on the basis of their permeabilities relative to the colluvium. The colluvium is assigned horizontal and vertical conductivities of 1.0×10^{-3} cm/s, on the basis of laboratory measurements by Fleming and others (1981). I assumed that weathered shale is 3–5 times more permeable than the colluvium, because it is highly fractured, and assigned K_x = 5.0×10^{-3} cm/s and $K_z = 3.0\times10^{-3}$ cm/s.

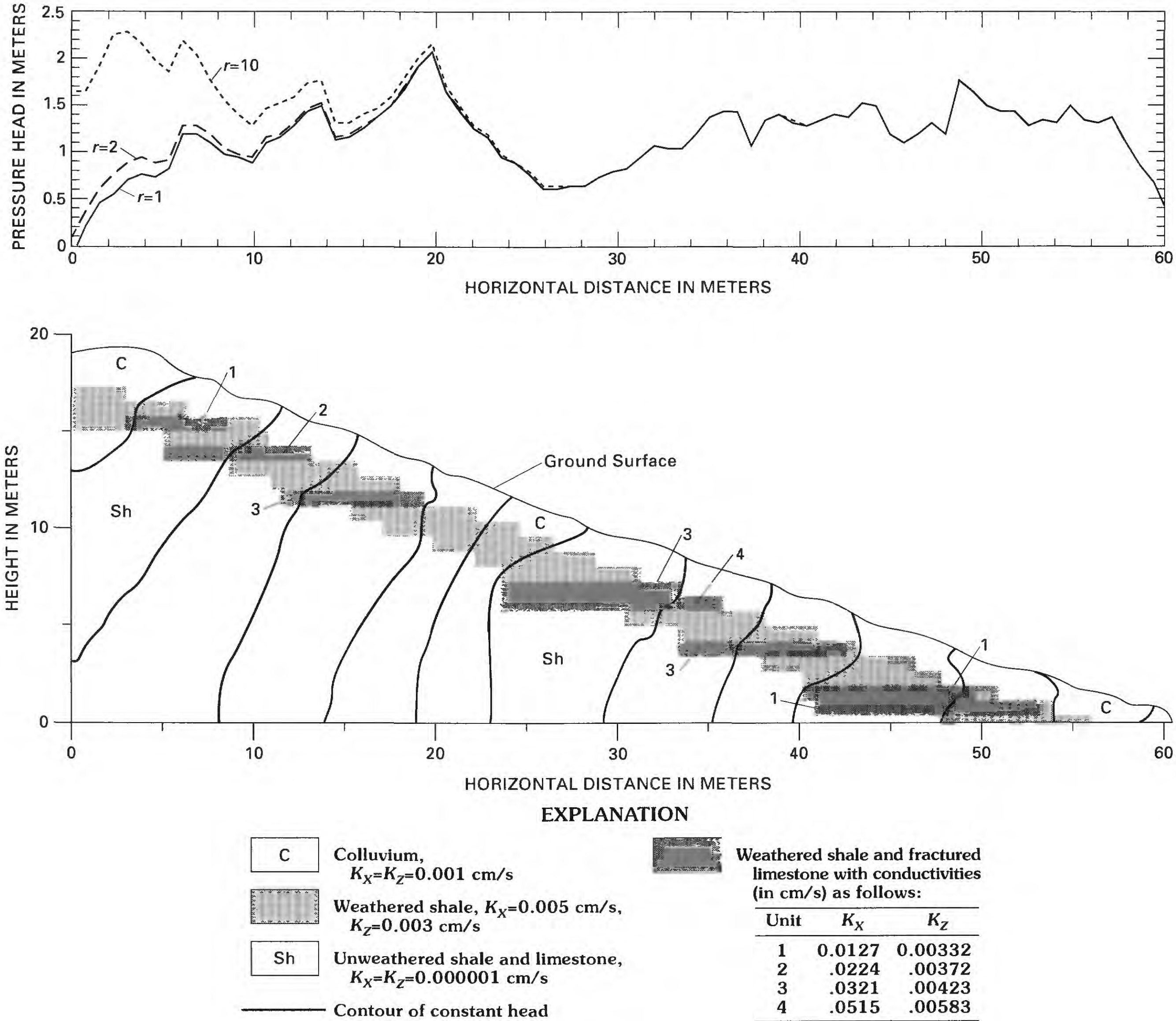
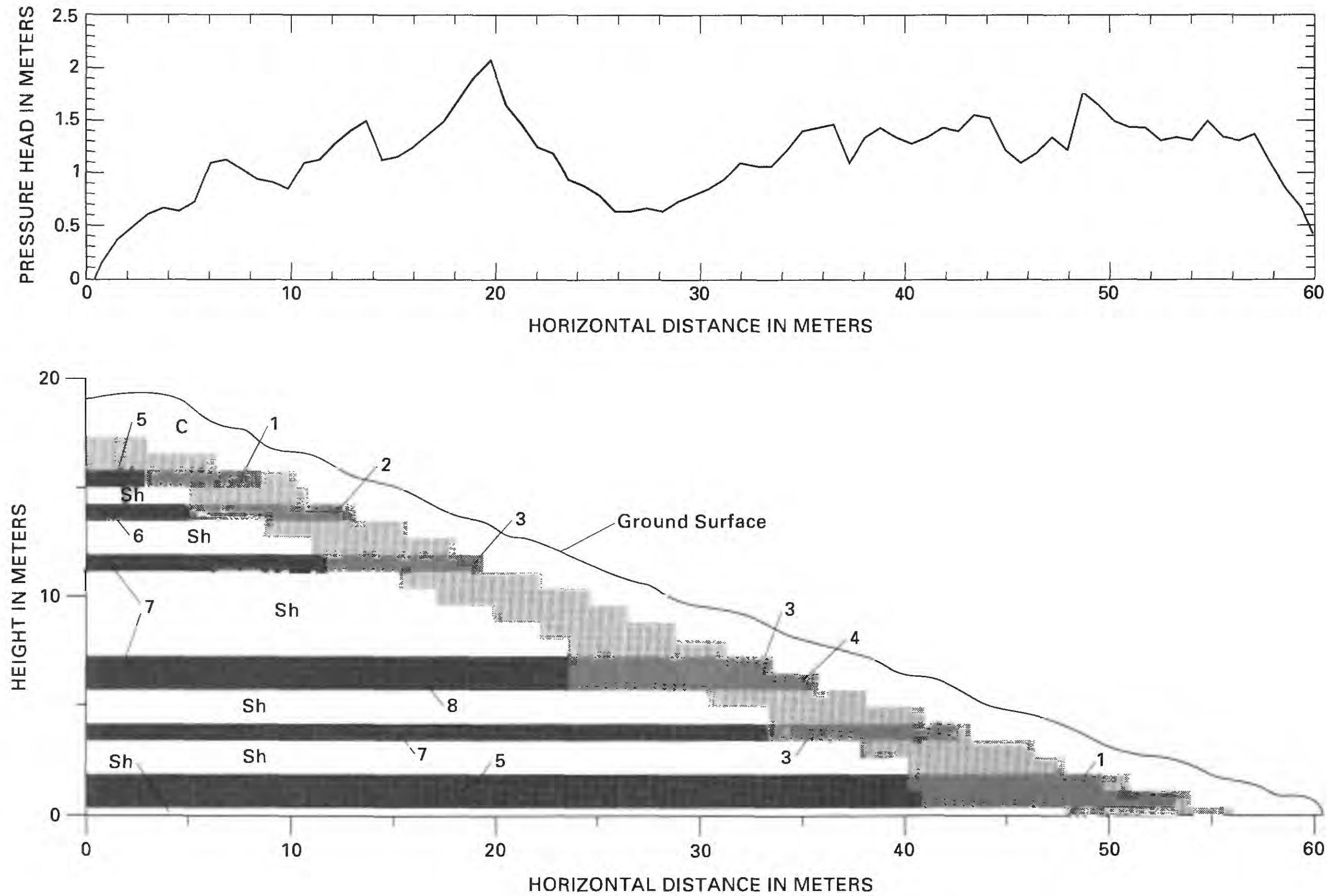


Figure 5. Modeled distribution of hydraulic conductivity in the Delhi Pike landslide and the resulting pressure head at the slip surface, assuming high permeability in weathered bedrock (shale) and low permeability in unweathered bedrock (limestone). The horizontal hydraulic gradient at the left boundary, $\partial h/\partial x$, is –0.1183, and the vertical hydraulic gradient at the basal boundary, $\partial h/\partial z$, is 0.0529. Pressure-head distributions are projected based on three different ratios of influx at the left boundary to outflux at the basal boundary (r=1, r=2, r=10).

Fractured limestone was assigned a conductivity of 0.1 cm/s, consistent with joint spacing of 0.1 m and average joint opening of 0.5 mm. (The conductivity of fractured rock can be estimated by means of equation 2.86 of Freeze and Cherry, 1979, p. 75). The limestone beds were assumed to be horizontal in the model. Individual beds within the weathered shale and fractured limestone unit in figure 5 are too thin to represent in the model, because the node spacing used was 0.76 m, and the thickest limestone bed is only 0.2 m, so average vertical and horizontal conductivities (Freeze and Cherry, 1979, p. 33–34) were assigned to the combined units based on the relative amounts of shale and limestone they contain. These conductivities are listed in figure 5. Unweathered bedrock is assigned a conductivity of 1×10^{-6} cm/s in both the horizontal and vertical directions. Freeze and Cherry (1979, p. 158) indicate that unweathered shale at depth generally has very low permeability. The upper limit of hydraulic conductivity for intact shale and limestone beds is much less than the assumed value of 1×10^{-6} cm/s (Freeze and Cherry, 1979, p. 29); however, the assumed conductivity of unweathered bedrock is three orders of magnitude lower than the conductivity of the colluvium and weathered shale. Hence, the calculated flow in the unweathered bedrock is negligible, in accordance with field observations and the model in figure 4.



EXPLANATION

C — Colluvium, $K_X = K_Z = 0.001$ cm/s

Weathered shale, $K_X = 0.005$ cm/s, $K_Z = 0.003$ cm/s

Weathered shale and fractured limestone with conductivities (in cm/s) as follows:

Unit	K_X	K_Z
1	0.0127	0.00332
2	.0224	.00372
3	.0321	.00423
4	.0515	.00583

Sh — Unweathered shale, $K_X = K_Z = 0.000001$ cm/s

Unweathered shale and fractured limestone with conductivities (in cm/s) as follows:

Unit	K_X	K_Z
5	0.0010	0.00000111
6	.0020	.00000125
7	.0030	.00000143
8	.0050	.00000200

Figure 6. Modeled distribution of hydraulic conductivity in the Delhi Pike landslide and the resulting pressure head at the slip surface, showing the effect of increased permeability in the unweathered bedrock (limestone), relative to figure 5. The hydraulic gradient at the left boundary, $\partial h/\partial x$, is –0.056, and the hydraulic gradient at the basal boundary, $\partial h/\partial z$, is 0.0529. Only pressure-head distribution for r=1 is shown, and it is almost identical to that shown in figure 5.

RESULTS OF THE GROUND-WATER MODEL

The cross section in figure 5 shows contours of equal head on a cross section through the Delhi Pike slope, and several inferences can be made from the contours about the pattern of flow. Seepage in the unweathered bedrock is roughly parallel to the mean slope, just as one would expect it to be near the middle of a long, uniform slope. A strong downward component of seepage exists in colluvium and weathered bedrock near the top of the cross section, probably because the ground surface has a slight backslope and the flow is recharging a weathered limestone bed. Ground water flows downward above the middle and left parts of weathered limestone layers. Water in the weathered limestone beds is subject to about 1.5 m of pressure head (that is, the head in any given layer is about 1.5 m greater than the elevation of the layer). Flow is nearly horizontal (toward the ground surface)

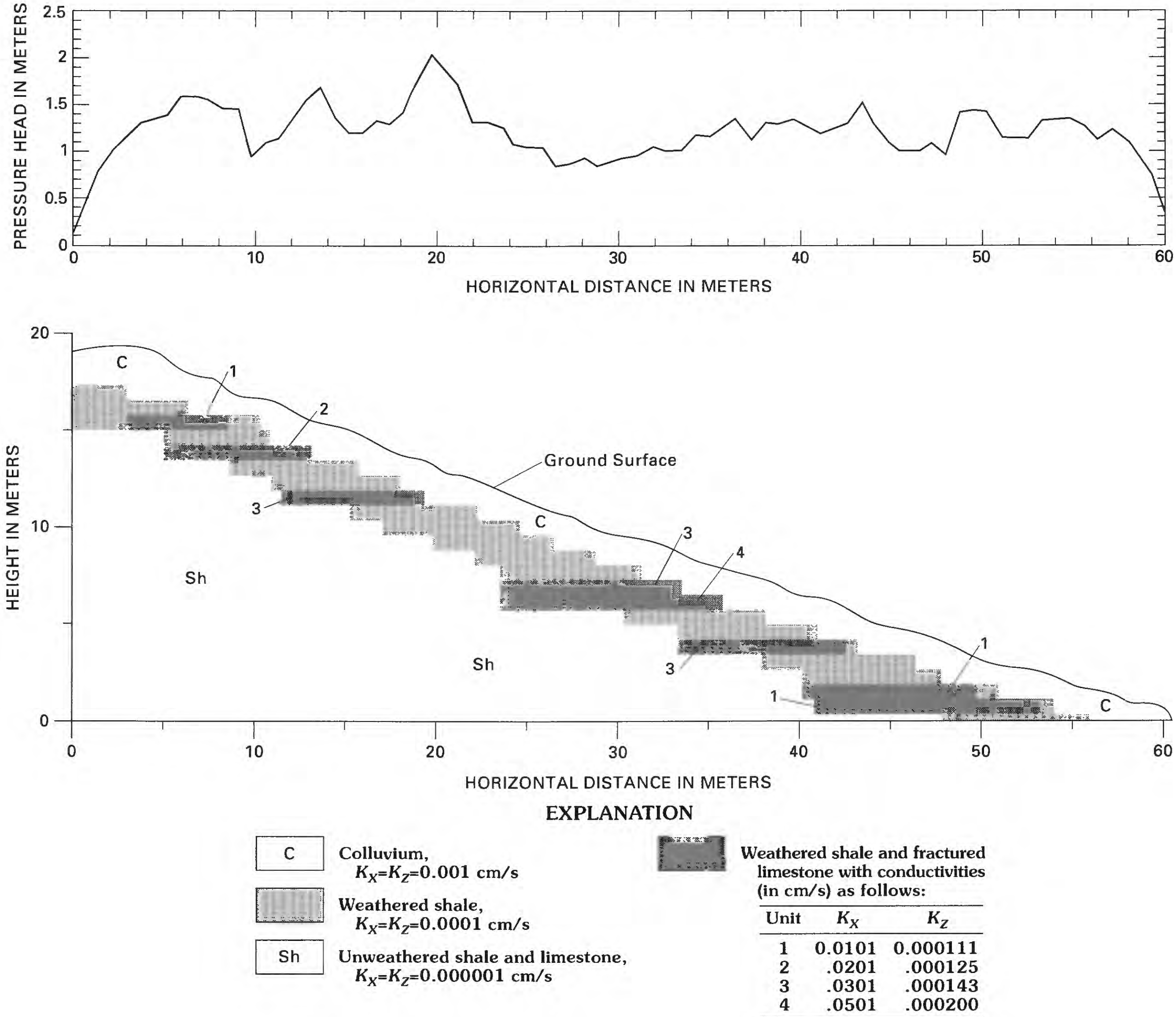


Figure 7. Modeled distribution of hydraulic conductivity in the Delhi Pike landslide and the resulting pressure head at the slip surface, showing the effect of decreased permeability in the weathered bedrock (shale), relative to figure 5. The hydraulic gradient at the left boundary, $\partial h/\partial x$, is zero, except in the weathered bedrock and colluvium, where it is –0.150, and the hydraulic gradient at the basal boundary, $\partial h/\partial z$, is 1.000, except in the weathered bedrock and colluvium, where it is 0.04606. Only pressure-head distribution for r=1 is shown, and it is smoother than that shown in figure 5, but relative maxima and minima occupy the same positions.

in the limestone beds. Flow of the artesian water from the right ends of the weathered limestone layers through the colluvium either is horizontal or has a slight upward component.

The graph at the top of figure 5 shows the distribution of pressure head at the slip surface for the seepage pattern shown in the cross section below. The pore pressure at the slip surface was determined by interpolating linearly between nodes in the model that were immediately above and below the slip surface. Pressure head is converted to pore pressure by multiplying it by the unit weight of water, 9.8 kN/m^3. Pressure head is relatively low at the ends and near the middle of the section, where seepage has a strong downward component. Pore pressures are locally elevated near the ends of weathered limestone beds (fig. 5), where flow in the colluvium is horizontal or has an upward component.

Changing the ratio of fluxes at the two boundaries, r, changes the pressure head at the slip surface in the upper one-fourth of the model. Figure 5 shows pressure head at the slip surface for r=1, 2, and 10. Pressure head to the left of the 17-m mark is slightly greater for r=2 than for r=1. The average pressure head is only 2 percent greater for r=2 than for r=1. The pressure head for r=10, however, is significantly greater than that for r=1 to the left of 17 m, and slightly greater between 17 and 27 m. The average pressure head is only 20 percent higher for r=10 than for r=1.

The most reasonable choice for the boundary conditions used for this model is to let r equal 1. If r=1, any water that flows out of the slope at the ground surface, near a limestone layer, soaks back into the ground surface a short distance downslope. Letting r be greater than 1 forces more water to flow out of the slope at the ground surface than can soak back into the ground elsewhere and, thus, results in surface runoff. However, surface runoff has not been observed at Delhi Pike, even during heavy rainstorms (A.Ö. Gökce, oral commun., 1982).

Changing the distribution of permeability can change the distribution of pore pressure at the slip surface. However, the pressure-head distributions shown in figures 5 and 6 are almost identical, even though the horizontal permeability of unweathered bedrock is increased four orders of magnitude in figure 6, while all other permeabilities remain equal. Decreasing the permeability of the weathered shale, while holding the permeability of other materials constant, causes the pressure head in the limestone to change, and the peaks and valleys in the distribution of pore pressure along the slip surface change in magnitude but maintain their positions (fig. 7). Of course, if the permeability of the weathered shale is significantly less than the permeability of the colluvium, then the flow will be roughly parallel to the top of the weathered bedrock, as in figure 4. Pressure head in the weathered limestone beds, computed from the model in figure 5, ranges from 1.1 to 3.6 m and averages 2.0–2.5 m, somewhat higher than but consistent with the pressure head of 1.5 m reported by Fleming and others (1981). Thus, the distribution of permeabilities shown in figure 5 gives plausible results, and the distribution of pressure head shown by the r=1 curve in figure 5 is my best estimate of the distribution of pressure head along the slip surface if the slope is fully saturated.

ROLE OF ARTESIAN WATER IN THE FAILURE OF COLLUVIUM AT DELHI PIKE

The distribution of pressure head from the model of ground-water flow (fig. 5) can be used to assess the role of ground water in the failure of the slope. Ground water can affect the overall factor of safety of a slope by reducing strength all along the potential failure surface. Furthermore, ground water can reduce strength preferentially in zones of high pore pressure (resulting from horizontal or upward seepage) along a potential slip surface. This preferential reduction of strength can influence the sequence of failure along a potential failure surface.

Artesian water from the limestone causes inconsequential reduction of the overall factor of safety; it is equivalent to the reduction of the overall factor of safety by theoretical slope-parallel flow. Ground-water flow in the slope (as modeled in fig. 5) has an effect on the overall factor of safety (as computed by the method of Janbu, 1973) approximately

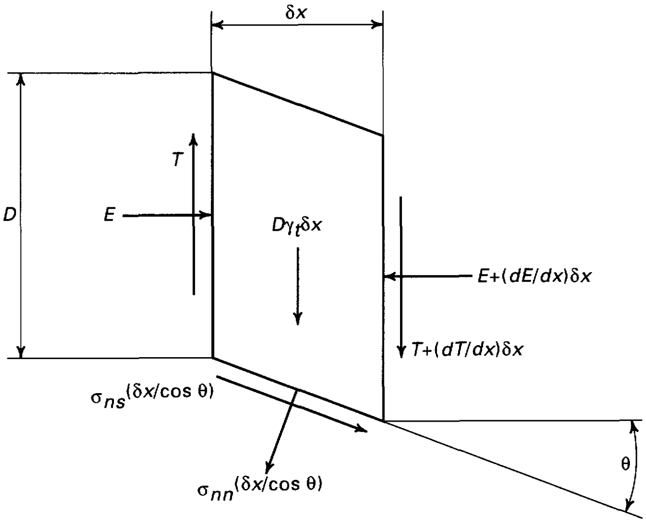


Figure 8. Free-body diagram showing forces acting on a slice of a landslide. The width of the slice is δx, the depth from the ground surface to the slip surface is D, and the weight (per unit width) is $D\gamma_t\delta x$. The local slope of the slip surface is determined by the angle θ. The normal and shear stresses acting on the slip surface are σ_{nn} and σ_{ns}, respectively. The horizontal (normal) resultant force acting on the surface of the slice is E, and the vertical (shear) resultant force acting on the surface of the slice is T.

equal to that of theoretical flow parallel to the mean slope, with the top flow line at the ground surface. Pore pressures due to slope-parallel flow are adequate to explain failure (table 1). The average of the modeled pore pressures is 98 percent of the average of the pore pressures due to slope-parallel flow.

High pore pressure associated with artesian water can also influence progressive failure of a slope. Among the factors that can affect local factors of safety, three will be analyzed here: local depth, local pore pressure, and local slope of the potential failure surface. If the depth and slope of the potential failure surface were uniform, then it would be straightforward to assess the role of high pore pressure in failure of the slope at Delhi Pike, because areas of the slip surface where pressure head is relatively high would coincide with the positions of low local factors of safety and, therefore, initial failures. The depth and slope of the slip surface enter the problem because both are nonuniform at Delhi Pike. Strength enters the problem only indirectly, through its variation with depth, because the cohesion and coefficient of friction are assumed constants along the failure surface.

The initial local factor of safety (that is, the local factor of safety before any deformation occurs) can be computed by neglecting variation in the horizontal and vertical forces acting in the soil (fig. 8):

$$\frac{dE}{dx} = 0, \text{ and } \frac{dT}{dx} = 0. \tag{4}$$

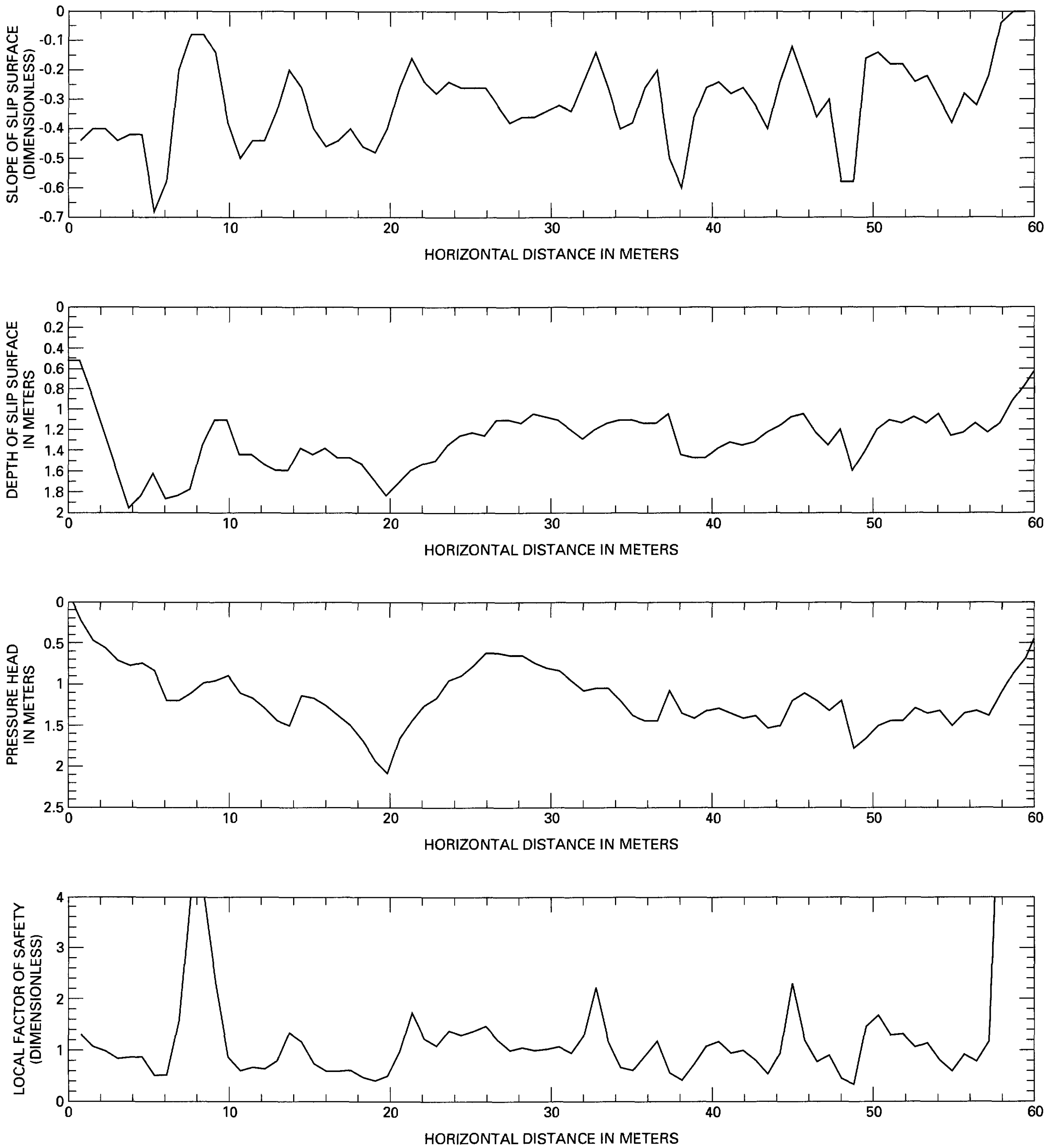


Figure 9. Local factor of safety, pressure head, slope of the slip surface, and depth to the slip surface at the Delhi Pike landslide, compared to distance from the toe of the trench. The top three graphs are inverted so that conditions that tend to decrease the factor of safety (specifically, high pore pressure, steep inclines of the slip surface, and deep parts of the slip surface) are represented as relative minima. Except for this inversion, the distribution of pressure head shown here is identical to the r=1 curve in figure 5. Minima in the local factor of safety coincide with the steepest parts of the slip surface.

In equation 4, E is the resultant normal force acting on a vertical plane passing through the landslide, and T is the resultant shear force acting on the same plane. Other forces that determine the vertical and horizontal equilibrium of a slice are the weight of the soil and the resultants of the normal and shear stresses acting on the slip surface. Thus, the forces considered in this analysis are the same as those in the ordinary method of slices (Lambe and Whitman, 1969, p. 364). However, the analysis differs from other methods of stability analysis in that a factor of safety is computed for each slice, rather than summing the driving and resisting forces of all the slices and computing a factor of safety for the entire landslide. The initial, local factor of safety, F', is determined by a formula identical to the formula for the overall factor of safety in an infinite slope, except that the slope angle, θ, the pore pressure, p, and the depth of the failure surface, D, vary along the failure surface,

$$F' = \frac{c' - p\tan\phi'}{D\gamma_t \sin\theta\cos\theta} + \frac{\tan\phi'}{\tan\theta} \tag{5}$$

In equation 5, c' is cohesion for effective stress, ϕ' is the friction angle of the soil for effective stress, and γ_t is the total (wet) unit weight of soil. Equation 5 shows that F' decreases as D, p, and θ increase.

The extent to which local thickness, slope, and pore pressure, combined, might have influenced the occurrence of initial, localized failures that led to progressive failure of the slope at Delhi Pike can be seen in figure 9. The local factor of safety (F'), slope of the slip surface (tan θ), depth to the failure surface (D), and pressure head (ψ), are graphed as functions of distance from the toe of the trench. Residual strengths were used in computing local factors of safety, but relative minima in the factor-of-safety graph represent the most likely sites of initial failures. The graphs of pressure head, slope, and depth are inverted in figure 9 so that conditions that tend to decrease the factor of safety (specifically, high pore pressure, steep inclines of the slip surface, and deep parts of the slip surface) are represented as relative minima in the graphs. Where relative minima in the graphs of pore pressure, slope, and depth coincide (as at 6, 19, 38, and 49 m, for instance), the response in the factor-of-safety graph is most pronounced. However, where the relative minima in these first three graphs are out of phase, their effects tend to ameliorate each other.

Local slope of the slip surface appears to have a stronger influence on the local factor of safety than pore pressure or thickness in the Delhi Pike landslide. Positions of both relative maxima and minima in the graphs of local factor of safety and local slope of the slip surface coincide almost exactly (fig. 9). However, positions of only a few of the local maxima and minima of pore pressure coincide with the maxima and minima of local factor of safety. The same is true of local thickness. Thus, whatever determined the slope of the failure surface also determined the position of the initial failures. Apparently, artesian water played only a minor role in determining the locations of initial failures in the slope.

CONCLUSION

Although artesian ground water is generally thought to be a major cause of landslides in colluvium overlying horizontally bedded sedimentary rocks, analysis of the Delhi Pike landslide indicates that artesian water probably plays a minor role in initial failure of thin colluvium in the Cincinnati area. Places may exist where artesian water plays a significant role, such as some deep-seated failures in thick colluvium, but data needed to document such cases are unavailable. For the models used here, average pore pressure in the colluvium is no higher than would be expected for the commonly assumed slope-parallel flow with the water table at the ground surface. Thus, the overall factor of safety is no lower than it would be if the weathered and unweathered bedrock were homogeneous. A few areas of locally high pore pressure coincide with local minima of the factor of safety and steep parts of the failure surface. Positions of other local maxima of pore pressure coincide with local maxima of the factor of safety along relatively flat parts of the slip surface. Thus, high pore pressure associated with the artesian water probably contributed slightly to initial failures at a few points on the slip surface, but it cannot have determined the locations of initial failures to the extent it would have if the depth and slope of the slip surface were constants.

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