Engineering Geology and Design of Slopes for Cretaceous Potomac Deposits in Fairfax County, Virginia, and Vicinity

GEOLOGICAL SURVEY BULLETIN 1556

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Engineering Geology and Design of Slopes for Cretaceous Potomac Deposits in Fairfax County, Virginia, and Vicinity

Engineering Geology of Potomac Formation Deposits in Fairfax County, Virginia, and Vicinity, with Emphasis on Landslides

By STEPHEN F. OBERMEIER

Design and Construction of Slopes in Potomac Formation Deposits

By PAUL G. SWANSON *and* JOHN S. JONES, JR.

Evaluation of Permanent Cut Slope Stability in Potomac Formation Deposits

By JAMES J. SCHNABEL

A Summation and Critique: A Review of Procedures for Design of Permanent Slopes on the Potomac Formation and Some Relevant Field Observations

By STEPHEN F. OBERMEIER

Edited by STEPHEN F. OBERMEIER

GEOLOGICAL SURVEY BULLETIN 1556

A collection of papers discussing the relations between the geologic setting and the design of slopes in the Potomac Formation in Fairfax County, Va., and vicinity

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PREFACE

Landslides are a commonplace and widespread problem in sediments of the Potomac Formation, which crops out in a narrow belt from southern Virginia to New Jersey. Although the belt is only a few miles wide, many cities and towns are located along the Fall Line where the Potomac Formation crops out. Serious land-use and development problems often result from the slope instability and the shrink-swell properties of the fine-grained Potomac sediments. The following papers address engineering geology and the design of slopes of the Potomac Formation in Fairfax County, Va., and vicinity (a part of the Washington, D.C., metropolitan area), but the principal conclusions are valid wherever Potomac sediments are present.

The Potomac Formation of Early Cretaceous age is clay rich and highly plastic in many places and is usually highly overconsolidated owing to the weight of formerly overlying formations, which have been eroded away. Engineering design of slopes is difficult and often somewhat subjective because of the lack of well-developed engineering analysis and design methods for highly plastic, highly overconsolidated materials. As a result, realistic engineering design must rely extensively on an empirical evaluation of previous experience with the Potomac sediments. The papers in this report are intended to present an overview of the nature of the problem and some different design methodologies that have been used.

The first paper (by Obermeier) discusses the geologic setting of the Potomac Formation, the physical and chemical properties of its sediments, and the nature of its slope stability problems. The second and third papers (by Swanson and Jones and by Schnabel) have been written by geotechnical engineers associated with private consulting firms that have accumulated much experience, especially in the Washington, D.C., metropolitan area. These authors were invited to submit papers about slope design because they have design philosophies that differ somewhat from one another and thus offer a comprehensive perspective on the problems presented by the Potomac Formation. Inclusion of their papers does not represent an endorsement of their designs by the U.S. Geological Survey but is intended only to disseminate information and to provide a forum for discussion.

The scopes of the two papers by the geotechnical engineers are somewhat different. Swanson and Jones discuss stability of cut, fill, and natural slopes for both the short term (during construction) and the long term. Schnabel concentrates on design for the long-term stability of cut slopes in clay-rich sediments.

The final paper (by Obermeier) is a critique and summary of the design engineers' papers and also points out some research needs.

These papers are intended mainly for engineering geologists and geotechnical engineers. Some redundancies are inevitable because each paper is intended to be intelligible individually, except for the final critique and summary.

Technical terms such as "compactness," "consolidation," and others are used in the engineering sense in all four papers. "Soil" is used interchangeably with "sediment" because Potomac sediments behave as soils in the geotechnical sense.

English units, followed by metric units in parentheses, are used throughout the text and in the tables. Conversion factors from English to metric units and semiquantitative terms for consistency and compactness follow the preface. Terms such as "soft," "stiff," "hard," "compact," and "loose" are used in accordance with their semiquantitative definitions.

SYMBOLS, TERMS, AND DEFINITIONS

- c' Cohesion intercept of Mohr-Coulomb strength envelope, in terms of effective stress
- Δ Shear displacement
- ΔU_s Shear-generated pore pressure
- τ Shear stress
- τ_r' Residual effective shear stress applied to specimen in direct shear apparatus
- σ Normal stress
- σ' Normal effective stress
- σ_n' Normal effective stress applied to specimen in direct shear apparatus
- ϕ_r' Residual friction angle, defined by the residual coefficient of friction $\left(\frac{\text{change of } \tau_r'}{\text{unit change of } \sigma_n'}\right)$
- K_0 Ratio of horizontal to vertical effective normal stress
- U Unconfined compression shear strength
- UU Unconsolidated undrained shear strength, measured in triaxial test apparatus
- FS Factor of safety $= \frac{\text{resisting forces}}{\text{driving forces}} = \left(\frac{\text{cohesion+friction forces}}{\text{driving forces}}\right)$

CONVERSION FACTORS

Metric unit	English unit
1 m	3.28 ft
1 cm	.39 in.
1 m/s	3.28 ft/s
1 kilogram force (kgf)	2.20 lb
1 kilogram force per square centimeter (kgf/cm^2)	2048 lb/ft^2
1 kilogram force per cubic meter (kgf/m^3)	.062 lb/ft^3

QUALITATIVE AND QUANTITATIVE EXPRESSIONS OF CONSISTENCY OF FINE-GRAINED SOILS AND COMPACTNESS OF COARSE-GRAINED SOILS

Consistency	Field description	Unconfined compressive strength, in kgf/cm^2	Standard Penetration Test resistance, in blows/ft
Fine-grained soils			
Very soft	Easily penetrated several inches by fist.	<1.0	<2
Soft	Easily penetrated several inches by thumb.	1.0–2.0	2–4
Medium	Can be penetrated several inches by thumb with moderate effort.	2.0–4.0	4–8
Stiff	Readily indented by thumb but penetrated only with great effort.	4.0–8.0	8–15
Very stiff	Readily indented by thumbnail.	8.0–16.0	15–30
Hard	Indented with difficulty by thumbnail.	>16.0	>30

Compactness	Standard Penetration Test resistance, in blows/ft
Coarse-grained soils	
Very loose	<4
Loose	4–10
Medium compact	10–30
Compact	30–50
Very compact	>50

ENGINEERING GEOLOGY AND DESIGN OF SLOPES FOR CRETACEOUS POTOMAC DEPOSITS IN FAIRFAX COUNTY VIRGINIA, AND VICINITY

ENGINEERING GEOLOGY OF POTOMAC FORMATION DEPOSITS IN FAIRFAX COUNTY, VIRGINIA, AND VICINITY, WITH EMPHASIS ON LANDSLIDES

By STEPHEN F. OBERMEIER

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ABSTRACT

Potomac Formation deposits crop out or are near the ground surface in a belt that includes parts of large metropolitan areas such as Washington, D.C., and Baltimore, Md. Both naturally occurring and construction-related landslides are commonplace and widespread in this belt and cause serious land-use problems. Landslides often take place many years after slopes are excavated in clay-rich sediments of the Potomac Formation, and the results can be disastrous to unsuspecting home owners.

The Potomac Formation has two regional facies. In one, the clay minerals contain much montmorillonite; in the other, the clay minerals are dominantly kaolinite and illite. Unweathered clay-rich sediments of the Potomac Formation are typically highly overconsolidated and stiff to hard in consistency. The consolidation was imparted by the weight of younger deposits that have since been eroded away. Within the uppermost 20 ft (6 m) of the Potomac Formation, however, the sediments are generally weakened by physical and chemical weathering. Fractures, joints, and parting planes are also preferentially weakened along the breaks within this zone of weathering.

Landslides generally take place in the zone of weathering. Engineering design of slopes in the clay-rich sediments is difficult and is often very subjective, especially where the Potomac Formation sediments are highly plastic. As a result, realistic engineering design must rely mainly on empirical evaluation of previous experience with the Potomac Formation.

Relevant geologic factors that affect slope stability include the geologic history of Potomac Formation sediments, the nature of sedimentation, the properties of clays in the montmorillonite and kaolinite-illite facies, the properties of sheared zones, joints, surface weathering fractures and bedding-plane partings, and the ground-water setting.

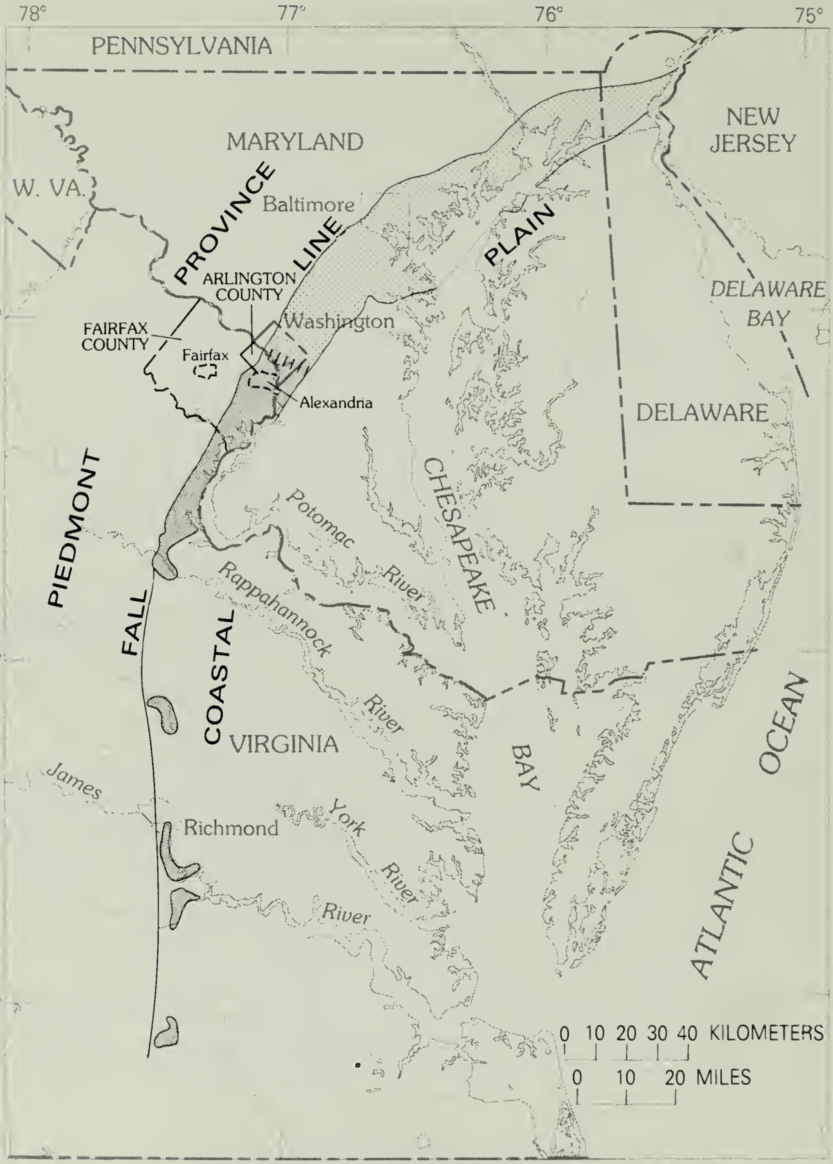
ACKNOWLEDGMENTS

This paper is based largely on information collected by the author in preparation for a presentation at the 1977 American Society of Civil Engineers, National Capital Section, Geotechnical Engineering Seminar, Washington, D.C. The following people kindly consented to discuss their experiences and make their data available: James D. Hainley, Ackenheil & Associates, Baltimore, Md.; Carl G. Bock and Robert N. Evans, Bechtel Associates, Coral Hill, Md.; E. O. Gooch, E. O. Gooch & Associates, Charlottesville, Va.; Joseph B. Klein III, Law Engineering Testing Co., McLean, Va.; James J. Schnabel, Schnabel Engineering Associates, Bethesda, Md.; Ronald E. Smith, Woodward-Clyde Consultants, Rockville, Md.; Clarence S. Coleman, William C. Smith, and Stuart T. Terrett, County of Fairfax, Va.; John D. Glaser, Maryland Geological Survey, Baltimore, Md.; Ernest G. Gray, Maryland State Highway Commission, Baltimore, Md.; and Richard D. McConnell, Veterans' Administration, Washington, D.C.

INTRODUCTION

Both naturally occurring and construction-related landslides are commonplace and widespread in the belt of Lower Cretaceous Potomac Formation[1] deposits extending from Virginia to New Jersey (fig. 1). The landslides usually occur in generally strong, massive-appearing silt and clay and in sand-clay strata. Slopes underlain by

[1]The Potomac Formation is raised in rank to Potomac Group and divided into three formations in areas of Maryland and in Washington, D.C. In Virginia, it is considered to be of formation rank only.



EXPLANATION

Potomac Formation outcrop belt

Predominantly kaolinite and illite (northern clay facies)

Predominantly montmorillonite (southern clay facies)

Transition between montmorillonite facies and kaolinite-illite facies

FIGURE 1.—Potomac Formation outcrop belt from Virginia to New Jersey (after Force and Moncure, 1978).

the Potomac Formation are a classical example of areas where slope stability cannot be evaluated indiscriminately by conventional design techniques. Conventional slope stability analysis methods using peak shear strength parameters often result in factors of safety that are much too high, on the basis of field experience. Residual shear strength parameters are appropriate for some geologic settings but too conservative for others. A major problem is selection of strength parameters that will ensure a safe yet economical design.

In order to document the mechanisms that cause the landslides and to determine the factors involved in selecting strength parameters, this author met with engineers and geologists from numerous consulting firms and government agencies in the greater Washington, D.C.–Baltimore, Md., area. He also studied about 20 landslide case histories documented by consulting engineers. Field investigations were then undertaken to evaluate additional factors that could contribute to the landslides. These investigations were made primarily in outcrops of the Potomac Formation in the Coastal Plain of Fairfax County northward to Arlington County, Va., and the city of Alexandria, Va., and in Washington, D.C. (fig. 2). This area is the "Fairfax County, Virginia, and vicinity" of the title. These studies were mostly restricted to the montmorillonite clay facies shown in figure 1, but limited investigations were made in the kaolinite-illite clay facies as far north as the northern limits of the Chesapeake Bay.

This report summarizes the results of the meetings with engineering practitioners, studies of landslide case histories, and the author's own field investigations. Specific recommendations for slope design are generally not made in this paper; rather, an attempt is made to identify relevant factors that cause the landslides. Although the report is applicable specifically to the greater Fairfax County area, most comments are thought to be valid for Potomac sediments throughout the region shown in figure 1.

GENERAL SETTING

The portion of the Atlantic Coastal Plain physiographic province shown in figure 1 is bounded by the Piedmont province on the west and the Continental Shelf on the east (not shown in fig. 1). Much of the Coastal Plain is separated from the Piedmont province by the Fall Line (or Fall Line Zone), the topographic feature where major streams rapidly descend from the crystalline uplands into tidal rivers, estuaries, and embayments. The Coastal Plain is a wedge of continental and marine sediments that thicken from a few feet on the western boundary to many thousands of feet at the coastline of the Atlantic Ocean. Potomac Formation deposits of Early Cretaceous age are the basal sediments throughout much of the Atlantic Coastal Plain.

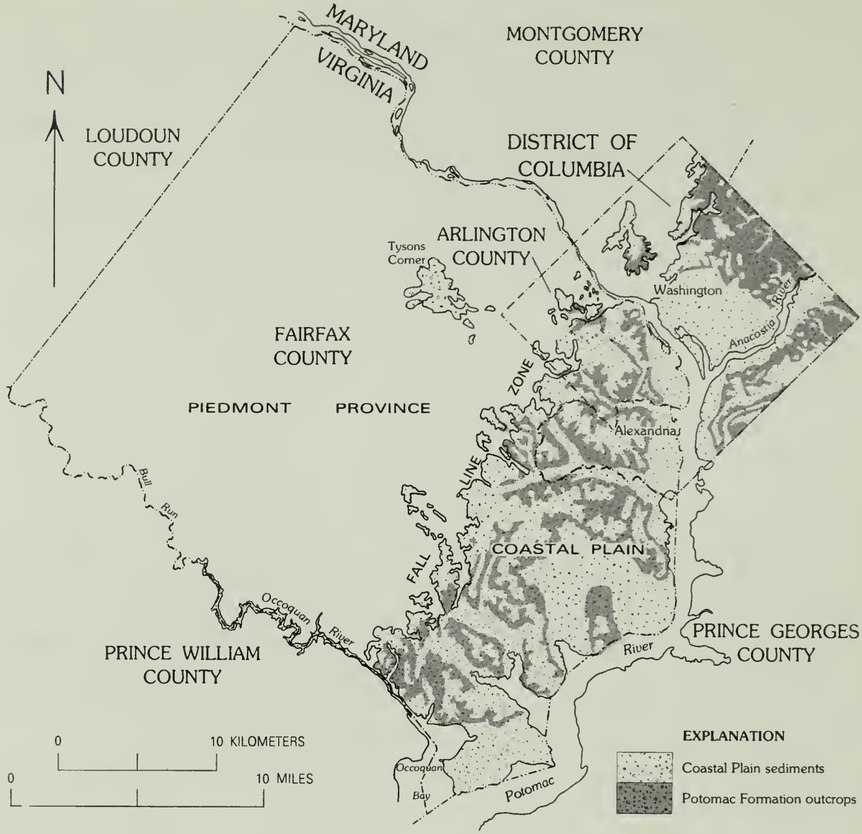


FIGURE 2.—Coastal Plain deposits in the principal study area.

Potomac sediments are commonly overlain in part by Upper Cretaceous and marine strata, but, in some places, they are overlain by Tertiary and Quaternary deposits and are separated from them by major unconformities. Principal geologic events and geologic units in Fairfax County and vicinity are shown in table 1.

The length of the Potomac outcrop belt in figure 1 exceeds 200 mi (320 km), but the width rarely exceeds 15 mi (25 km). Erosional outliers too small to be shown on the map occur along the western edge of the belt and south of Fredericksburg, Va. The maximum thickness of the Potomac Formation in the vicinity of the outcrop belt is perhaps 1,200 ft (370 m).

In Maryland and in Washington, D.C., the Lower Cretaceous strata are subdivided into three nonmarine formations that comprise the Potomac Group: (1) the basal Patuxent Formation, (2) the lenticular middle unit, the Arundel Clay, and (3) the upper unit, the Patapsco Formation. These formations are not separable by field mapping

TABLE 1.—*Geologic time chart and relation to major units of the Coastal Plain of Fairfax County, Va., and vicinity*

Chronostratigraphic units		Absolute age, in years	Geologic units	Map symbol (fig. 3)	Origin	Range of thickness, in feet
Quaternary system	Holocene		Alluvium, dominantly sand, silt, or clay, highly variable.	Q	Nonmarine (fluviatile).	0–20 (avg., 10)
		10,000	Unconformity			
	Pleistocene		Terrace deposits, various levels, dominantly gravel or sand in silt or clay matrix.	Q		0–150 (avg., 35)
		2,000,000	Unconformity			
Tertiary system	Pliocene		Upland gravel deposits, various levels, dominantly sand and gravel.	Tug		0–50 (avg., 35)
	Miocene		Unconformity			
	Oligocene		Dominantly clayey and silty sand deposits of great lateral extent, separated by multiple unconformities.	--	Marine	0–400 (avg., 200)
	Eocene					
	Paleocene					
		63,000,000				
Cretaceous system	Upper					
			Unconformity			
	Lower		Potomac Formation, dominantly sand, silt, or clay layers, highly variable.	Kp	Nonmarine (fluviatile and deltaic?).	0–1,200 (avg., 500)
		138,000,000	Unconformity			
Precambrian			Crystalline rocks		Metamorphic and igneous.	

south of the Potomac River (Drake and others, 1979), and the sequence is called the Potomac Formation. In this report, the Potomac sequence is considered as the Potomac Formation, irrespective of geographic locale.

The Potomac deposits have at least two regional sedimentary facies that show no sharp lateral demarcation; one is orthoquartzite sand interbedded with clay-rich beds of kaolinite and illite north of the Potomac River (the kaolinite-illite facies), and the other is arkosic sand interbedded with montmorillonite-rich clay layers (the montmorillonite facies) south of the river. Glaser (1969) has hypothesized that these facies result from different source provenances, and Force and Moncure (1978) have presented supporting data. Irrespective of the facies, however, the upper part of the Potomac sequence generally has more clay than the lower part.

GEOLOGIC HISTORY

Potomac Formation sediments were deposited on subsiding crystalline rocks by freshwater rivers meandering along the inner margin of the Coastal Plain and are characterized by crossbedding, channels filled with clays or sands, rapid pinching and thickening of beds, and lateral and vertical size grading. Continued subsidence and tilting of the region resulted in a gentle eastward dipping of the sediments. Possibly, some of the deposits are deltaic in character, formed where rivers debouched into brackish or marine water. Small- and large-scale erosion surfaces caused by ancient meandering rivers are common within this group of sediments.

Potomac Formation deposits were intermittently eroded and were overlapped by primarily fine grained sediments, which were deposited in an oscillating marine environment during Late Cretaceous and Tertiary times. A major regional unconformity, during which hundreds of feet of Potomac strata were eroded away in many places, separates the Potomac Formation from the overlying marine deposits. These marine deposits are presently preserved east of the Potomac River (fig. 3). In many places, a gravel layer a few feet thick is present immediately above this unconformity. Above the basal gravel layer, the marine sands and clays tend to be in thin beds, commonly of great lateral extent.

The seas next withdrew and subjected the area to subaerial erosion. Fluvial sands and gravels that were spread over the eroded surface now unconformably overlie the marine strata and the Potomac sequence and locally extend to the Piedmont rocks. During the late Tertiary, an ancestral Potomac River system (Hack, 1955; Schlee, 1957) apparently deposited broad alluvial aprons of sand and quartz

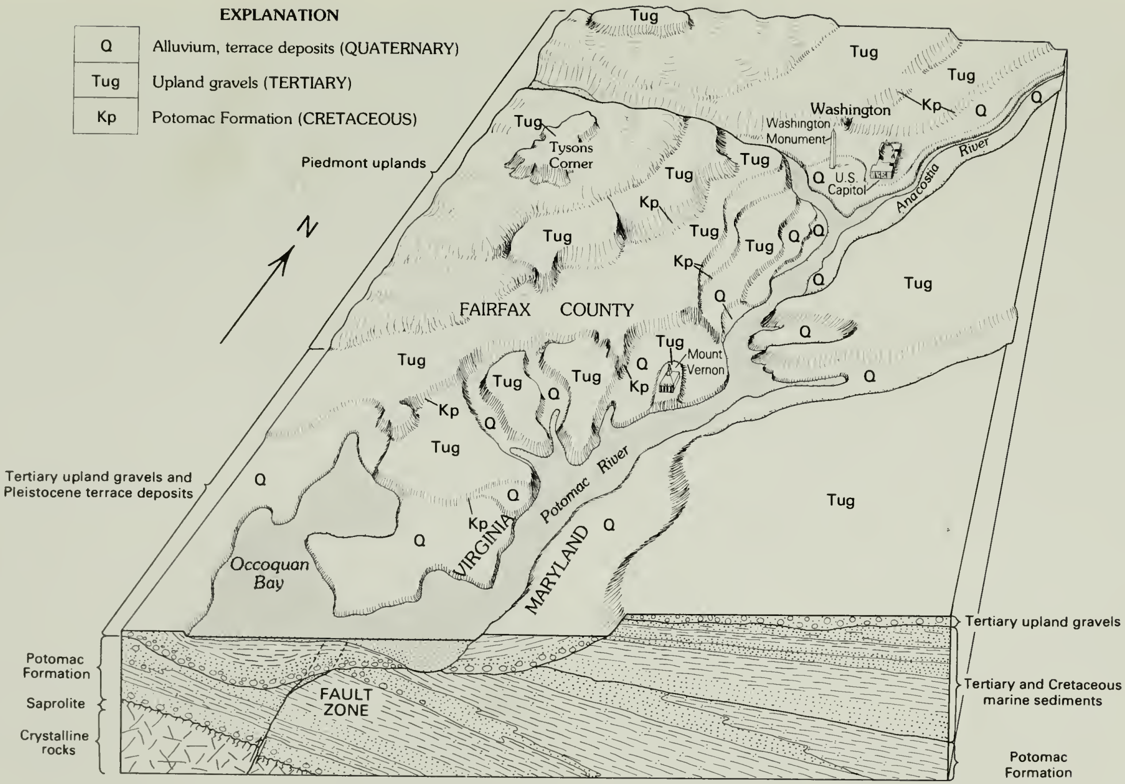


FIGURE 3.—Geologic-topographic setting in Fairfax County, Va., and vicinity.

gravel at various steplike levels across low relief plains. These deposits are commonly referred to as the "upland gravels."

In Fairfax County, the upland gravels are believed to be a succession of at least four terraces (Force, 1975; Drake and others, 1979) that are 0.5 to 2 mi (0.8–3.2 km) wide and locally up to 60 ft (18 m) thick (average about 20 ft (6 m)). The terraces descend stepwise to the tidal Potomac River. The upland gravel terraces have been dissected extensively during periods of erosion as the streams cut to lower levels. The older deposits are found only as caps of isolated hilltops. The relationship of terraces and other geologic units to topography in the area along the Potomac River immediately south of the Nation's Capital is shown in figure 3.

During the Pleistocene, continental glaciers did not reach Washington, D.C., but glaciation had a profound effect on erosion and deposition in the area. Sea level was hundreds of feet lower during times of maximum glaciation than it is at present and caused major downcutting of Coastal Plain valleys (Hack, 1957). During recession of the glaciers, stream flow increased, sea level rose, and comparatively coarse grained materials were deposited. As a warming trend continued, valleys were inundated and fine-grained sediments laid down. As a consequence of the multiple glacial advances and retreats, a series of Pleistocene river terrace deposits was left behind. The terraces are a mixture of silty and sandy clay and sand and gravelly sand, complexly interlayered and lensed. Most of downtown Washington, D.C., is underlain by these terrace deposits, which have an average thickness on the order of 30 ft (9 m), although, locally, they may exceed 60 ft (18 m). In the ancient channels of the Potomac River in Fairfax County, the deposits are as much as 160 ft (50 m) thick.

During the last 5,000 or 6,000 years, sea level has risen 25 to 35 ft (8–11 m) with respect to land along the northern Atlantic coast. This rise submerged the Potomac and Anacostia Rivers to the Fall Line and deposited fine-grained alluvium in river channels.

Clay, silt, and clay-bearing sand of the Potomac Formation have been consolidated to a generally stiff to hard consistency, principally by the weight of sediments subsequently stripped away. In many places near the ground surface and along the unconformity separating the Potomac Formation from younger sediments, however, intense chemical and physical weathering have greatly softened the Potomac sediments. Numerous joints and fractures are present in many localities, especially in near-surface deposits of weathered fine-grained sediments.

Post-Cretaceous parallel faults at the Fall Line and through the Potomac Formation extend at least from the southern part of Fairfax County to Fredericksburg, Va. (fig. 1) (Mixon and Newell, 1977).

These faults are within the Stafford fault system, a zone at least thousands of feet wide. Within the fault system, the faults having the largest offsets trend northeast, but smaller faults are present at various strikes (Prowell, 1983) and at unpredictable locations. The major zones of shearing are commonly steeply dipping reverse faults thought to have a strike-slip component (D. C. Prowell, unpub. data). Cumulative vertical displacement exceeds 300 ft (90 m) near Fredericksburg (Mixon and Newell, 1977). Other fault systems in the Coastal Plain have been documented about 20 mi (33 km) east of Fairfax County (Jacobeen, 1972) and in Washington, D.C. (Darton, 1951).

In Fairfax County, several minor faults and associated swarms of fractures near the faults have been documented (Langer and Obermeier, 1978; Froelich, 1978) in the Potomac Formation deposits. In most places in Fairfax County, the fault displacements are probably less than 1 to 2 ft (0.3–0.6 m). Figure 4 shows one of these small faults exposed in a stream valley. The orientation and mode of occurrence of these faults strongly suggest that they are associated with the tectonic activity south of the county. Many geologists suspect that this faulting extends north of Washington, D.C., possibly through the Potomac Formation, and the author has observed fractures thought to be faults through the Potomac Formation near the northern end of Chesapeake Bay.

FIGURE 4.—Fault cutting Potomac Formation clay in Alexandria, Va. Fault passes along line indicated.

LANDSLIDE CHARACTERISTICS

Some typical landslide types are shown in figure 5. Slumps and planar glide blocks are predominant in occurrence and economic significance. Minor sliding, usually of small economic significance, is also caused by sloughing of very steep exposures of clay and silt on slopes exceeding about 50 percent, although sloughs can occur on slopes as low as 30 percent. Many slopes showing no evidence of landslides display evidence of creep in the form of trees bent at the base.

Slumps (fig. 5*A*) are the most common type of landslide and are normally about 50 to 200 ft (16–63 m) from crown to toe, although saturated clayey soil at the toe can be remolded to the consistency of a viscous fluid and flow hundreds of feet. Slumping is commonly retrogressive (that is, new portions of the surface of rupture form progressively upslope in previously unfailed soils). Some natural landslides have pressure-induced ridges and hummocks near their toes. The length of the slide along the slope generally equals or exceeds the length of rupture down the slope. The maximum depth of the surface of rupture ranges from about 15 to 20 ft (4.6–6 m) for most slumps, the deepest recorded being about 40 ft (12 m).

Local depositional characteristics and physical properties of the Potomac Formation are very important with respect to locations of sliding. Ancient channels filled with sand provide conduits for subsurface water migration. Relative positions of coarser and finer grained materials can serve to improve or reduce stability. These stratigraphic details, together with the severity of weathering and the presence of structural features (such as sheared zones, ancient landslide slip surfaces, and partings and fractures of different types), control the types and locations of the slope failures. These landslides can occur on natural or construction-altered slopes, either during construction or many years later.

Figure 6 shows the relationship of geologic-hydrologic features to weakened zones commonly critical to landsliding. For many slumps, the surface of rupture dominantly traverses soils greatly softened by near-surface weathering and cut with fractures or goes in part along a particularly weak, weathered stratum or bedding-plane parting or a fracture. The crown of the slump also heads in a very permeable sand and gravel cap in many places. The unconformity separating the Potomac Formation from younger sediments is a locus of concentration for both slumps and planar glide blocks, especially where the Potomac Formation is overlain by water-bearing, relatively permeable sand-and-gravel-filled channels. Planar glide blocks (fig. 5*B*) develop most often where the channel dips in the direction of the slope. Planar glide blocks are often not recognized in the field, possi-

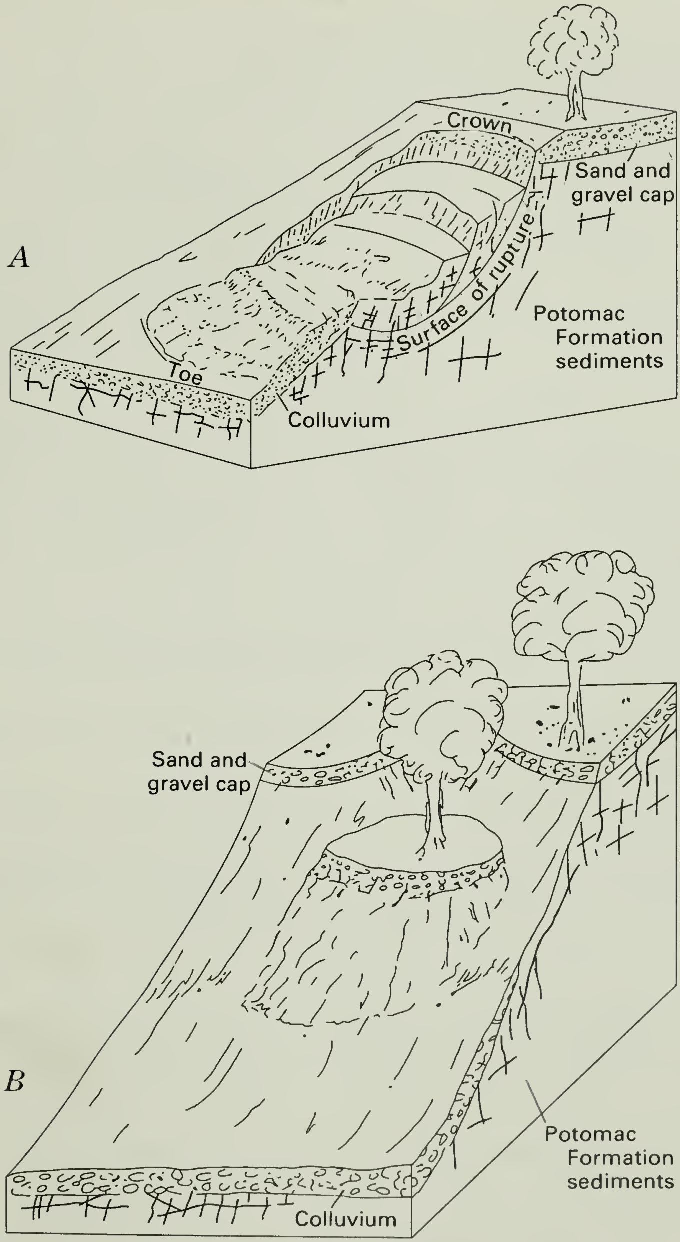


FIGURE 5.—Typical landslide types in Potomac Formation sediments. *A*, Slump. *B*, Planar glide block.

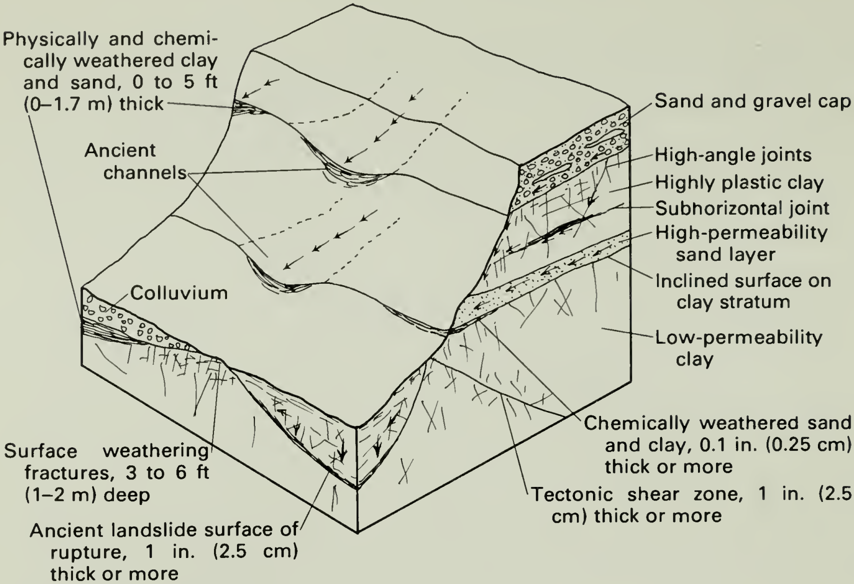


FIGURE 6.—Relationship of weak zones to local geologic-hydrologic features. Arrows indicate locations of concentrated water flow.

bly because soils at the toe have a tendency to flow when they are remolded.

Slides are rather commonplace on natural terrain underlain by either massive clay or by interbedded sand and clay where long slopes exceed 15 percent, but slides can also begin on flatter slopes. Field mapping by the author has shown that landslides are so common on the steep slopes that they must be a major factor in landform development on a geologic time scale (periods on the order of thousands of years).

Catastrophic movements of topples are encountered in construction excavations having steep walls. Opening of relict joints is thought to have been responsible for a major construction disaster that killed five men in Greenbelt, Md., a suburb of Washington, D.C. (Withington, 1964). A schematic cross section showing the setting of the accident is given in figure 7. According to Withington, relict joints in the clay opened slightly during excavation because of dessication and lack of lateral support. Surface water probably flowed into the joints and toppled an unsupported clay slab about 40 ft (12 m) long, 15 ft (4.6 m) high, and 1 ft (0.3 m) thick.

Construction-period problems also have been reported for trenches excavated in uncemented sands (Cleaves, 1968). Where flowing water is present, uncemented sands are susceptible to caving in and also

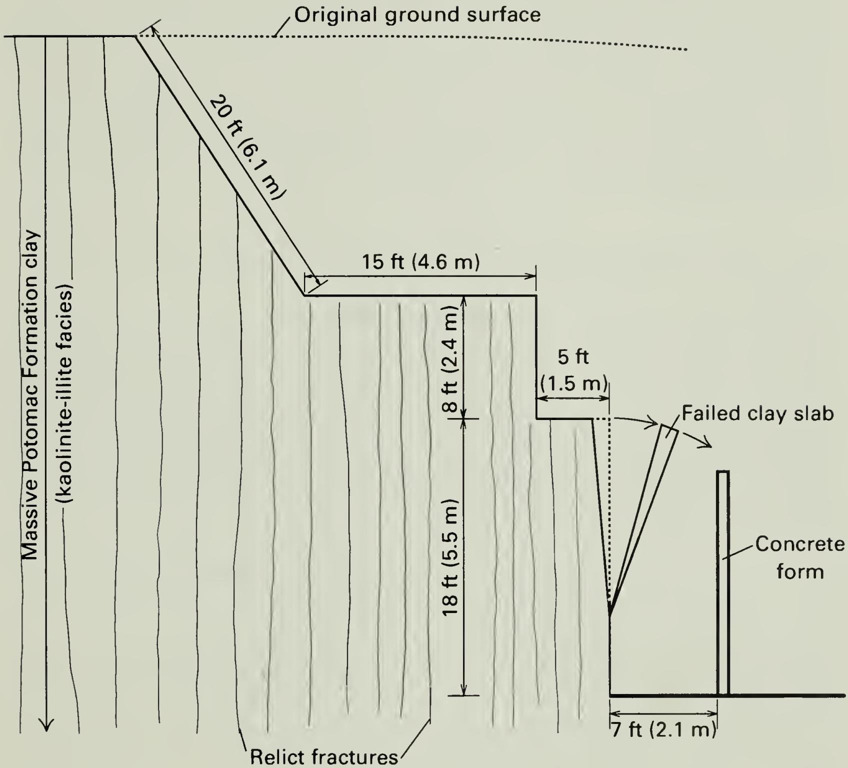


FIGURE 7.—Schematic cross section of excavation at Greenbelt, Md., showing part of a wall that failed (after Withington, 1964).

have the potential for undermining clay beds. This problem is most common in very weakly cemented, clean sands associated with the kaolinite-illite facies and, to a much lesser extent, in sands associated with the montmorillonite facies, because these sands are commonly bound with clay.

Landslide initiation is nearly always closely associated with ground-water-related mechanical forces. The water may be in the more permeable sand-rich strata, under confined conditions, or in joints or near-surface fractures. The slides normally begin (or at least move enough to be easily noticed) after a prolonged rainy season or after an intense rainstorm, which commonly causes increased pore-water pressure. Years may elapse before movement starts again in response to another prolonged rainy season or intense rainstorm, but the next small rain may be sufficient to reactivate sliding. Casual observations by many investigators suggest that the rate of movement is highly variable during the early stages of development; observed scarps of about 0.5 in. (1 cm) to 3 ft (1 m) or more can develop at the crown within a day. The rate of movement tends to increase as a landslide develops until the slide finally moves rapidly (up to 10 ft/d) and returns to equilibrium. Once a landslide has begun to move, corrective measures are often undertaken as soon as practicable because, at many places, the slides fail progressively and continue to enlarge upslope.

OVERVIEW OF SOIL ENGINEERING PROPERTIES

The generalizations on engineering properties that follow apply to the montmorillonite facies of the greater Fairfax County area and are based on hundreds of borings and many laboratory test data. Properties described are penetration resistance, plasticity, strength, and consolidation characteristics for both the unweathered and the weathered condition, swelling properties, and the regional significance of the data.

PENETRATION RESISTANCE, PLASTICITY, STRENGTH, AND CONSOLIDATION PROPERTIES

Standard Penetration Test (American Society for Testing and Materials, 1978) blow counts on unweathered clay typically vary between 30 and 100 blows/ft. On weathered clay, the blow counts generally range between 10 and 35, although lower values are occasionally observed. The blow counts for unweathered arkosic sand typically range between 30 and 60, but counts are higher at many places and can exceed 100. Where the sand is weathered, within about the upper 10 ft (3 m), the blow counts commonly range between 10 and 40; scattered lower values also occur.

The liquid limits of the clays in the thick massive clay bodies average about 60 to 70 but can exceed 100, and the plastic limits average in the high 20's and low 30's. At many places in these thick bodies, the clays are classified as CH.[2] Some unweathered clay samples do not break down easily during remolding for Atterberg limit testing but form very small, gritty clasts. As remolding continues, the clasts are further broken and the clay fraction increased; thus, the liquid limit, the plastic limit, and the plasticity index increase.

The natural water content is normally less than the plastic limit at depths greater than 20 to 30 ft (6–9 m) beneath the surface where Potomac clays crop out. At shallower depths, the natural water con-

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[2]Capitalized letters refer to the Unified Soil Classification System (Wagner, 1957).

tent tends to be higher and often exceeds the plastic limit by as much as 10 percent. The relation between natural water content and plastic limit is rather erratic as a function of depth at many places, however, even in the same borehole. This erratic variation probably results from the multiple water entries in sand lenses, joints, fractures, and sheared zones.

Weathering effects are normally most intense on hilltops, as the higher water contents of intact clays and the oxidization of feldspathic sands indicate. On slopes, slumping and erosion have apparently removed much of the intensely weathered soil.

Consolidometer tests on unweathered samples commonly have preconsolidation values 20 to 40 kips per square foot (ksf) (10–20 kfg/cm²) in excess of existing overburden, even on hilltops. Preconsolidation values tend to increase with depth by an amount approximately equal to the increased overburden stress. Therefore, it appears that the high preconsolidation has been imparted primarily by the weight of overlying sediments, which were later removed by erosion. Weathered and softened samples commonly have much lower preconsolidation values; in many places, near-horizontal thin zones along softened fractures have preconsolidation values less than or equal to the present overburden stress.

For intact unweathered samples, the unconfined compression (*U*) and unconsolidated undrained (*UU*) shear strengths of clay-rich sediments commonly range between 4 and 6 ksf (2–3 kgf/cm²). Shear strengths of unweathered clay-rich sediments are commonly reduced by the presence of fractures and slickensides. Peak *UU* strengths on fractured and slickensided samples are very erratic and average about one-half the values for intact samples. Axial strains at peak strengths on the unweathered samples generally range between 2 and 8 percent.

The undrained shear strength of intact weathered clay averages about 2 ksf (1 kgf/cm²) near the ground surface and along the unconformity at the top of the Potomac Formation. In many localities, however, the soil is weakened only a little along the unconformity. The natural water content of the softer soils is significantly higher than that of soils in the unweathered condition.

Clays and clay-cemented sands above a landslide surface of rupture typically have peak undrained shear strengths between 0.5 and 2.0 ksf (0.25–1.0 kgf/cm²), but, occasionally, they are so hard that they cannot be sampled with thin-walled tubes. Landslide materials are usually highly fractured and broken or remolded by the movement.

For normally consolidated, highly plastic clays, the effective friction angle of the peak shear strength in drained direct shear tests averages about 25° and increases to about 35° for the clayey sands. The residual drained direct shear strength of unweathered soils is much lower than the peak shear strength, as figure 8 shows; the ratio of residual strength to peak strength is commonly quite large for overburden stresses equivalent to about 10 to 20 ft (3–6 m) of soil. Angles of 12° to 15° are common for clays, and residual friction angles as low as 6° have been reported for highly plastic clays. Figure 8 also shows that there can be a significant difference between peak and residual shear strengths of intensely weathered and softened clay.

SHRINK-SWELL PROPERTIES

The highly plastic montmorillonite clay has a high potential for shrinking and swelling and can cause problems for houses and roads wherever it occurs. Large vertical movements of a house foundation can be caused by alternate wetting and drying of clay beneath the foundation. Some houses have been destroyed, and economic losses caused by this problem in Fairfax County may exceed those caused by landslides. To reduce these destructive effects (Dallaire, 1976), Fairfax County has enacted ordinances requiring that foundations on highly plastic clays be at least 4 ft (1.2 m) beneath final exterior grade, that foundations be drained, and that trees be kept away from foundations.

Other swelling-related problems include collapse of basement walls where highly plastic clays have been used as backfill material. Damage to roads occurs in the form of distorted pavements and curbs and weakened subgrades.

Swelling is also a factor in causing landsliding in that it must be an effective mechanism for the destruction of diagenetic bonds and the consequential weakening of the soil, in the sense discussed by Bjerrum (1967). It seems likely that diagenetic bonds are broken to depths of at least 10 to 20 ft (3–6 m) in the highly weathered plastic clays, where the natural water content approaches or exceeds the plastic limit. Within a few years, at construction sites where water can enter into permeable sand or fractures in unweathered deposits, the bonds in clay may be largely destroyed in a thin, swollen zone adjacent to the path of flowing water. The softening and swelling produce stresses in surrounding materials. These stresses, of course, lead to further softening and weakening along the fractures and more fracturing. After many years, at places where creep and slope movements are very small, the shear strength is probably further reduced over a large area to a value approaching that retained by consolidation under existing vertical stresses. Where there is much creep, the strength may be much lower.

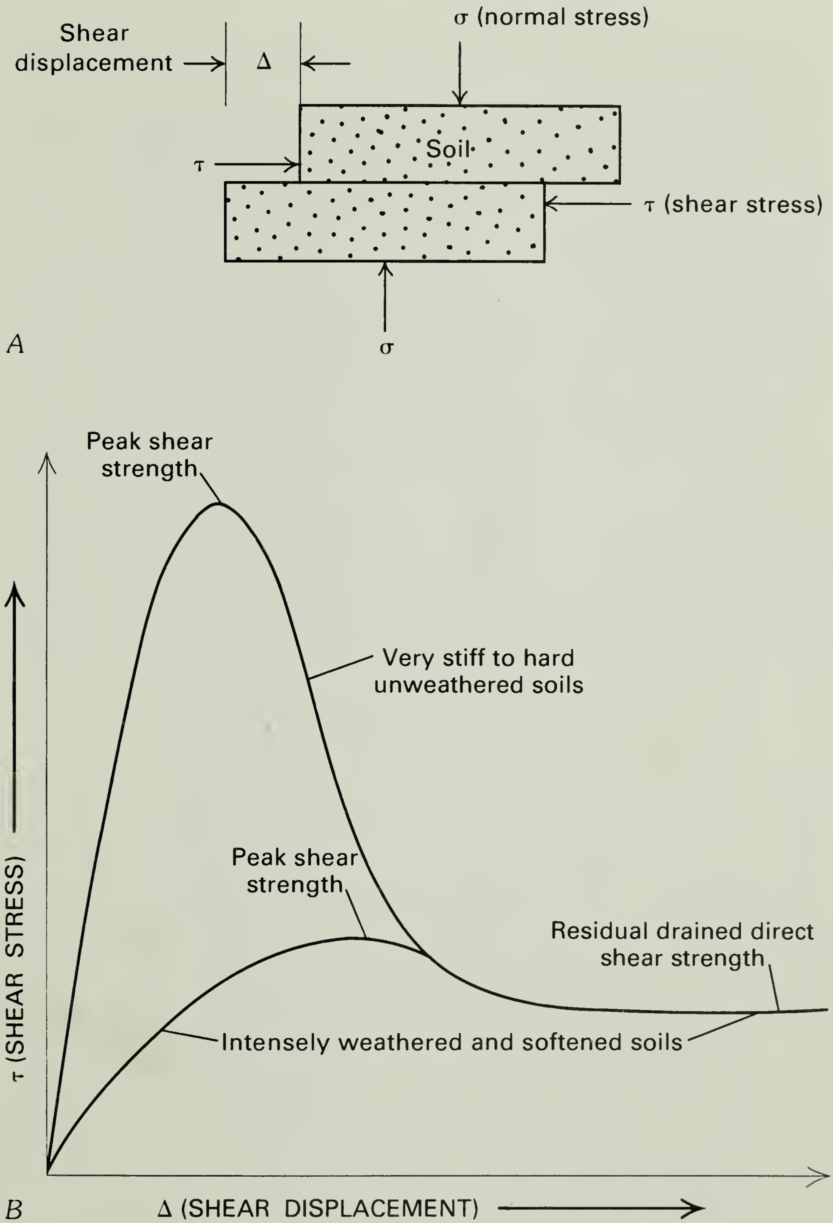

FIGURE 8.—Typical strength relationships for drained direct shear tests on Potomac Formation sediments. *A*, Direct shear test stress and displacement conditions. *B*, Plot of shear stress versus shear displacement constant normal stress.

REGIONAL SIGNIFICANCE OF DATA

The preceding generalizations apply in the main not only to montmorillonite-facies deposits outside Fairfax County and vicinity but also to clay- and silt-rich deposits of the kaolinite-illite facies. Many thick clay beds in the kaolinite-illite facies have sufficient plasticity to be designated as CH soils, but clays having a large shrink-swell potential are probably not as commonplace as they are in the montmorillonite facies.

Sand-rich deposits of both the montmorillonite and the kaolinite-illite facies are generally compact; a major difference is that sands in the kaolinite-illite facies are commonly clean quartz and only weakly cemented, although, at some places, lenses are strongly cemented.

POTOMAC FORMATION CHARACTERISTICS RELEVANT TO SLOPE STABILITY

Specific geologic and engineering geologic characteristics that bear on investigations of slope stability include sedimentation and mineralogical characteristics, faults and other fractures, and ground-water conditions. Each characteristic is discussed separately, by facies if the difference between facies is important, and related to slope stability.

SEDIMENTATION CHARACTERISTICS

The nature of the bedding is commonly closely involved with landsliding in that thick clay-rich deposits are more prone to sliding than thin-layered sand and clay strata are. In layered strata, the surface of rupture can follow a single thin lens for a long distance.

Sedimentation characteristics of the Potomac Formation (Glaser, 1969) include large and abrupt variations in lithology, both vertically and laterally, and variable combinations of interbedded and intertongued layers of gravel, sand, silt, and clay (see fig. 9 for example). As a result, the composition and grain size of the soils in a borehole may differ greatly from those in a location only 10 feet (3 m) away.

Gravel-bearing sands (fig. 10) are present at some places within the lowermost 100 feet (30 m) of the Potomac Formation but are much less common higher in the section. The section generally fines upward, a notable exception being some extensive, continuous, sand-filled channels at different levels. Sands are generally crossbedded, of medium texture, and uniformly graded (fig. 11).

Fine-grained sediments of the Potomac Formation were deposited as overbank flood-plain deposits or as fillings in abandoned meanders. These sediments occur in irregular or lenticular bodies interbedded with the sands and gravels at all levels. Remnants of bedding are

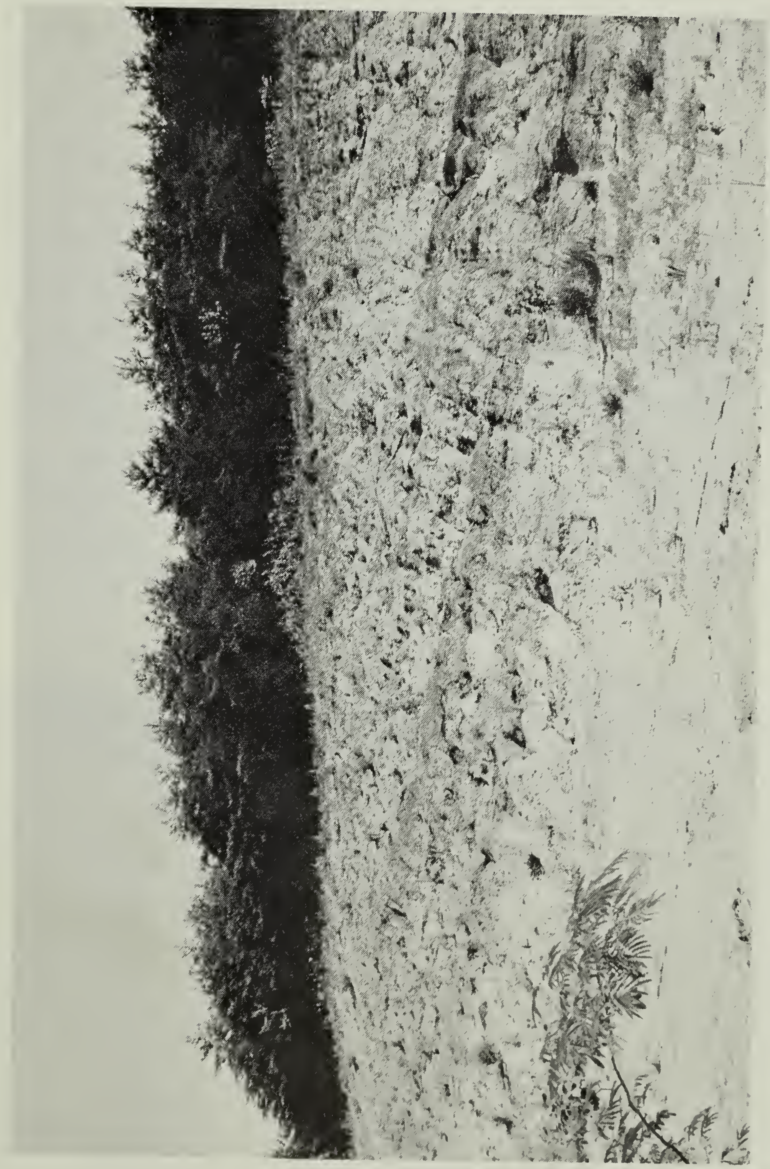

FIGURE 9.—Typical interbedded strata of Potomac Formation sand (light colored) and silty clay (dark colored).

FIGURE 10.—Closeup of typical Potomac Formation sand and gravel deposit. Entrenching shovel shown for scale.

FIGURE 11.—Typical outcrop of Potomac Formation thick sand deposit.

variably preserved in the clay-rich strata (figs. 12, 13), but the thicker bodies are commonly devoid of any trace of internal stratification.

The areal extent and dip of an individual stratum are generally functions of bed thickness and texture. Fine-grained and thick beds tend to persist for greater distances. Although individual thin beds of sand, silt, or clay commonly change lithology or end laterally over a short distance, thick beds can be continuous for thousands of feet. Characteristic areal extents and dips of the thinner strata are shown in table 2. All dips shown in table 2 can continue in a clay bed for 100 feet (30 m) or more downslope, a distance long enough to be important to the types of landslides commonly occurring in the Potomac Formation.

The regional tilt of the formation (that is, the dip of the base of the formation) is generally toward the southeast and varies between 15′ and 50′ (25–75 ft/mi) (Glaser, 1969). The steepest regional tilts are usually in the westernmost sediments.

Deposition-related slump features are commonplace and are evidenced by crumpled beds and thoroughly mixed materials. These ancient slumps were subsequently consolidated by the weight of the overburden, and they presently have no inherent weaknesses that are particularly important to modern landsliding. Irregular undulatory bedding is common and undoubtedly the result of differential compaction in most cases.

FIGURE 12.—Typical outcrop of thick Potomac Formation clay-rich deposit unconformably overlain by gravel.

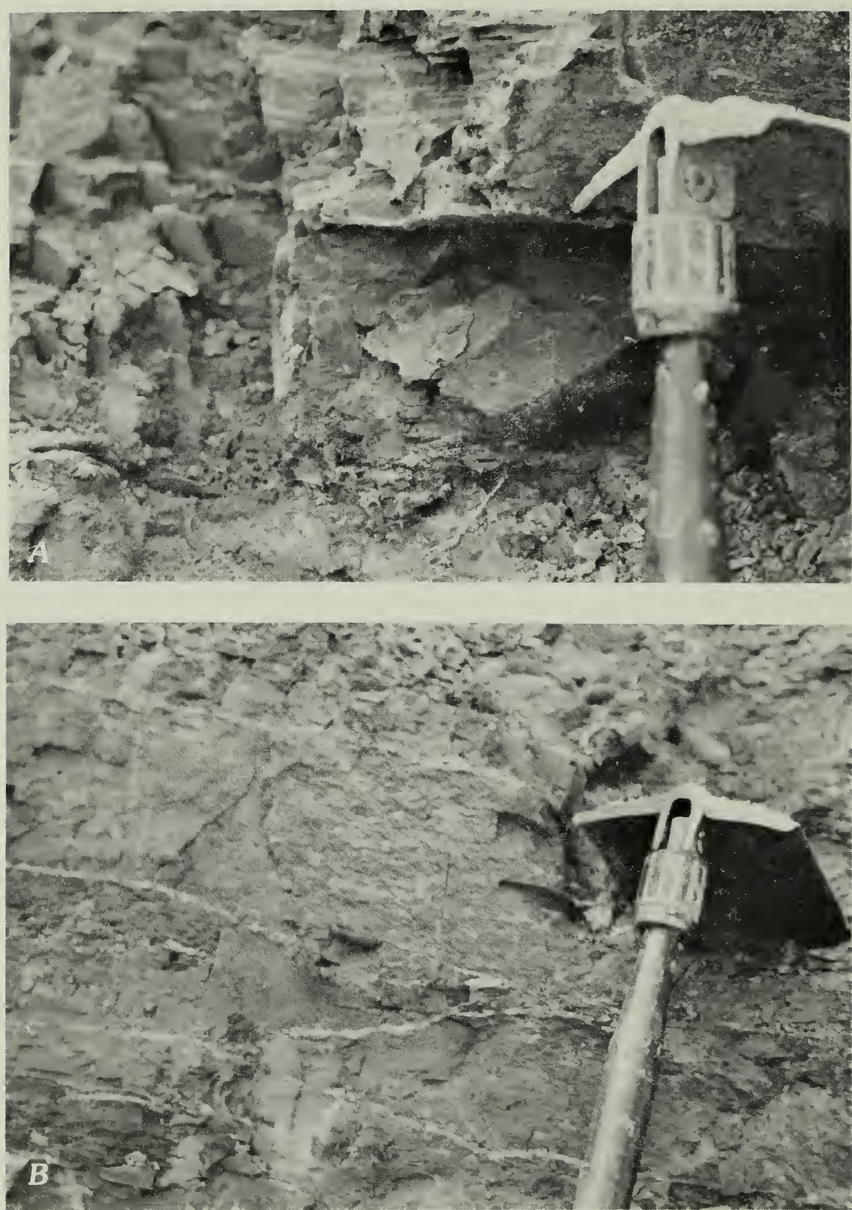

FIGURE 13.—Weathered, softened bedding surfaces (thin, light-colored zones) in Potomac Formation clay-rich deposits. Entrenching shovel shown for scale.

TABLE 2.—*Areal extent and dip of discrete thin layers in the Potomac Formation*

[Based on detailed field observations throughout the Baltimore, Md., area and supplementary observations throughout the greater Fairfax County, Va., area. Data by Juergen Reinhardt (written commun., 1978)]

Material[1]	Bed thickness	Areal extent,[2] in feet (meters) Maximum[4]	Areal extent Average	Dip,[3] in percent Maximum	Dip Average
Clay	1 in. (2.5 cm)	100 (30)	5 (1.5)	25	≤2
Do.	6 in. (15 cm)	500 (150)	25 (8)	20	≤2
Do.	3 ft (1 m)	1,000 (300)	100 (30)	10	≤2
Silt	1 in. (2.5 cm)	20 (6)	3 (1)	35	≤3.5
Do.	6 in. (15 cm)	100 (30)	10 (3)	20	≤3.5
Do.	3 ft (1 m)	Not observed			
Sand	1 in. (2.5 cm)	10×5 (3×1.5)	1 (0.3)	55	≤10
Do.	6 in. (15 cm)	100 × 50 (30×15)	5 (1.5)	35	≤10
Do.	3 ft (1 m)	500×100 (150×30)	25 (8)	20	≤5

[1]Materials approximate end members of clay, silt, or sand beds; that is, they are comprised almost entirely of clay, silt, or sand.

[2]Maximum areal extent values are probably rare.

[3]Observed dip of beds is a result of initial (depositional) dip plus tectonic dip owing to regional tilt.

[4]Bodies approximately equidimensional except where two dimensions are listed.

Another common deposition-related feature is clay clasts in sand or gravel beds, particularly along the base of the beds. These clasts are roughly spherical balls ranging from 0.1 in. to 3 ft (0.3 cm–1 m) or more in diameter, the smaller sizes typically being most abundant. In some places, the clasts comprise the major proportion of the sand- or gravel-sized materials, and the shear strength of the mass can be controlled by properties of the clasts.

MINERALOGICAL CHARACTERISTICS

Force and Moncure (1978) have described primary and weathering-related clay minerals of the Potomac Formation. Their work, in conjunction with Glaser's (1969) treatise, comprises the most thorough extant mineralogical studies.

PROPERTIES OF MONTMORILLONITE FACIES

The clay-rich beds in the montmorillonite facies are dominated by montmorillonite-illite mixed-layer clay and relatively pure montmorillonite; these beds also have some layers of almost pure montmorillonite or illite or of clay mixtures of montmorillonite and illite containing small amounts of vermiculite or kaolinite. Clay beds in the southern third of Virginia have much more mixed-layer clay and illite. Exchangeable cations in unweathered clay-rich Potomac Formation sediments in Fairfax County are dominantly Ca^{++} and Mg^{++}; greatly subordinate Na^{+} and K^{+} are also present (table 3). Dominant cations are probably the same or very similar throughout the montmorillonite facies.

The unweathered clay-rich layers in the montmorillonite facies are waxy and commonly contain a notable proportion of silt. Their colors vary considerably. These layers are commonly gray or greenish brown but, in many places (usually the overbank deposits), are variegated yellow and red owing to brightly colored iron oxides precipitated during or very shortly after deposition. In some places (usually the meander fillings), the clay-rich layers have been colored dark gray or black by carbonaceous matter and contain pieces of plants and woody material.

Unweathered dark-colored clay-rich sediments (that is, gray, greenish-brown, black, and some reddish-tinted sediments) have inclusions of pyrite or marcasite in some places. Water introduced into or flowing over these sediments is usually highly acidic (pH 2–3) (Wagner and others, in press). The color can change to a yellow brown within a few hours after exposure to air and water, probably because iron oxidization is accelerated by acidic water.

Force and Moncure (1978) have shown that the montmorillonite is currently weathering to other clay minerals (dominantly kaolinite) where it has been softened and leached by freely flowing ground water. Tests by the author and by Wagner and others (in press) show that, in many places, the Ca^{++} cations have been largely removed from the soft clays. Highly acidic water probably greatly accelerates the rate of extraction of Ca^{++} and the alteration of montmorillonite to kaolinite. The rate of alteration of montmorillonite to kaolinite is still probably slow (say, 100 years) in relation to man's actions, but it is possible that, in some places, the cations can be extracted quite easily and quickly. Alteration of montmorillonite to kaolinite almost certainly increases the residual shear strength of the clay (Mitchell, 1976); the removal of cations has an unpredictable influence (Mitchell, 1976) but probably decreases the residual shear strength.

With few exceptions, the sand-sized fraction of sand layers in the montmorillonite facies consists of potash and plagioclase feldspars and quartz, which are typically cemented with kaolinite, clasts of montmorillonite-rich soil, and possibly some iron oxides and silica. The feldspars typically make up a large percentage of the sample and are intensely weathered. The kaolinite cement was probably derived from weathering of the feldspar soon after deposition, and this clay-mineral matrix was subsequently hardened largely by the weight of the overburden. Typical light-gray unweathered sand bodies have unconfined compression strengths approaching those of the clay

TABLE 3.—*Textural and chemical data and Atterberg limits for some unweathered clay-rich Potomac Formation samples*

[Tests performed by the U.S. Soil Conservation Service, National Soil Survey Laboratory, Lincoln, Nebr. Test procedures described in a report by the staff of the U.S. Soil Conservation Service (1972). tr, trace amount]

Sample	Texture			Cation exchange capacity by sum of cations, in milli-equivalents/100 g	Ammonium acetate extractable bases, in milli-equivalents/100 g				Atterberg limits, in percent water content	
	Clay (< 2 mm), in percent	Silt (2–5 mm), in percent	Sand (> 5 mm), in percent		Ca^{++}	Mg^{++}	Na^{+}	K^{+}	Liquid limit	Plasticity index
Montmorillonite facies (Fairfax County, Va.):										
Site A	69.0	29.3	1.7	62.8	33.1	21.7	0.4	2.4	104	61
Site B	52.8	46.7	.5	55.4	25.2	18.7	.1	1.4	82	46
Kaolinite-illite facies (Washington, D.C.)	35.0	57.0	8.0	5.4	.5	.7	tr	tr	38	18

layers. Selective chemical weathering in the sand layers has produced very thin reddish-tinted weak zones along the contact with an underlying clay-rich layer.

Iron-oxide-cemented sands are widespread and commonplace. Very strong cemented layers 6 ft (2 m) thick or more are not rare. Normally, the iron-oxide-cemented zones are in layered crusts concentrated at unconformities where ground water has been flowing and are present in association with the more permeable sands.

PROPERTIES OF KAOLINITE-ILLITE FACIES

Clay-rich beds in the kaolinite-illite facies are typically dull white, yellow, or red, except locally, where they are dark gray or black when carbonaceous material is present. Unweathered clay-rich beds of this facies are also waxy, and silt is abundant in many of the clay-rich beds. The clay minerals are primarily kaolinite and illite; rarely, a small amount of montmorillonite is present. A small percentage of vermiculite occurs in the upper 6 ft (2 m) of the formation in many places, probably from ongoing, very slow weathering. The presence of vermiculite has been associated with regions of most intense oxidization and probably results from alteration of illite and other minerals (Schluger and Roberson, 1975). Thus, there is a good possibility that chemical factors are involved in weakening of the clays.

Large masses and concretions of rocklike siderite are randomly distributed in the more massive clay deposits of the kaolinite-illite facies.

Sand layers of this facies are normally almost pure quartz and contain only minor amounts of feldspar. These sands are usually very weak and friable and exhibit little or no cohesion; locally, however, the sands are so strongly cemented with siderite or silica that they cannot be penetrated with tools routinely used for soil investigations. Where sands are at the ground surface, they are commonly weakly cemented, probably with silica.

Iron-oxide-cemented sands are as common in this facies as they are in the montmorillonite-clay facies. Typical thicknesses, strengths, and locations of the cemented sands are about the same for both facies.

FAULTS AND OTHER FRACTURES

The faults that cut the Potomac Formation in Fairfax County and southward (that is, in the montmorillonite facies) are not commonplace, but they are widespread. Numerous sheared zones (or very small faults) and relict joints[3] are present in many places near the larger faults at diverse orientations dipping from nearly vertical to horizontal. Near the faults, the joints are commonly continuous for long distances and have strongly preferential strikes (Langer and Obermeier, 1978). No faults have been documented in the kaolinite-illite facies, possibly because there have been no significant studies of tectonic effects in the Potomac Formation north of Washington, D.C. A feature suspected to be a fault, observed by the author near the northern end of Chesapeake Bay, had many smaller ancillary shear zones and joints, which appeared to be similar in form and occurrence to those in Fairfax County.

Faults, sheared zones, and joints form zones of weakness in otherwise stronger sediments. Faults and shear zones have similar shear strength characteristics that are important to landslide considerations; they typically differ from characteristics along bedding-plane partings or joints and surface weathering fractures.

PROPERTIES OF SHEARED ZONES, JOINTS, AND BEDDING-PLANE PARTINGS: MONTMORILLONITE FACIES

The properties discussed reflect observations made primarily in Fairfax County, although the author believes that, in the main, they apply throughout the montmorillonite facies. Sheared zones (and faults) have gouge that ranges in thickness from 0.25 in. (0.6 cm) or less (fig. 14) to 6 in. (15 cm) or more. They occur as a single shear or as a set of smaller shears spaced tenths of inches (0.2 cm) to 6 ft (2 m) apart. Shear surfaces are commonly planar to gently undulating and slickensided (fig. 15). The sheared zones are commonly softened because of remolding during shearing or because of weathering or soaking with water. Fragments of hard, unaltered, original material surrounded by soft soil are present in some of the sheared zones. Sheared zones are rarely coated with strong black material (probably a manganese oxide). The author believes that the shear strength of the soil in sheared zones (except those coated with this black material) should be considered as being at the residual state, with only a very small or no apparent cohesion on the Mohr strength envelope.

Even far removed from faults, joints are rather common in the uppermost 20 ft (6 m) of the Potomac Formation in thick, highly plastic clays. In Fairfax County and vicinity, both vertical and subhorizontal joints in the thick, highly plastic clays are commonly throughgoing for long distances and are generally planar (fig. 16) (Langer and Obermeier, 1978); some of these "joints" are bedding-plane partings, and it is impossible to distinguish between the two in

[3]Joints are defined as those fractures, excluding faults, sheared zones, and bedding-plane partings, that are present beneath the zone of intense weathering and wetting and drying near the ground surface (that is, joints are the breaks present at depths greater than about 5 ft (1.5 m)). Fractures include all breaks, irrespective of location, form, or origin.

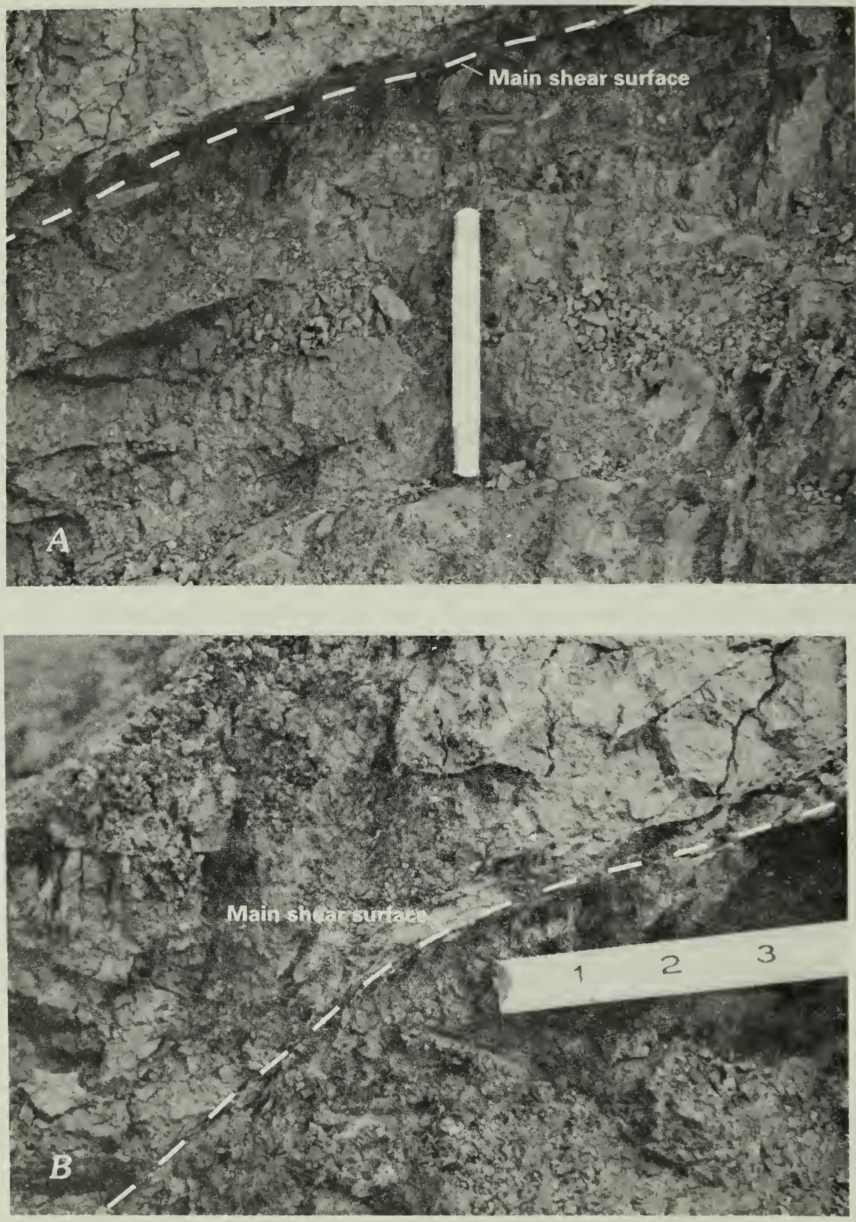

FIGURE 14.—Sheared zones in clay-rich Potomac deposits. Six-inch ruler for scale.

FIGURE 15.—Slickensides along a sheared zone in the Potomac Formation. Nine-inch ruler for scale.

many places. Although they are generally planar for long distances, the subhorizontal joints and bedding-plane partings typically have many small, closely spaced undulations; their wavelengths are on the order of a few inches to several feet (5 cm–1 m), and their amplitudes range from 0.25 in. to a few inches (0.5–5 cm). These subhorizontal fractures generally parallel the land surface and tend to be much more closely spaced in the uppermost 3 ft (1 m) of the Potomac Formation. Weathering and softening along joints and partings are more intense near the ground surface and at the erosional unconformity at the top of the Potomac Formation than they are at greater depth. The subhorizontal fractures are commonly offset horizontally a fraction of an inch (a few millimeters), even on nearly level ground. These subhorizontal fractures are probably formed by an exfoliation type of phenomenon caused by rebound from erosional unloading and by swelling of the montmorillonite.

In Fairfax County, joints are commonly well developed in massive, highly plastic (CH) clay beds thicker than 3 to 5 ft (1–1.5 m). Thinner layers of highly plastic clay have either poorly developed joints or no joints, and alternating beds of clay and low-plasticity silt or sand thinner than 6 in. (15 cm) rarely have joints. Thick, massive clay of low plasticity (CL) and silt of high plasticity (MH) have well-developed, closely spaced (1–2 ft) (0.3–0.6 m) vertical to horizontal joints at some localities, but joints are rare or absent at other sites. Thinly bedded

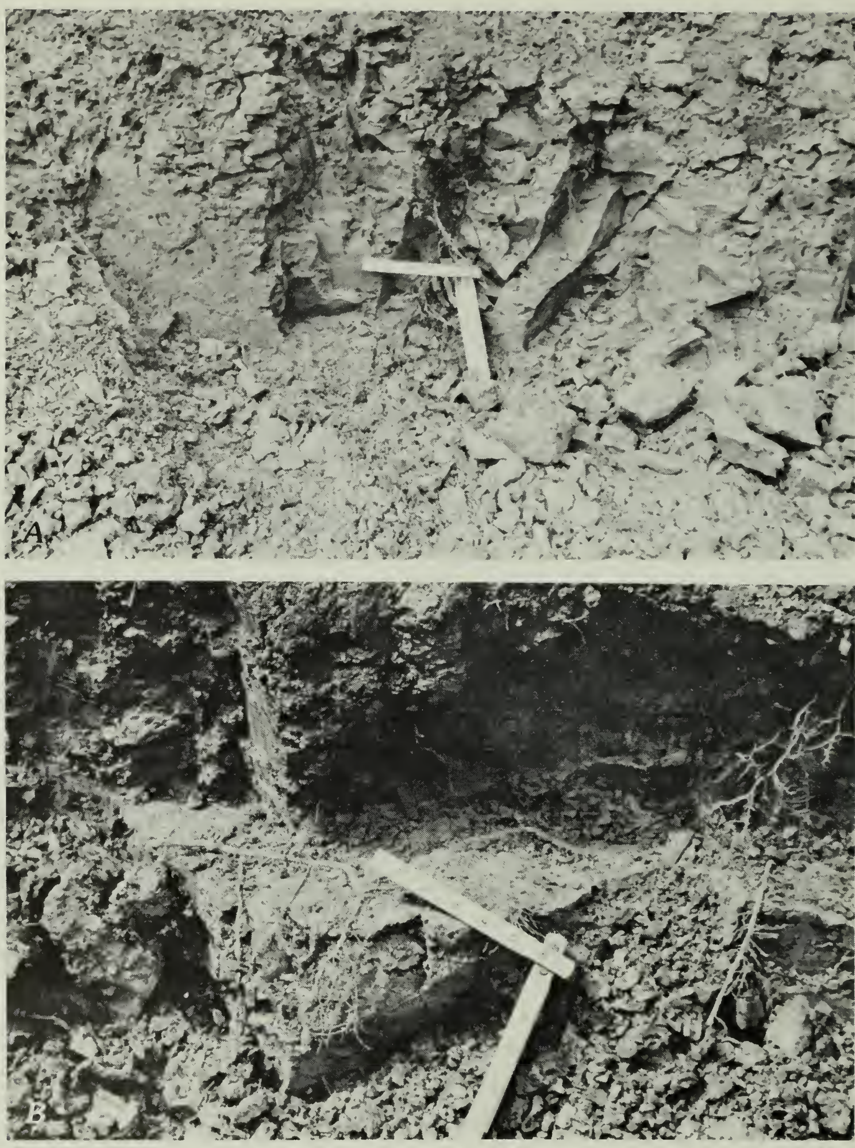

FIGURE 16.—Well-developed vertical and subhorizontal joints in the montmorillonite facies. Ruler is divided in inches.

layers of silt and sand intercalated with low-plasticity sediments commonly have no recognizable joint development. Massive sand units (SC or of lesser plasticity) and massive silts of low plasticity (ML) rarely have joints. Rare occurrences of jointing in sand and silt of low plasticity are probably related to nearby shears of tectonic origin and are normally strongly cemented with a black manganese oxide.

Almost all sites in Fairfax County that have abundant high-angle joints also have low-angle joints that are slightly undulating and continuous for at least several tens of feet. Water seeps from these joints at many localities, and the soil along the joint generally has a soft to medium consistency. The softened soil is present in a zone having a thickness of between 0.1 and 0.5 in. (0.3–1.2 cm). The transition from the softened soil to the hard, unweathered soil is commonly very abrupt (fig. 17). The weakest material is generally light colored. Subhorizontal zones containing soft soil are probably commonplace in clay and highly plastic silt within approximately the upper 20 ft (6 m) throughout the montmorillonite facies.

PROPERTIES OF JOINTS AND BEDDING-PLANE PARTINGS: KAOLINITE-ILLITE FACIES

Joints are also common in the upper 20 ft (6 m) of thick bodies of highly plastic clays in the kaolinite-illite facies. Relict vertical, randomly oriented, generally irregular throughgoing joints spaced 1 to 2 ft (0.3–0.6 m) apart commonly extend much farther than 20 ft (6 m) beneath outcrops and beneath the erosional unconformity at the top of the Potomac Formation in the thick clays of the kaolinite-illite facies in Washington, D.C. In the Baltimore area, near-vertical, very irregular throughgoing joints commonly extend at least 20 ft (6 m) in the most highly weathered massive clays; these fractures are typically about 2 to 10 ft (0.6 to 3 m) apart at random strikes (Juergen Reinhardt, oral commun., 1978). Subhorizontal joints appear to be much less common in the kaolinite-illite facies than they are in the montmorillonite facies, possibly because clays in the kaolinite-illite facies typically have lower swell potential.

SHEAR STRENGTH ALONG JOINTS AND BEDDING-PLANE PARTINGS

The softened zone along a subhorizontal fracture commonly contains soil in highly variable preconsolidation states (determined by a consolidometer); the state can vary from being less than existing overburden vertical stress to being greater than that stress within a distance of less than 1 ft (0.3 m) along the joint. Figure 18 illustrates how the small, slightly weathered projections in point-to-point contact can carry most of the overburden load. Between these points, the fractures are open in many places; in other places, they are filled with

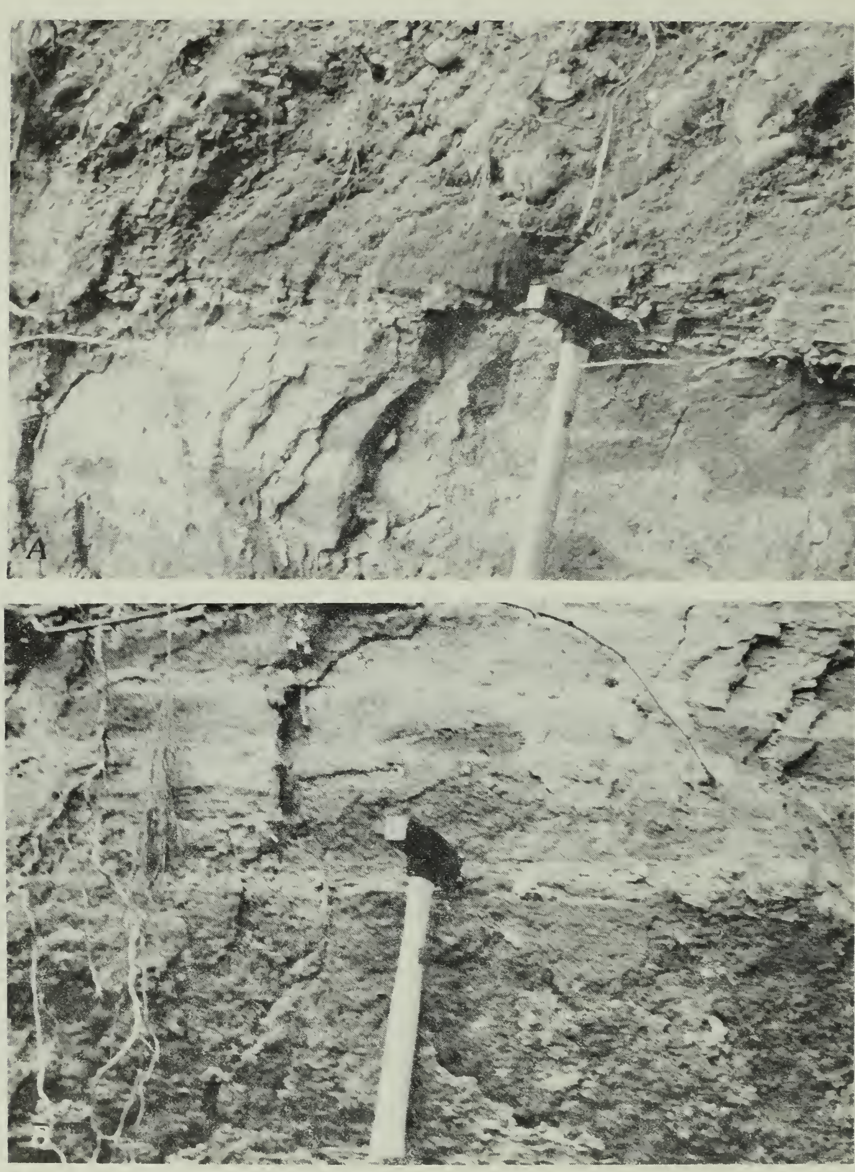

FIGURE 17.—Typical softened zones (light colored) in unweathered clay (dark colored) of the Potomac Formation. Hammer is approximately 12 in. long.

washed-in or swollen clay. The lowest possible average vertical preconsolidation state of stress along the fracture surface equals the present average vertical overburden stress.

The shear strength along a subhorizontal fracture that resists landslide movement depends on (1) the consolidation state along the fracture, (2) the state of drainage along the fracture during shearing, and (3) the force required to move the hard points over one another or the force required to shear through the hard points. For slope design, in many places, it is realistic and prudent to assume that the consolidation state along the fracture equals the existing overburden stress. Many fractures are so clay choked that the soils must fail in undrained shear along a thin, restricted zone; this failure can be closely simulated by the direct shear test method. The force required to move hard points over one another or to shear through the hard points obviously imparts an apparent cohesion to the shear strength but cannot be realistically appraised in the laboratory by using conventional equipment. Only a lower bound shear strength that accounts for the three factors listed above can be determined, and that strength may be much lower than the true shear strength.

PROPERTIES OF SURFACE WEATHERING FRACTURES: MONTMORILLONITE FACIES

Surface weathering fractures, defined as breaks at or near the ground surface, make up another class of breaks that are important to slope stability. Surface fractures are numerous and very commonplace in the clay-rich deposits of the montmorillonite facies. In Fairfax County (Langer and Obermeier, 1978), surface fractures commonly extend about 5 ft (1.5 m) beneath the ground surface at various orientations between vertical and subhorizontal (fig. 19); the fractures commonly form irregular chunks, and subhorizontal breaks can be well developed and throughgoing. Movement on some subhorizontal breaks has been sufficient to make slickensides in the highly plastic clays, although the relative movement along the two sides of the break probably rarely exceeds a fraction of an inch.

Surface-fracture density decreases with diminishing clay content. In sand-rich deposits, surface fractures are present only where weathering has been intense and probably prolonged. Here, the fractures are typically short (0.25–3 in.) (0.6–7.5 cm), discontinuous planar breaks, either vertical or horizontal and usually widely spaced.

PROPERTIES OF SURFACE WEATHERING FRACTURES: KAOLINITE-ILLITE FACIES

Surface fractures are also numerous and commonplace in clay-rich sediments of the kaolinite-illite facies. Here, however, the surface

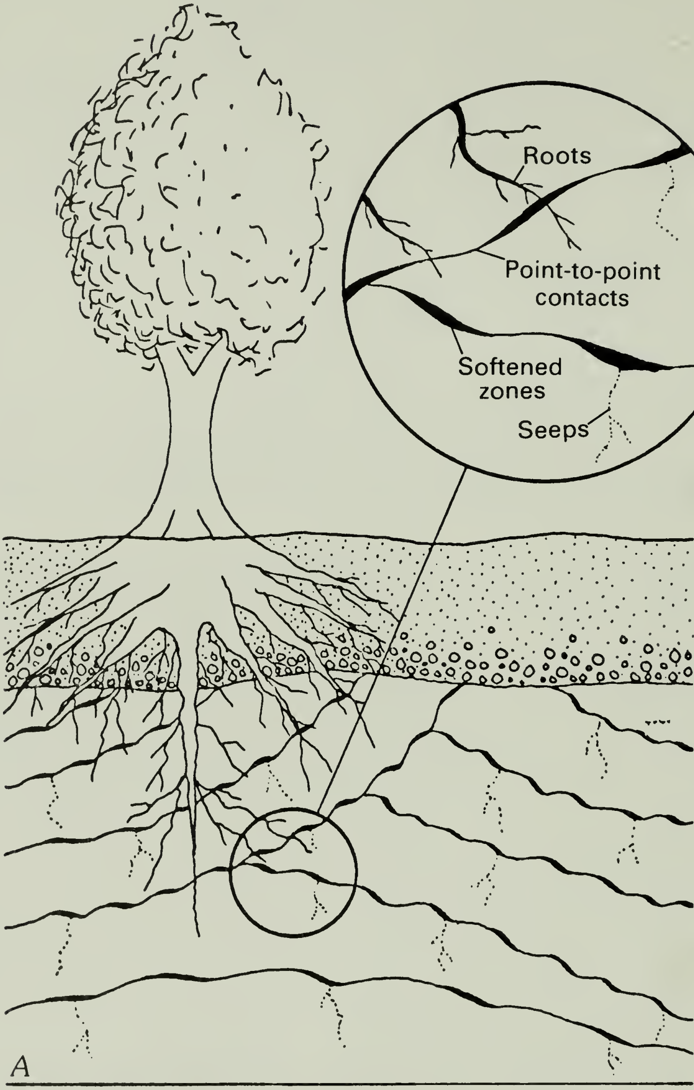

FIGURE 18.—Locations of softened zones along weathered and infilled partings and fractures of the Potomac Formation. *A,* Along bedding partings. *B,* Along fractures.

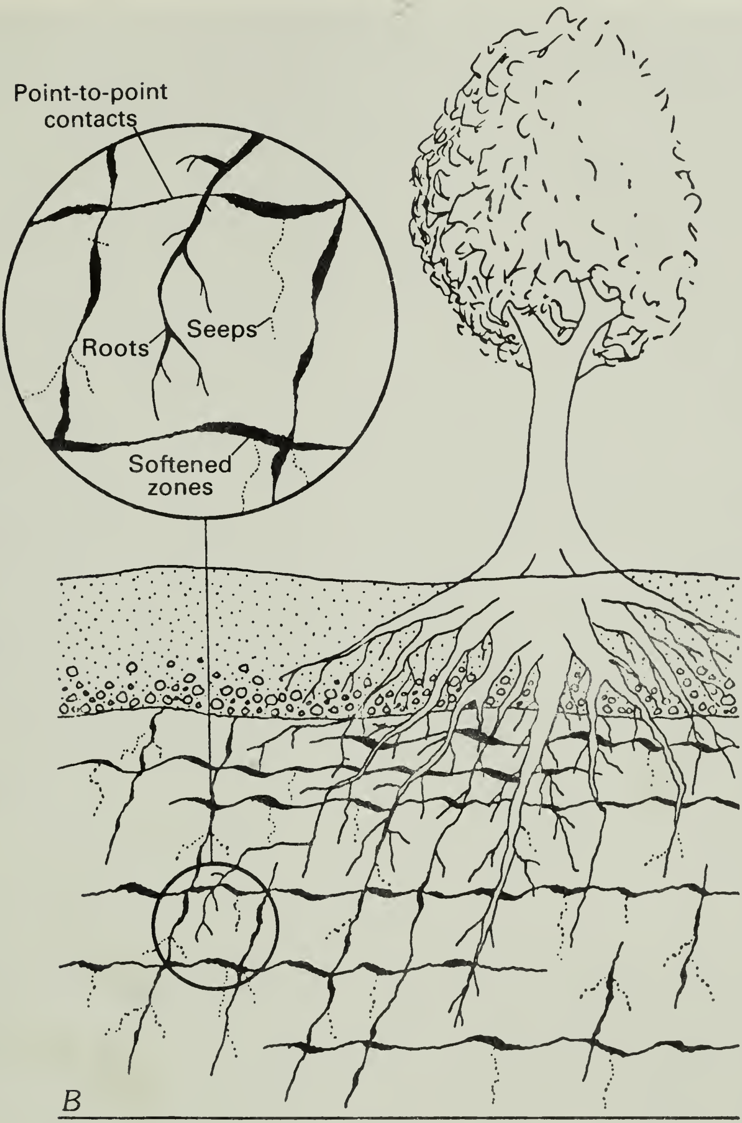

FIGURE 18.—Continued.

FIGURE 19.—Surface fractures in clay-rich Potomac Formation sediments of the montmorillonite facies. Shovel handle approximately 21 in. long.

FIGURE 19.—Continued.

fractures are oriented more nearly vertically and make large blocks (fig. 20). Many of these fractures extend at least 5 ft (1.5 m) beneath the ground surface. The fractures in the sands are usually only short, discontinuous breaks.

SHEAR STRENGTH ALONG SURFACE WEATHERING FRACTURES

Surface fractures probably develop within a few years after exposure of unweathered clay-rich sediments. Because of the intense development and movement along fractures, the shear strength of highly plastic soils must be much less than that of adjacent intact sediments. The movement along fractures, however, is probably not large enough to reduce the shear strength to the residual strength condition. The presence of stronger projections along the fractures and infilled soils also contributes to strengths greather than residual.

GROUND-WATER CONDITIONS

The water table is generally a subdued replica of local topography on outcrops of the Potomac Formation and in places where the Potomac Formation is near the ground surface. The water table is commonly very close to or at the ground surface, beneath clay-rich slopes and at the base of slopes; on hilltops, it is rarely deeper than a few tens of feet. During wet seasons, the water table is very close to or

FIGURE 20.—Surface fractures in clay-rich Potomac Formation sediments of the kaolinite-illite facies. Entrenching shovel approximately 18 in. long.

at the ground surface, and perched water tables and artesian conditions are very common.

The local geology exerts an important influence on ground-water conditions because of the highly differing permeabilities of sands and clays and because fractures form passageways for water.

The thin feldspathic sands of the montmorillonite facies are typically so clay rich that, except where there are coarse-grained channel fillings, they are usually not good aquifers. The sands are still much more permeable than the montmorillonite-rich layers, however. The quartz sands associated with the kaolinite-illite facies are generally very permeable. As a result, in both facies, perched and confined water tables are common in the layered clay and sand sequences. Water in some confined sand layers is under high artesian pressure.

The water table in thick sand deposits on hilltops and slopes is commonly deeper than that in adjoining clay and silt bodies. Both layered and massive clays and silts are typically so impermeable that they greatly restrict the rate of flow and help to prevent drainage of water from these sites. It is possible (and indeed probable, given the remarkable lack of weathering of montmorillonite in most places) that some massive nonfractured clays are impermeable to laminar flow at gradients commonly present.

Most massive clays and silts have many fractures near the land surface that serve as ready entrances for surface water. The fractures are usually slightly opened by offsetting or have root holes that persist to a depth of about 10 ft (3 m) beneath the ground surface. Water can fill these fractures during wet seasons.

In Fairfax County and vicinity, the unconformity separating the upland gravels from the Potomac Formation marks the site where numerous small channels were cut into the Potomac Formation. These sand- or gravel-filled channels are currently sites for concentrated flow of water, as figures 6 and 21 show. Freely flowing water normally persists in these channels even during dry seasons.

In many places, near the ground surface, the upland gravels have a nearly impermeable hardpan, less than 3 ft (1 m) thick, that ponds water during wet seasons. It is possible that breaking this hardpan permits significantly increased water flow into the underlying permeable sand and gravel and, consequently, enhances the potential for landslides.

SLOPE STABILITY ENGINEERING

Residual shear strength parameters should be used to evaluate slope stability along any preexisting surface of rupture, which is a permanent zone of weakness. A practical problem often encountered by the designer is the recognition of former landslides. Many of the old slides are so subtly expressed that recognition in the field is

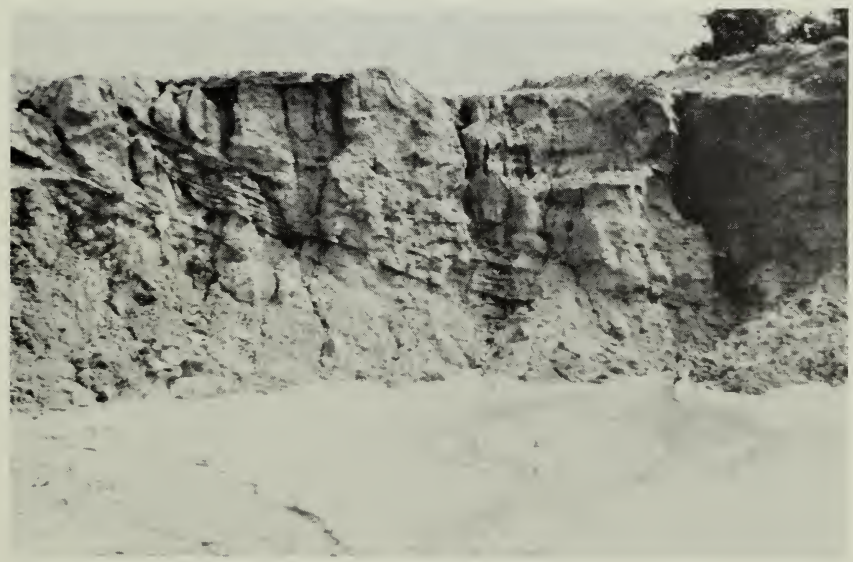

FIGURE 21.—Water seeping from a sand-filled channel cut into Potomac Formation clay-rich sediments. Outcrop is approximately 15 ft (4.6 m) high.

extremely difficult (Obermeier and Hollocher, 1976). Although tilted and bent trees can sometimes be an identification aid, they must be used with caution because mechanisms unrelated to landsliding also can tilt and bend trees. Slides that are in the early stages of development and that have very small scarps at the crown are virtually impossible to detect in most instances.

There are geologic settings in which it is impossible to recognize old slides from morphologic evidence because they are buried beneath younger sediments or have been modified. One such setting is along some major rivers, such as the Anacostia River (see fig. 3), where downcutting during glacial times at low sea level caused sliding. Subsequent readvance of the seas caused deposition of sediments over the slides. Other locations suspected of being old slide sites are at steeply sloping unconformities separating the Potomac Formation from younger sediments.

Where there is not an existing surface of rupture, the selection of appropriate shear strength parameters is much more complicated. Some of the factors known to be important to slope stability of Potomac Formation sediments are the presence of surface fractures, the presence of joints and partings, the presence of tectonic shear zones, the breakdown of diagenetic bonds by swelling and by creep, water penetration into joints, and the presence of physically and chemically softened zones. Obviously, there can be no completely theoretical basis for selecting shear strength design parameters when long-term slope stability is being evaluated.

A time-honored method of slope design in highly overconsolidated swelling sediments is to look for similar geologic settings in existing slopes. This method can work well for deposits that are not highly variable, but Potomac Formation sediments do not lend themselves to this sort of analysis unless a large factor of safety is used to account for uncertainties. Accumulated experience, summarized in the following papers, is the only realistic basis.

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ENGINEERING GEOLOGY AND DESIGN OF SLOPES FOR CRETACEOUS POTOMAC DEPOSITS IN FAIRFAX COUNTY, VIRGINIA, AND VICINITY

DESIGN AND CONSTRUCTION OF SLOPES IN POTOMAC FORMATION DEPOSITS

By PAUL G. SWANSON[1] and JOHN S. JONES, JR.[2]

CONTENTS

ILLUSTRATIONS

[1]Senior geotechnical engineer, Law Engineering Testing Co., McLean, Va.
[2]Chief engineer, Law Engineering Testing Co., McLean, Va.

ABSTRACT

On the basis of experience with Potomac Formation sediments, specific guidelines have been developed for investigation, design, and construction of slopes. These guidelines cover four phases: (1) field investigation, (2) design parameter selection, (3) analysis and selection of factors of safety, and (4) construction. All phases are used for evaluation of construction-period and permanent slope stability.

The field investigation includes a surface reconnaissance, during which indicators of previous landslides may be noted. Observations are made of bowed tree trunks, hummocky and disturbed ground, wetland vegetation, springs, and drainage pathways. This surface reconnaissance is followed by a subsurface investigation to determine subsurface stratification and the consistency of the layers, the locations of water-bearing zones and anticipated potentiometric levels, and the locations of fractures or shear surfaces.

Design shear strength parameters for normally consolidated Potomac Formation sediments are based on conventional test procedures. For first-time slides in overconsolidated sediments, however, design parameters are based largely on fracture intensity and laboratory shear strength data. Lower strength parameters are used for more highly fractured sediments. Either for highly fractured soils or along previously sheared surfaces, residual strength parameters are used for design of construction-period and permanent slopes. For soils having no fractures or widely spaced fractures, "fully softened" (normally consolidated, drained) shear strength parameters are used for design of permanent slopes; peak undrained triaxial shear strength data are used for design of construction-period slopes.

Selection of a safety factor for design depends on the uncertainty of slope conditions, the cost of construction or repair, and the potential hazards of slope failure. The critical design condition for the completed slope requires, in addition, that the slope be marginally stable, in case a subsurface drainage system fails completely.

It is important that a geotechnical engineer familiar with the design be at the site during construction to ensure that assumptions used for design are valid. Problems can arise when clay-rich soils are exposed to water for prolonged construction periods; consequent softening can cause localized failure.

INTRODUCTION

Rational design for fill and cut slopes in Potomac Formation sediments requires detailed knowledge of the material properties and geologic setting of the site. Clay- and silt-bearing sediments of the Potomac Formation are highly overconsolidated, stiff to hard soils, except where they have been softened locally by mechanisms such as weathering or by previous shearing. The highly overconsolidated soils have fractures in which softened soils are restricted to a thin zone along the fracture surfaces at many localities. Because slope design parameters commonly depend on the method of softening (previous shearing or weathering), a thorough field examination of the site is usually required. If a thorough investigation is not made, conservative design parameters must be used. As for permanent slopes, design parameters for construction-period (short-term) cut slopes and fill sections are strongly affected by the presence of softened soils.

Typical of highly overconsolidated soils, the stiff to hard unweathered Potomac Group sediments tend to dilate (that is, expand in volume) and thus develop negative pore pressures when they are stressed during construction of a cut or fill slope. The overconsolidated soils in the potential failure zone weaken and soften (that is, strain soften) during landslide development; the amount of strength reduction caused by this softening depends on the plasticity characteristics and shear displacement. In contrast to the highly overconsolidated soils, intensely weathered soils that are approximately normally consolidated can develop positive pore pressure during shearing, the result being either strain hardening at small strains or strain weakening at large strains. Previously sheared zones in landslides and faults can develop either positive or negative pore pressures during renewed shearing, depending on loading conditions, and the lower bound strength in these sheared zones will be equal to the residual shear strength.

Specific guidelines have been developed for investigation, design, and construction of slopes on the basis of experience with Potomac Formation sediments. These guidelines are for four phases: (1) field investigation, (2) design parameter selection, (3) analysis and selection of factors of safety, and (4) construction. All phases are used for evaluating short- or long-term slope stability.

FIELD INVESTIGATION

Slope design requires that a field investigation carefully document information ranging from an overview of the geologic setting to local geotechnical details exposed in trenches or revealed by borings. A field investigation begins with a surface reconnaissance, during which indicators of previous landslides may be noted. Observations are made of bowed tree trunks, hummocky and disturbed ground, and wetland vegetation. Also noted are springs, swales, and drainage pathways. Aerial photographs are used whenever it is possible. The surface reconnaissance is followed by a detailed topographic survey and then by a subsurface investigation.

The amount and detail of the subsurface data collected depend to a large extent on the anticipated consequences of any slope movement. Consideration is given to both potential economic losses and the possibility of endangering people. At a minimum, the subsurface investigation must determine the subsurface stratification and approximate consistency of the layers, the location of water-bearing zones, and the location of fractures or shear surfaces.

Test pits, soil borings, static cone probes, slope inclinometers, and observation wells are all commonly used to define subsurface conditions. In general, test pits and static cone probes are the best methods for locating water-bearing zones and fractured or sheared zones. Thin sand seams capable of transmitting water to the failure surface and thin softened zones are commonly not detected by using tube samples from borings. In contrast to borings, both test pits and static cone probes provide a continuous, detailed record of subsurface materials and conditions. A test pit is preferable for locating shallow shear surfaces, whereas cone probing is more suitable for detecting deeper shear surfaces.

Relatively undisturbed specimens of highly overconsolidated sediments are often difficult to obtain. The best specimens for laboratory evaluation of in-situ shear strength are from block samples excavated in test pits or from piston samples. For depths greater than about 15 ft (4.6 m), samples are obtained from borings advanced either by using hollow-stem augers or by continuous use of Shelby tubes. Where a shear zone is present, the shear strength is often determined by using remolded samples; this method eliminates the need for undisturbed samples.

It is imperative that sheared zones in previously failed slopes be located. Often, a change in drilling resistance, an abrupt change in water content, or a loss of drilling fluid (in the sheared zone) indicates a sheared zone. The best method for accurately locating the shear surface of an actively moving slide is by using slope inclinometer data. Figure 1 is a plot of slope inclinometer readings taken during investigation of a slide. Data on the figure illustrate that there are two shear surfaces and that the lower shear surface is about 16 ft (4.9 m) deep, in the weathered portion of the Potomac Formation.

The shear surface in an active slide can be located approximately by using a simple inclinometer consisting of a small-diameter plastic pipe or a stiff rubber hose (into which a steel rod has been inserted to maintain initial straightness) embedded in the borehole. The steel rod is withdrawn after backfilling around the hose. Probes are periodically inserted in the pipe or hose to detect any squeezing or shearing of the casing, which indicates the upper boundary of the failure zone. To define the lower boundary, a short steel rod is placed in the bottom of the tubing; a thin wire attached to the rod is used to pull the rod up the tube periodically until displacement resulting from slope movement restricts movement.

Slotted sections of slope inclinometer casing wrapped in filter cloth can be used to monitor slope movements and potentiometric levels simultaneously. Although this system is relatively inexpensive, it does have several disadvantages. It can monitor only one potentiometric level, and its response time may be greatly delayed because of

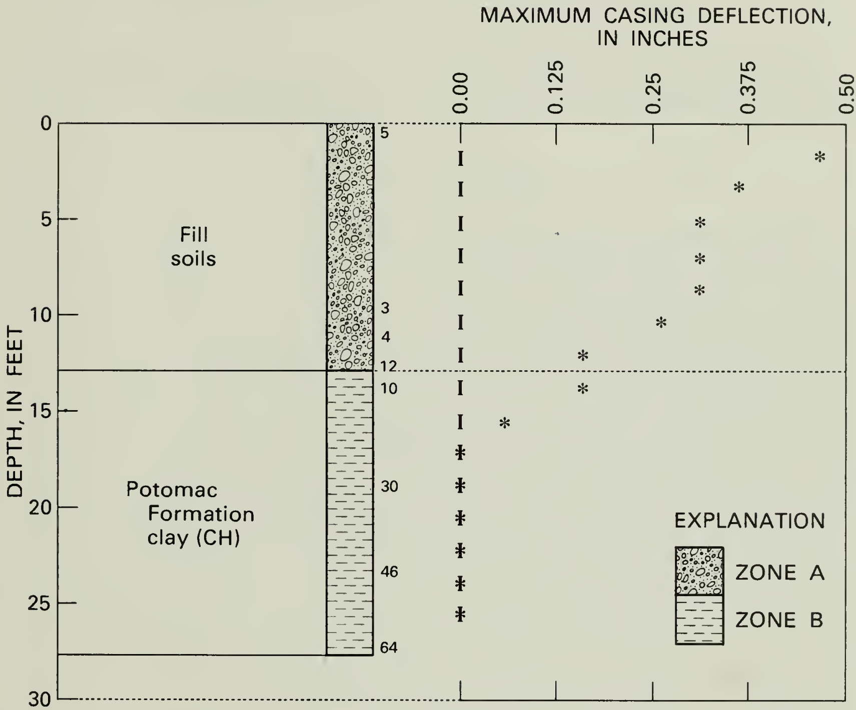

FIGURE 1.—Typical relationship between slope indicator and soils data. Numbers to the right of the soil symbols are Standard Penetration Resistances (in blows per foot). Inclinometer reading dates are I (initial), May 26, 1978; *, June 2, 1978. Soil properties are as follows: zone A, liquid limit, 60 percent; plastic limit, 30 percent; natural water content, 38 percent. Zone B, liquid limit, 69 percent; plastic limit, 33 percent; natural water content, 39 percent.

the large volume of water in the casing required to equalize pore pressure within the casing and pore pressure in the soil.

One of the more accurate methods for determining the potentiometric level at the failure surface or within confined layers uses a system of pressure seals and low-volume-change potentiometric collars. The system can be installed in a single borehole. The potentiometric collars have small pneumatic diaphragms that require only a small change in water volume to equalize pressures within the casing and existing ground-water pressures. Readings of the potentiometric levels can be made periodically by using a pneumatic probe. This type of installation has the advantage of monitoring several layers from a single boring. Although this type of system is expensive, it is recommended for investigations in the Potomac Formation where accurate knowledge of piezometric levels is needed.

SELECTION OF DESIGN PARAMETERS

Slope design parameters depend on the intended permanence of the slope, the geologic setting, and anticipated ground-water conditions.

DESIGN CONDITIONS

Design parameters for cut slopes or embankments or for correction of existing slope failures depend on the critical design situation. Upper and lower bound strength characteristics depend on the drainage status of pore pressures induced by a change in stress conditions. An undrained condition implies that an excess pore-water pressure has been generated in the soil by a change in stress conditions. A drained condition implies that enough time has elapsed for the excess pore-water pressure to dissipate. For most typical design situations, the construction period of an embankment or cut slope is considered to be the undrained or short-term condition. Figure 2 illustrates that excavation or filling causes an initial loss of strength, but, if excess pore-water pressure is drained, normally consolidated soils can strengthen. Highly overconsolidated soils are initially strengthened by negative pore-water pressures but can weaken over the long term as negative pressures dissipate. Both consolidation states are commonly present in the Potomac Formation. Thus, for the long-term drained condition, the Potomac Formation can be either stronger or weaker than it is for the short-term condition.

Design shear strength parameters for normally consolidated Potomac Formation sediments are evaluated by using conventional test procedures outlined in basic soil mechanics texts. For first-time slides in overconsolidated sediments, however, the design procedure is based on concepts proven approximately valid for overconsolidated clays elsewhere (Skempton, 1964, 1970, 1977) modified by local experience with the Potomac Formation; specifically, the concept of the critical state (Roscoe and others, 1958) is used in combination with the anticipated magnitude of shear deformation at failure to select the design shear strength.

A saturated clay in a drained test is at its critical state if any further increment in shear distortion does not cause any change in water content (Skempton, 1970). In the ideal case, a clay will expand or contract during drained shear until the critical state is reached, irrespective of the original state of consolidation of the clay. When the critical state is reached, the clay continues to deform under constant shear stress at constant volume. The critical state corresponds to the

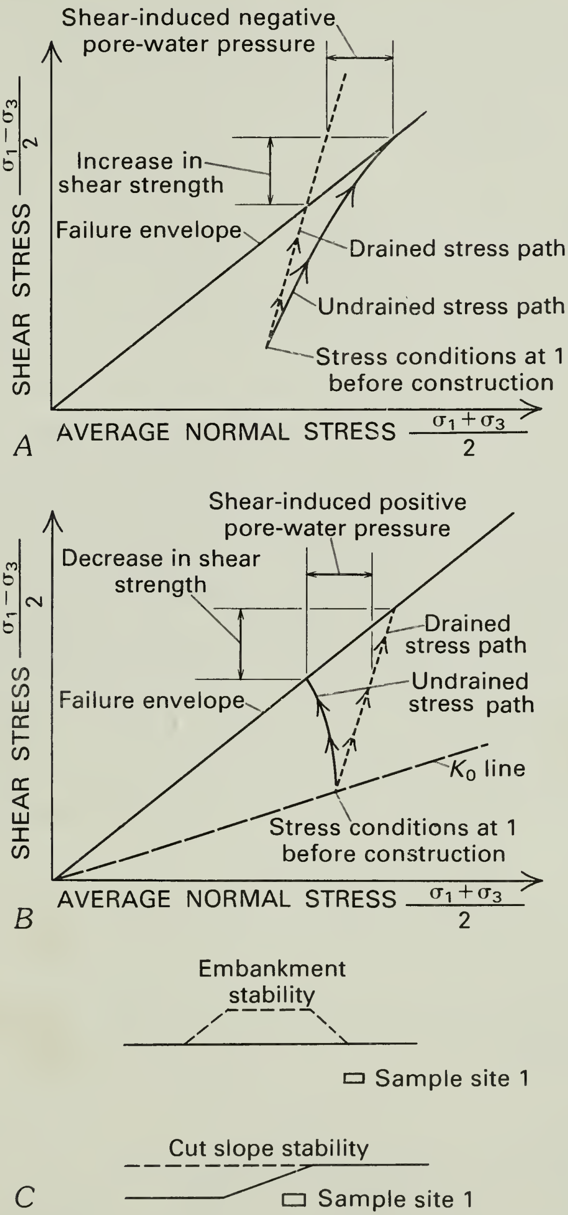

FIGURE 2.—Shear stress response as a function of consolidation and drainage states. *A*, Shear stress response for highly overconsolidated soils. *B*, Shear stress response for normally consolidated soils. σ_1, major principal stress; σ_3, minor principal stress; K_0, ratio of horizontal to vertical effective stress on level ground. *C*, Sample locations for embankments and cut sections.

strength of a normally consolidated, idealized clay; by definition, the water content at this state is equal to that ultimately achieved by the overconsolidated clay owing to dilatency with subsequent increase in water content during shear. This strength is designated the "fully softened" shear strength. Figure 3 is a schematic depiction of the relationship of the fully softened shear strength to the peak shear strength for real, highly overconsolidated clay-rich soils. For real soils, the fully softened shear strength is affected in a complex way by sensitivity, bonding, and particle orientation, but, for practical purposes, it can be considered as the peak strength associated with the remolded, normally consolidated, drained condition (Skempton, 1970). The peak strength of a remolded, normally consolidated clay is also the theoretical upper strength limit of an overconsolidated clay that has undergone weathering intense enough to obliterate evidence of previous overconsolidation (excluding the possible influence of changes in clay mineralogy and in absorbed and adsorbed ions).

For practical purposes, the fully softened shear strength is the lower bound strength of an overconsolidated clay-rich soil that has had particles reoriented and bonds disrupted by swelling or weathering but that has not experienced enough shear displacements to develop a discreet, smooth, continuous shear surface during the weakening process. Where there are large shear displacements during weakening and development of a continuous shear surface, real soils can weaken further to the residual strength, as figure 3 illustrates. The residual strength is characterized by an alining of the clay particles from continued deformation along a failure plane (Bjerrum, 1967; Chandler and Skempton, 1974; Lupini and others, 1981).

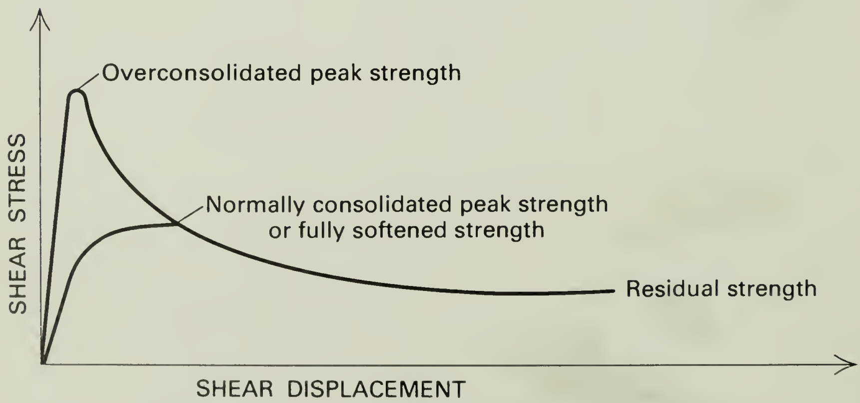

FIGURE 3.—Relative drained shear strengths for different states of consolidation.

The displacement required to reduce an overconsolidated clay to the fully softened condition is several times that required to mobilize the peak strength but is nevertheless not large and is considerably less than that corresponding to the residual strength. Typically, clay-rich soils develop residual strengths in laboratory direct shear tests after accumulating about 2 to 3 in. (5–7.5 cm) of displacement along the failure surface.

For first-time slides in highly overconsolidated Potomac Formation soils, the shear strength deemed appropriate for design is based on an evaluation of the soil's tendency to expand. This tendency, in turn, is determined by the fracture intensity, as the following section explains.

DESIGN SHEAR STRENGTH

Figure 4 is a flow diagram showing the procedure used for selecting design parameters for short- and long-term slope stability of Potomac Formation sediments. This procedure is used for design of cut slopes and embankments and for correction of existing landslides. Figure 4 shows that, after the subsurface investigation, a decision is made to design for "intact soil" or "failed soil." Criteria are based on the local experience of engineers and represent 30 to 40 slope stability studies conducted by the authors over the past 10 years.

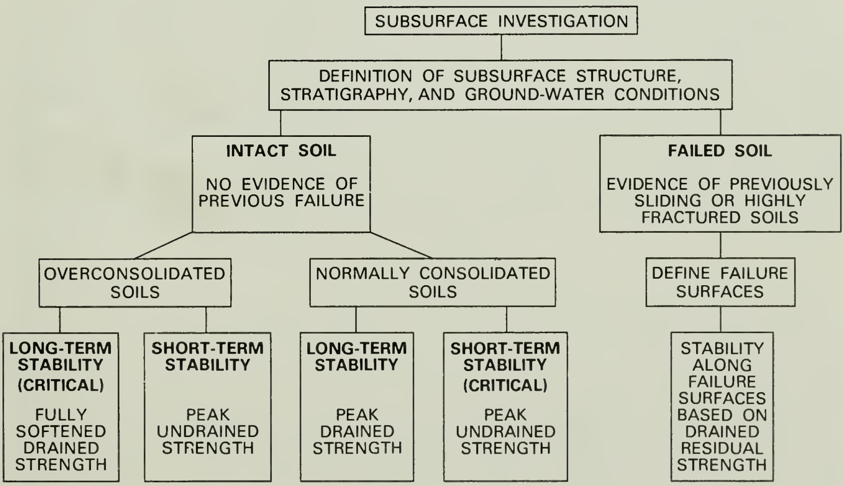

FIGURE 4.—Flow paths for selection of design parameters for slopes on Potomac Formation sediments.

INTACT SOIL

"Intact soil" applies to slopes where there is no evidence of previous failures or creep and the soils are not intensely fractured. An intensely fractured soil has many cracks that are observable when it is extracted from the ground. An intensely fractured clay-rich soil is extremely difficult to sample, and excavated portions of the soil are fragmented and can break into pieces 2 in. (5 cm) in diameter or smaller during gentle handling. Soils within the top 2 to 4 ft (0.7–1.2 m) of the soil profile are often intensely weathered and contain many fractures caused by the weathering process. Highly fractured soils below this depth may be more indicative of slide movement.

The critical design situation for permanent slopes in highly overconsolidated Potomac Formation soils is the drained condition. The stress changes described earlier that were caused by embankment construction or regrading induce excess negative pore-water pressures, which, in turn, temporarily strengthen the soil (fig. 2). A drained condition corresponding to long-term stability fails at a lower value of shear stress. The drained, fully softened shear strength is used for design of permanent slopes in massive to somewhat but not highly fractured, highly overconsolidated clay- and silt-bearing soils where no evidence of previous sliding has been observed. The fully softened shear strength may be obtained from the peak strength of the same clay when it is remolded and tested to determine the normally consolidated shear strength (Skempton, 1970).

The critical design situation for softer, normally consolidated soils is reversed. An increased shear stress temporarily induces positive excess pore-water pressure, which causes the soil to fail at a lower shear stress. Therefore, for these soils, short-term stability is critical. For short-term stability, the peak undrained shear strength as determined from laboratory triaxial shear tests or direct shear tests is used for intact soils. The direct shear test is often used to determine strength properties along a bedding plane or a zone of weakness.

FAILED SOIL

Residual strength parameters are appropriate for design of slopes having previously sheared or highly fractured soils. The residual parameters are generally determined by using direct shear tests with large amounts of accumulated deformation along the shear surface (Chandler and Skempton, 1974). For preexisting landslides where the geometry of the failure surface and ground-water conditions are known, a back calculation of strength parameters that assumes a factor of safety of 1.0 can be used as a check on laboratory test data.

On small projects that do not justify the cost of shear strength testing, shear strength parameters are estimated on the basis of a correlation between plasticity characteristics and residual shear strength. Laboratory residual strength data versus liquid limit for Potomac Formation clays are shown in figure 5. The rather large scatter probably results from the wide variations in clay content and clay mineralogy in the test specimens (Lupini and others, 1981).

GROUND-WATER PRESSURES

The potentiometric level used for design is generally the highest recorded during the investigation. If levels are not measured during an unusually wet weason or if some alteration of the observed potentiometric level is expected as a result of construction, then some conservative estimate must be made. Where fractured sediments are present, it is assumed that surface water can fill the fractures throughout their depths and develop hydrostatic pressures.

The potentiometric level used to back calculate strength parameters is the lowest level recorded during observed movement. This assumption permits calculating the lowest residual strength for continued failure. Design of corrective measures is based on using the minimum shear strength parameters and the highest potentiometric levels anticipated after construction.

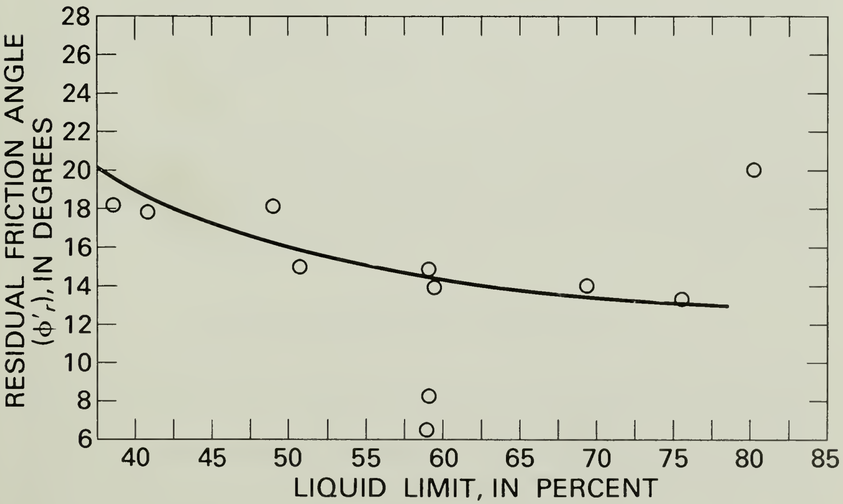

FIGURE 5.—Laboratory residual strength data for Potomac Formation clays.

ANALYSIS AND SELECTION OF FACTORS OF SAFETY

The accuracy of any slope stability analysis is highly dependent on the capability of the analytical method to model the field situation accurately. The analysis should account for factors such as changes in ground-water levels, seepage forces, tension cracks, the geometry of the failure surface, and the influence of weak planes. The authors generally use Bishop's (1960) Modified Method for analysis of circular arc failures and the Morgenstern and Price (1965) or Janbu (1973) method for noncircular surfaces. All of these methods evaluate the influence of the factors previously mentioned on the analysis. The wedge method can be used when a planar shear surface exists; this method, however, is generally conservative and yields a factor of safety on the order of 15 percent higher than that of the previously mentioned analyses (Law and Lumb, 1978).

To analyze a cut slope or embankment in the clay-rich Potomac soils (which have a history of retrogressive-type failures), the authors use a failure analysis similar to the one developed by the National Research Council of Canada (Law and Lumb, 1978). This type of limit equilibrium analysis divides the soil mass into a number of slices in which the conditions for localized failure are considered. Strengths of the various slices are chosen according to the procedure outlined in figure 4. If localized failure is possible, a postpeak strength equal to the residual shear strength is assumed; thus, redistribution of interslice forces and the possibility of further localized failure are introduced. This procedure is continued until an overall factor of safety, expressed in terms of reserve strength, can be assessed.

The selection of an appropriate factor of safety for design is dependent on several factors. Among these are the degree of uncertainty of the slope conditions, the cost of construction or repair, and the hazards of slope failure to human life.

Generally, a safety factor of 1.25 is considered adequate for the design of small projects in which the probability of loss of life and property is small. A factor of safety of 1.5 is generally required for projects potentially involving loss of life or large property loss; this factor may be increased to 2.0 in high-risk situations.

CONSTRUCTION PROCEDURES

Various remedial construction methods are used for correcting slides in Potomac Formation soils. These methods include excavation-replacement schemes, various retention systems, berm counterbalances, and others. Irrespective of the construction method used, it is very important for the design to incorporate procedures that isolate the slope from ground water and surface water as much as possible.

It has been the authors' experience that subsurface drainage systems have limited applicability in Potomac Formation clays and silts. The residual shear strength of these soils is generally too low to stabilize failed slopes economically by using improved subsurface drainage only. Drainage systems are quite useful, however, for stabilizing permanent slopes or portions of slopes where failure has not yet reduced the strength to the low residual values.

Relatively few guidelines have been published for the design of subsurface drainage systems to stabilize slopes. A recent model study has provided some generalized design charts for establishing the spacing and length of subsurface drains to obtain an incremental increase of the safety factor (Kenney and others, 1977). However, because of the highly fractured nature of many Potomac Formation clay-bearing soils and their low intact permeabilities, generalized design charts may not be applicable. It has been the authors' experience that the locations of springs or seeps encountered during construction often change as the landslide moves. Drainage systems constructed into seeps often yield little or no water.

The authors feel that slopes having observed drainage problems require some kind of drainage correction even though the apparent success of such measures may be questionable. The authors typically use trench drains placed 30 to 50 ft (11–15 m) apart and constructed as deep as practical. In many places, the drains are connected to cutoff drains parallel to the slope at the top and bottom of the slope.

The critical design conditions for the completed slope require the slope to be marginally stable (a factor of safety equal to about 1.1) in case the subsurface drainage system should fail completely. This requirement is considered prudent because existing ground-water conditions, as well as the ground-water conditions that cause failure, often cannot be defined accurately during the investigative phase. Furthermore, clay- and silt-rich Potomac Formation sediments are easily eroded in many places and can clog drainage systems. To minimize the clogging, subsurface drainage systems are normally used in combination with filter fabric.

It is important that a geotechnical engineer familiar with the design be at the site during construction to ensure that assumptions used for design are valid. Problems can arise when clay-rich soils are exposed to water for prolonged construction periods and consequently soften and cause localized failure. Other important items that require careful field inspection include proper alinement and installation of piling or tieback systems, proper compaction of embankment soils, and proper placement of subsurface drains to intercept surface and subsurface water.

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ENGINEERING GEOLOGY AND DESIGN OF SLOPES FOR CRETACEOUS POTOMAC DEPOSITS IN FAIRFAX COUNTY, VIRGINIA, AND VICINITY

EVALUATION OF PERMANENT CUT SLOPE STABILITY IN POTOMAC FORMATION DEPOSITS

By JAMES J. SCHNABEL[1]

CONTENTS

ILLUSTRATIONS

[1]Principal and professional civil engineer, Schnabel Engineering Associates, Consulting Geotechnical Engineers, Bethesda, Md.

ABSTRACT

Evaluation of slope stability in Potomac Formation deposits is complicated because of variable geologic settings and soil strength parameters, the irregularity of the shear surface, and other factors not fully understood at this time. Stability conditions are evaluated by traditional methods of investigation and analysis, coupled with soil mechanics laboratory testing to estimate the in-situ strength of potential failure planes. Selection of soil strength parameters is based in part on geologic factors such as the presence of water-bearing layers within potential slide zones, the intensity and orientation of fractures and bedding-plane partings, the presence of slickensides on fractures, clay consistency, the thickness of clay-rich beds, the plasticity characteristics of clays, and the stability of natural slopes at the site. Evaluating the importance of each of these factors requires a careful evaluation of the geologic setting at each site.

Surface and subsurface drainage systems are used to improve the stability of most cut slopes. Factors of safety used in design are based on the consequences of land slippage, as well as on the amount and sophistication of field studies and laboratory testing, and on the geotechnical engineer's confidence in his evaluation of stability conditions.

INTRODUCTION

The geotechnical engineering evaluation of slope stability conditions in Potomac Formation deposits is particularly difficult because of variable geologic settings and soil strength parameters, the irregularity of the shear surface, and other factors not fully understood at this time. Engineers are required to evaluate stability conditions by traditional methods of investigation and analysis, coupled with soil mechanics laboratory testing to estimate the in-situ strength of potential failure planes. Selection of soil strength parameters must take into account geologic factors such as shear zones and fractures, previous landsliding, and previous experience with slope instability in a similar geologic setting. Surface and subsurface drainage systems are used to improve the stability of most cut slopes. Factors of safety used in design are based on the human and economic consequences of land slippage, as well as on the amount and sophistication of field studies and laboratory testing, and on the geotechnical engineer's confidence in his evaluation of stability conditions. Design procedures for permanent cut slopes discussed in this paper have been developed over the years by Schnabel Engineering Associates of Bethesda, Md.

GEOTECHNICAL INVESTIGATION

Geotechnical investigation for slope stability in Potomac Formation sediments requires careful evaluation of site conditions. An investigation for a cut slope in a hillside that has no apparent actively moving slides should meet the following minimum requirements:

1. The pattern of subsurface layering and intertonguing of sediments having different engineering properties must be clearly established. Texture and strength are the principal properties used to define this pattern, in combination with observations of sheared or fractured zones. Field tests used to define this pattern are primarily soil test borings made with the Standard Penetration Test (SPT) (American Society for Testing and Materials, 1978). SPT tests are made in the borehole at vertical intervals less than 5 ft (1.5 m) apart, except in zones suspected to have been sheared by previous landsliding, where tests are usually continuous. Borings should be at critical locations on a slope, especially where slopes are steepest or where preexisting landslides are suspected. Cuts greater than about 30 ft (9 m) high should have borings located along the slope at the top, at the midpoint, and at the base to define subsurface conditions adequately. For cuts less than 30 ft (9 m) high, one boring at the top and one at the base are usually sufficient to provide necessary data. It is important to delineate thin layers during this investigation. SPT spoon samples can detect sand layers that are only 2 to 3 in. (5–7.6 cm) thick; although these sand layers are quite thin, they may be water bearing during wet seasons.
2. SPT blow count values are used to evaluate the in-situ compactness of granular soils and the consistency of cohesive soils and to guide the undisturbed sampling program. SPT values are also helpful for locating weak shear zones within otherwise strong deposits; abruptly lower penetration resistances are commonly encountered in these shear zones. SPT blow count values of 15 per foot or less are often recorded in old slide debris, whereas the underlying undisturbed materials commonly have values ranging from 20 to 70. Relatively low SPT values of 10 or less in cohesive soils may also indicate the presence of water-bearing sand seams interlayed with weaker clay layers and weathered materials.
3. Water observation wells should be installed in borings in the upper regions of a slope to allow long-term observations of groundwater levels. Perforated plastic pipe surrounded by sand is normally suitable for completing these wells.
4. Undisturbed samples 3 in. (7.6 cm) in diameter are taken in fine-grained soils, where sampling is possible. Sampling normally can be done by pushing a Shelby tube into clays and silts that have SPT values of less than 20 blows/ft. The tubes are sealed and delivered to a soil mechanics laboratory for testing.
5. Test pits should be dug to supplement data obtained from the soil test borings and to allow undisturbed sampling of soils that cannot be sampled by normal sampling methods. Test pits excavated

as deep as 12 ft (3.7 m) with a small backhoe are usually sufficient. Pits should be logged by an experienced engineering geologist who will note geologic details that cannot be detected by examining samples from test borings. These details include the jointing pattern, the presence of fractures and slickensides, and the thickness and orientation of strata. Any water flowing into the pit also should be noted. Block specimens of soils also may be removed from the pit for laboratory testing.

6. Experienced geotechnical engineering personnel must make at least one trip to a site to check for evidence of previous instability. This site visit is of paramount importance and should be conducted with the aid of a topographic map and other available geologic aids, such as detailed maps or aerial photographs. The geotechnical engineer must investigate variations in the normal topography to ascertain if they are caused by previous landslides, erosion, or the activity of man. This investigation should be conducted after field investigation data are available, when there is a clear understanding of subsurface conditions.

Additional borings and sampling may be required if the site investigation reveals possible previous landsliding, jointing, or tectonically induced shearing. Such previously failed areas must be delineated accurately enough to determine localities that may need special treatment.

During site visits, particular attention should be paid to tilted trees and to tree trunks bent at the base, which can be indications of creep or landsliding. Seasonal streams and permanent and seasonal springs must be located and mapped because they are potential discharge sites of a locally high ground-water table. Wet-ground vegetation should be noted because it is an indicator of springs or a high ground-water table.

Where active landslides are present, slope inclinometer systems are installed; piezometers are also placed in some slides. Slope inclinometer data permit the recognition and identification of shear zones in active landslides. Readings taken on rapidly moving landslides 2 to 3 days after installation are commonly sufficient to determine the zone of shearing. Longer term readings are required for locating shear zones in more slowly moving soil masses.

Piezometers are used to determine artesian pressures in confined granular layers. These piezometers can be made from well points packed with sand at the tip and then packed with bentonite above the sand. Readings should be made on piezometers during wet and dry seasons to determine the relationship between precipitation and piezometric levels within confined layers.

GEOTECHNICAL EVALUATION

Geotechnical evaluation requires determining the critical potential failure surface and the shear strength of the soil along this failure surface and applying various limiting equilibrium techniques to assess the overall stability of the soil mass. Slope stability evaluation proceeds in one of two modes, depending on the presence or absence of either landslides or sheared zones. The same methodology is used for assessing ancient or recent slides.

For an existing landslide, the analysis of a cut slope should use the geometry of the existing failure surface. For previously stable, unfailed slopes, investigation for a proposed cut slope commonly uses a trial-and-error procedure to determine the critical failure surface.

The Simplified Bishop (1960) Method is used to evaluate slope stability where circular slip surfaces can be used in the analysis. This method is readily adapted for computer analysis, and various trial circles are calculated until a minimum factor of safety is determined.

The Simplified Bishop Method assumes that the forces acting on the sides of any slice of soil have zero resultant in the vertical direction, and static equilibrium therefore is not satisfied. Lateral and shear forces on the sides of slices thus are eliminated from consideration, but the method still permits inclusion of seepage forces. The Simplified Bishop Method yields a factor of safety within 2 to 7 percent of the more accurate, statically satisfied Morgenstern and Price (1965) Method. The Simplified Bishop Method is preferred over the statically correct methods because it is simple.

Stability calculations for surfaces other than circular surfaces of sliding are evaluated by the Modified Swedish Method (U.S. Army Corps of Engineers, 1970). This method is used for less than 20 percent of stability calculations because most calculations are for unfailed slopes.

SELECTION OF SOIL STRENGTH PARAMETERS

It is of major importance to select soil strength parameters that fit the conditions anticipated at failure. Different strength parameters are normally used for stability calculations, depending on the presence or absence of a shear surface (such as a landslide rupture surface or a fault shear zone) in the proposed cut.

For the test program, consolidated drained direct shear tests using three normal loads are conducted on specimens of undisturbed soil 2.5 in. (6.35 cm) in diameter. Strength testing is done at a very low strain rate, and the following sequence is used:

1. A specimen is carefully extruded from a thin-walled sampling tube 3 in. (7.62 cm) in diameter and trimmed to fit a direct shear box ring 2.5 in. (6.35 cm) in diameter.
2. The specimen is cut horizontally in the 2.5-in. (6.35 cm) shear box by using a wire saw.
3. A vertical pressure (normal load) is applied to the specimen, which is then allowed to equilibrate (consolidate). The vertical pressure used is normally arbitrarily taken as 1.00 kip/ft^2 (ksf) (4.88 kgf/cm^2) for the minimum pressure; increments of 1.00 to 1.50 ksf (4.88–7.32 kgf/cm^2) are used for the second and third normal stresses.
4. After consolidation is essentially complete, the specimen is sheared while the normal load is still being applied; a controlled strain rate of about 2 percent per hour, intended to allow complete drainage during shearing, is used. The shearing is stopped at a horizontal displacement of 0.50 in. (1.27 cm).
5. While the normal load is still being applied, the specimen is reversed in the testing machine, and the top and bottom segments are returned to their original positions.
6. The sample is again sheared along the failed surface, as described above, in the same direction as the original shearing. This procedure is repeated until the total cumulative shear movement along the failure surface is at least 1.50 in. (3.81 cm). A plot is made of the cumulative displacement of the specimen versus applied shear stress for all three normal loads, as figure 1*A* illustrates.
7. Two more specimens of similar material are tested as described above under different normal stresses.
8. Normal stress versus shear stress data are plotted to determine the angle of internal friction and the cohesion intercept of the Mohr-Coulomb envelope, as figure 1*B* shows. The minimum shear strength after 1.50 in. (3.81 cm) of displacement is plotted versus normal stress, and a line is fitted between the points.

The method described above will usually result in a shear strength envelope that passes through or slightly above the origin. The test method permits determination of the "residual shear strength," which is defined as the lower limit of the drained strength envelope when cohesion is reduced to zero.

The slope of the shear strength envelope (ϕ_r') obtained by using this method gives a measure of the residual friction angle that can be used for slope design, but values of ϕ_r' obtained from laboratory testing are adjusted to conform to experience with similar slopes in similar geologic settings. The measured cohesion also must usually be adjusted downward for the evaluation of slopes where landslide surfaces of rupture or fault shear zones are absent. This adjusted cohesion, the

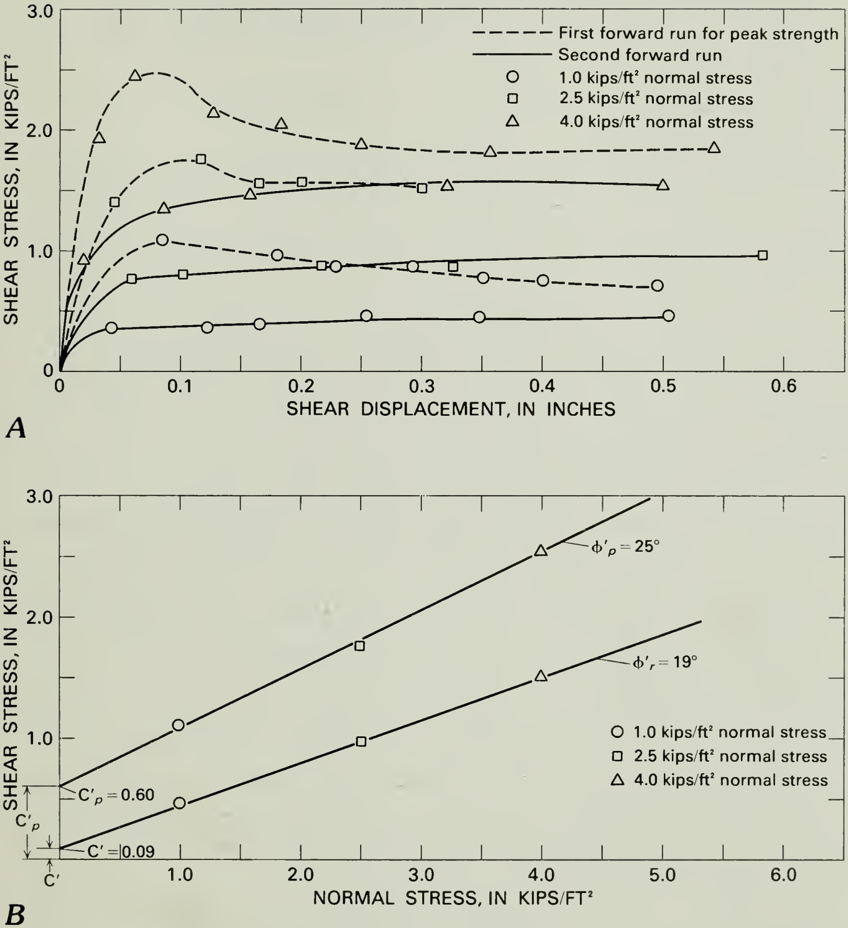

FIGURE 1.—Typical consolidated drained direct shear test results for a clay-rich Potomac Formation sediment having precut shear surfaces. *A*, Shear stress versus displacement for different normal stresses. *B*, Peak and residual shear strength envelopes. ϕ_p', peak friction angle along precut surface; ϕ_r', residual friction angle; c_p', cohesion for peak strength curve; c', cohesion for 1.5-in. displacement curve. Points used for determining maximum shear strength (upper curve) were taken at the peak of their corresponding stress displacement curves. Points used for determining residual shear strength (lower curve) were taken after 1.5 in. of total displacement.

"available cohesion," is reduced to a minimum to account for the presence of joints and other features that allow penetration of water into the soil and thus cause further long-term weakening of clay-rich soils. This weakening is believed to be associated with stress release caused by cutting, which allows joints and fractures to open. The mechanisms whereby strong clay-rich sediments weaken after a cut is made are not fully understood, and the strength parameters selected must be reduced to reflect the uncertainty in the amount of anticipated softening.

The effective cohesion (c') (see fig. 1*B*) recorded in the testing normally varies from 0 to 0.175 ksf (0–0.855 kgf/cm^2). For the design of slopes cut in previously stable slopes, these cohesion values are arbitrarily reduced to 0.025 to 0.050 ksf (0.122–0.244 kgf/cm^2); the reductions are based on experience gained from engineering geologic studies of Potomac Formation landslides. For the design of slopes in which no previous movement has occurred, such as first-time slides in cuts in London Clay, Skempton (1977) advocated the theory that some cohesion is present in first-time slides; this hypothesis seems to be confirmed by experience with Potomac Formation sediments (Schnabel and Grefsheim, 1982). It is recognized that direct comparisons of soil behavior between London Clay and Potomac Formation clay-rich deposits cannot be made. It is the author's opinion, however, that enough similarities exist between these clays to allow consideration of Skempton's hypothesis; that is, displacements preceding first-time slides are sufficient to cause some progressive failure, which reduces the available shear strength toward a "fully softened" value, but movements are not sufficient to reduce strength to residual values. This strength lies between the lower residual strength and the peak strength and possesses a small cohesion. The author's observations of slope failures in the Potomac Formation seem to confirm that the clay structure does not break down completely and behave as a purely frictional material.

The angle of internal friction for first-time slides in clay-rich Potomac Formation sediments is usually taken as the measured residual friction angle, ϕ_r', determined by using the test method described, although some adjustment to this value may be necessary. Laboratory testing of precut samples provides an estimate of the minimum or residual strength value applicable after initial movements. Adjustments to the test values can be made depending on whether a first-time slide condition is being analyzed, the degree of confidence in a first-time slide condition, experience with similar clay soils, and evaluation of the local geologic factors. For this adjustment and for the final selection of design strength parameters, it is useful to utilize back-analysis calculations for full-scale existing landslides in soils of similar classification and geologic conditions (Schnabel and

Grefsheim, 1982). Judgments concerning shear strength parameters can best be formed by careful study of the geology and the ground water of a site, back-analysis calculations of previous slides, and test data. The influence of various geologic factors on the selection of appropriate strength parameters is as follows:

Geologic conditions	*Effect on strength parameters (ϕ', C') selected for design*
Presence of water-bearing layers within potential slide zones, possibly only seasonal flow.	Use slightly lower strength parameters.
Presence of fractures	Use lower strength parameters with higher intensity of fracturing.
Orientation of fractures and bedding planes.	Use lower strength parameters where fractures and bedding parallel potential slide surfaces.
Presence of hard, thick, unfractured or weakly fractured clay beds having no granular layers or lenses.	Use higher strength parameters.
Presence of lower plasticity clayey silt soils or high percentage of granular grain sizes.	Use higher strength parameters with either factor.
Existing natural slopes relatively steep.	Use higher strength parameters.
Absence of measurable ground water within potential slide zone.	Use higher strength parameters.
Overall consistency of clay-rich soils relatively low as measured by SPT blow counts.	Use lower strength parameters.
Presence of troughs filled with permeable soils or a high ground-water table.	Use lower strength parameters.

Design shear strength parameters developed by using the test procedure outlined herein vary considerably, apparently owing to the plasticity of the specimen tested, clay mineralogy, sand content and gradation, details of testing, and interpretation of test results. The range of parameters experienced for the soil types noted is as follows:

Highly plastic clay, CH (liquid limit 70 percent), small percentage of sand.	$\phi_r' = 8°$ to $19°$ $c' = 0$ to 0.050 ksf (0–0.244 kgf/cm^2)

Plastic clayey silt MH (liquid limit 55–65 percent)	$\phi_r' = 14°$ to $22°$ $c' = 0$ to 0.050 ksf (0–0.244 kgf/cm²)

Testing of lower plasticity clay (CL) appears to yield strength parameters similar to those of CH specimens.

Normally, minimum residual friction angles (ϕ_r') of 12° to 15° are used for Potomac Formation clays or clayey silts, cohesion being zero if previous movements are suspected. A nominal cohesion of 0.025 to 0.050 ksf (0.122–0.244 kgf/cm²) may be used in some cases if there is no evidence of previous sliding.

For evaluating failure along existing shear surfaces or where existing slide surfaces are suspected, it is customary to assume that cohesion is zero on the failure surface. The friction angle is taken as the laboratory-measured residual friction angle. Considerable judgment should be exercised in selecting the parameters used for design, however; factors such as back analysis of slides at the site should be taken into account because the laboratory test method often results in values that are unrealistically high or low.

The Potomac deposits in Fairfax County and vicinity contain many extensive, relatively compact silty and clay-bearing sands. These soils are generally quite stable on natural slopes and do not weaken as severely as the more plastic sediments. The following effective strength parameters are commonly used for design:

SM, silty sand, compact.	$\phi' = 30°$ $c' = 0$
SC, clayey fine sand, compact.	$\phi' = 30°$ $c' = 0$ to 0.050 ksf (0–0.244 kgf/cm²)

Laboratory testing should always be performed to evaluate parameters at sites that are marginally stable and at sites where slopes are relatively high or where life or property may be endangered.

FACTORS OF SAFETY

Uncertainties that are commonly present in slope evaluation may be categorized as follows:

1. Geologic features such as the dip of strata, water-bearing strata, and the presence of fissures, slickensides, and old slip surfaces or fault planes.
2. Soil strength parameters along the surface of sliding.

3. Piezometric levels within the slope and variations in piezometric conditions with time.
4. Destabilizing forces that may act on the slope in the future.
5. Locations of critical surfaces of failure.
6. Simplifying assumptions used for analysis.

The geotechnical engineer must consider these uncertainties when he is selecting a reasonable factor of safety for design. A factor of safety of 1.5 is normally required for design of a permanent cut slope where failure would endanger life or property or would have serious adverse effects. For analysis of stability of existing slopes, a value of 1.2 is usually acceptable, if available information is adequate and if conservative strength parameters are used. Low factor-of-safety values may be used for areas where there is only a possibility of minor slippage or where adjacent natural or cut slopes have remained stable. Higher factors should be used where structures are located on the slope or near the crest of the slope (that is, less than half the slope height from the crest). Considerable judgment must be exercised in selecting a factor of safety consistent with the various uncertainties noted above.

It should be realized that present methods of investigation, testing, and analysis may fail to disclose geologic details that can lead to failure of a slope. Slopes in clay and silt materials of the Potomac Group may prove unstable regardless of the numerical factor of safety that may be developed from analysis.

SLOPE PROTECTION MEASURES

Surface water and ground water should be drained from all major cut slopes. Stability is enhanced by reducing the possibility that water will soften the soil and by increasing the available shear strength for resisting sliding on the critical sliding surface. Groundwater pressures can have a major influence on the effective shear strength along the surface of failure because a major portion of that strength is derived from the normal intergranular forces in the soil. Strength reductions caused by a high water table and high pore pressures can be decreased by using subdrainage and surface drainage to control the potentiometric levels in the cut slope. Subdrainage is also used to drain seasonal aquifers within the slope.

Often, surface water infiltrates into weathering fractures to depths of 2 to 5 ft (0.6–1.5 m) below the surface of Potomac Formation materials. This perched water is caused by precipitation and can best be drained by a subdrainage system placed about 6 to 8 ft (2–2.5 m) below final grade. This subdrainage should be located immediately above the crest of the cut slope, particularly in low areas where surface water can accumulate.

Subdrainage is usually accomplished by placing perforated or slotted plastic pipe 4 to 6 in. (10–15 cm) in diameter and having a protective filter in trenches spaced vertically at intervals of about 10 ft (3 m) down the slope. A typical subdrain is shown in figure 2. Except in the upper 2 ft (0.6 m), the trench is filled with pervious sand or stone to intercept water flowing in fractures, joints, and more permeable layers. Pipes are placed at a minimum slope of 0.05 in./ft and should lead to catch basins for disposal. Cleanouts should be placed to allow periodic flushing of subdrains. Drains should be constructed to avoid freezing and to allow monitoring of flow.

Drains in previously unfailed slopes are placed parallel to the final grade contours and are installed in benches 5 to 6 ft (1.5–2 m) wide to permit maintenance and access. Drains should always be placed near

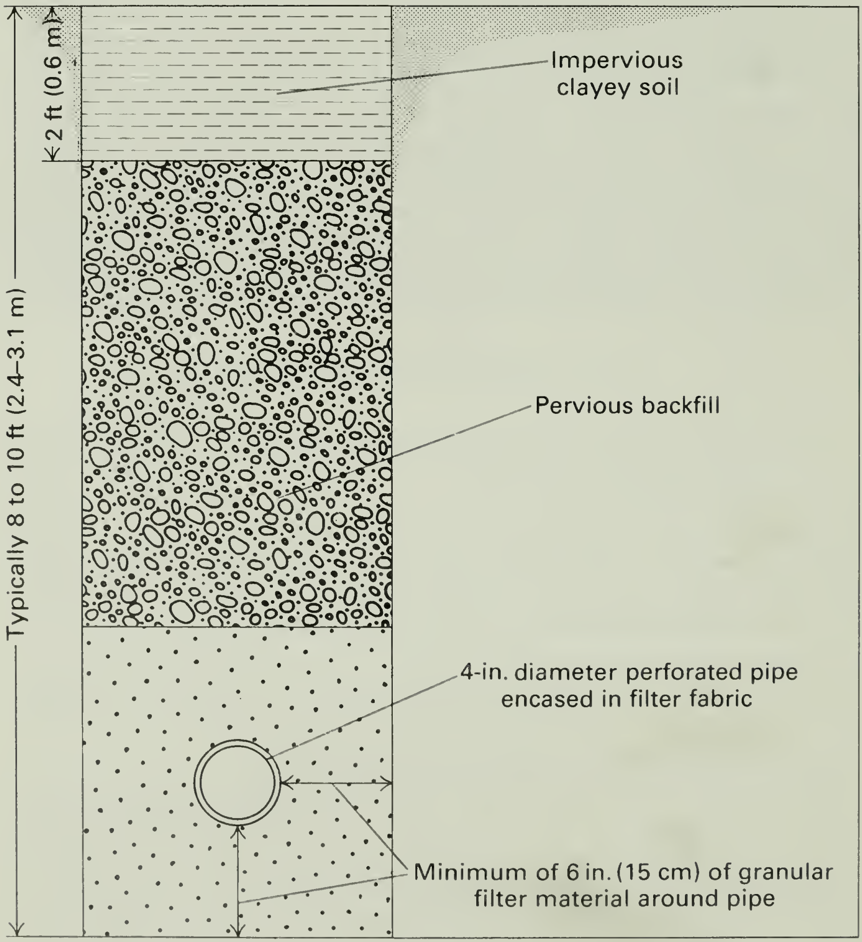

FIGURE 2.—Typical subdrain.

the crest to intercept seasonal perched ground water. In many places, a shallow drain is placed at the toe of the slope to minimize the possibility that ground water will come to the surface. A typical drainage cross section is shown in figure 3.

Where spring activity is localized or the ground-water table is deep, it may be desirable to install a perpendicular drainage arrangement into the slope, as figure 4 illustrates. This drainage system can be installed from the toe of the slope where seepage is observed. Normally, the depth into the slope is controlled by the excavation equipment available and seldom exceeds about 20 ft (6 m). The drainage pipe and porous material are placed in such a way that workmen do not have to enter the excavation.

Subdrains in previously failed slopes are oriented in a general downslope configuration, as figure 5 shows. Where additional movement can be anticipated, the subdrains are often positioned at an

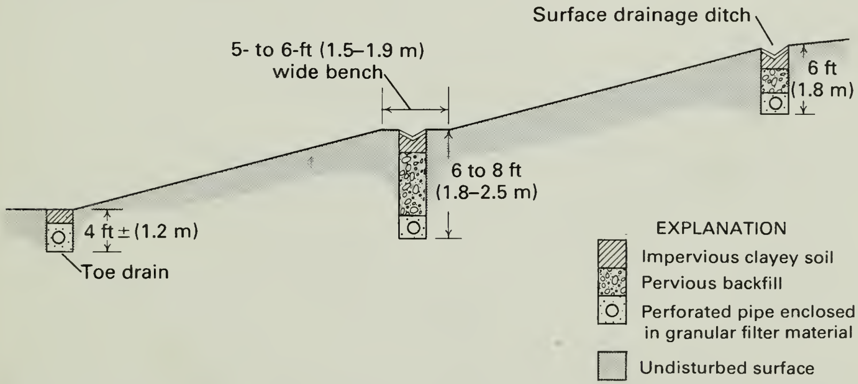

FIGURE 3.—Typical surface and subdrainage detail showing drainage parallel to slope. Details of subdrain shown in figure 2.

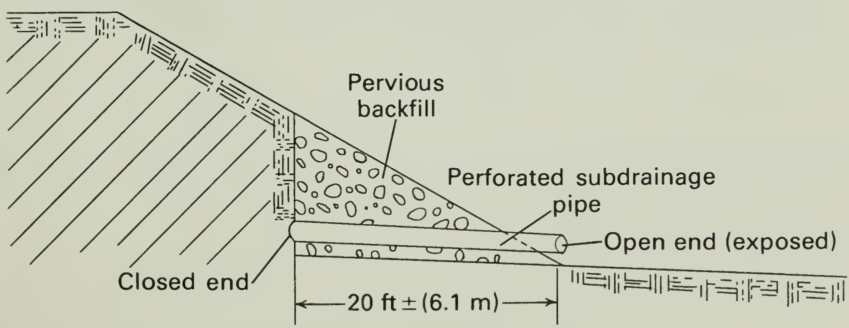

FIGURE 4.—Typical subdrainage detail showing drainage perpendicular to slope.

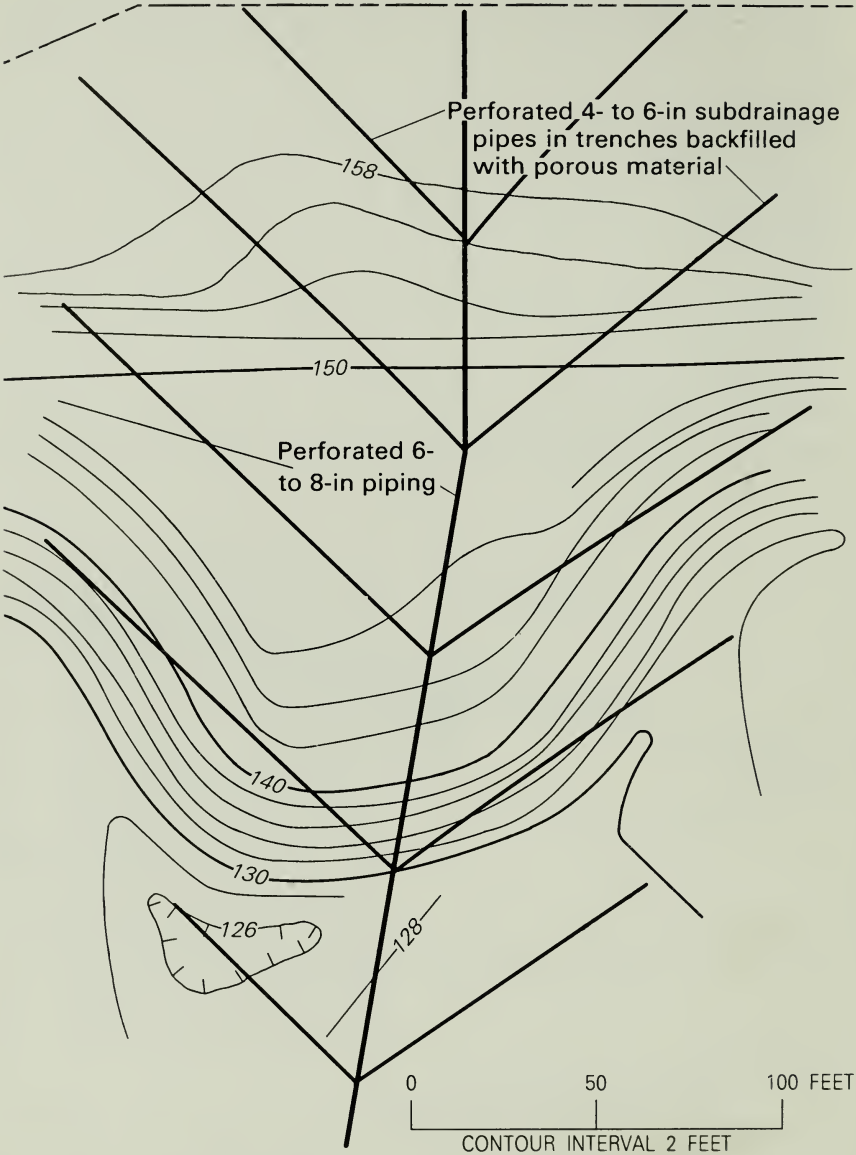

FIGURE 5.—Typical subdrainage for previously failed slope. Contour elevations in feet.

angle of about 45° to the slope in a herringbone pattern. On actively moving slides, the subdrains are located perpendicular to the contours. Horizontal spacing of drainage pipes is usually about 25 to 40 ft (7.5–12 m), depending on the permeability of the soil, the slope angle, existing slide movements, and the amount of water present. Subdrains in failed slopes are usually 8 to 15 ft (2.5–4.5 m) deep and should extend below the shear surface to be most effective. The piezometric level within the slope is assumed to be represented by a straight line extending through the inverts of the subdrainage pipes.

Surface water is also collected and removed from the slopes, usually by means of paved concrete ditches located at the crest of cut slopes where there may be extensive runoff and on benches on the slopes. Benches are constructed on high slopes to allow access by maintenance equipment in case of local instability. Slopes are seeded or sodded as soon as possible after final grading to avoid erosion of soil and possible clogging of drainage facilities.

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ENGINEERING GEOLOGY AND DESIGN OF SLOPES FOR CRETACEOUS POTOMAC DEPOSITS IN FAIRFAX COUNTY, VIRGINIA, AND VICINITY

A SUMMATION AND CRITIQUE: A REVIEW OF PROCEDURES FOR DESIGN OF PERMANENT SLOPES ON THE POTOMAC FORMATION AND SOME RELEVANT FIELD OBSERVATIONS

By STEPHEN F. OBERMEIER

CONTENTS

ABSTRACT

The design engineers who authored the papers in this volume all agree that residual shear strength parameters should be used to evaluate the possibility that failure will be reinitiated along the surfaces of rupture for preexisting landslides. These residual parameters may be determined by laboratory testing, but laboratory data should be verified by back analysis of the landslide if recurrence would have serious conse-

quences. The engineers also agree generally that low shear strength parameters should be used for slope design on intensely fractured clay-rich sediments. Swanson and Jones advocate using residual strength parameters, whereas Schnabel may also include a "reduced cohesion" component of strength.

The designers also agree that effective shear strength parameters higher than residual shear strength parameters should be used to evaluate the stability of slopes where there are no preexisting shear zones and in places where the soils are intact and not intensely fractured. Failure is assumed to take place in drained shear. Swanson and Jones advocate using "fully softened" parameters. The strength parameters used by Schnabel seem to be about the same, although direct comparison is not possible.

This author believes that the geotechnical properties of natural slopes are fundamentally different from those of cut slopes and that these differences are important to understanding slope instability in the Potomac Formation. Natural slopes commonly have many weathered, weakened zones and fracture surfaces in the uppermost 20 ft (6 m), the main region traversed by the surface of rupture. The shear strength is probably controlled by soil in highly varying states of drainage and consolidation, and determination of exact strength parameters is almost impossible. However, the "fully softened" strength is probably a realistic value for the great majority of slopes; slopes that have unusually adversely oriented fractures, that have experienced significant creep, or that have clays whose tendency to swell is strong are exceptions. Unweathered, less intensely fractured clay-rich sediments are probably stronger than those that are more weathered.

Much additional research is needed to better evaluate the slope behavior of Potomac Formation sediments. The highly variable nature of the sediments makes it necessary to pay great attention to geologic details.

INTRODUCTION

This paper summarizes the preceding two papers' most salient points regarding permanent slope design in materials of the Potomac Formation. As those papers state in many ways, the behavior of many slopes cannot be evaluated with any great degree of precision where clay-rich layers of Potomac Formation sediments are present. As a consequence, the design engineer must try to determine realistic bounds for the shear strength parameters and ground-water pressures. This author attempts to compare the methodologies that the consultants who have contributed to this volume use to determine those bounds and then discusses the relationship of their design techniques to the geologic setting.

In addition, some fundamental problems of the mechanics of slope failure are discussed, as well as how these problems bear on slope design. Finally, some research needs are suggested.

SLOPE STABILITY EVALUATION OF PERMANENT CUT AND NATURAL SLOPES

Even though Schnabel specifically addresses the problem of cut slope stability, he notes that the same basic procedure is used to evaluate natural slopes. Neither paper makes any fundamental distinction between the methodology used to evaluate a cut and that used to evaluate a natural slope. Both papers place major emphasis on estimates of fracture intensity and slickenside development; in addition, Schnabel uses the overall consistency of the sediments in the slope to guide his selection, lower parameters being used for weaker materials. Both papers agree that design shear strength parameters and ground-water pressures are major influences on slope stability evaluation.

PARAMETERS FOR EXISTING LANDSLIDES AND FOR INTENSELY FRACTURED AND (OR) SHEARED SEDIMENTS

For existing landslides, both papers agree that residual shear strength parameters should be used to evaluate the possibility that failure will reinitiate along the surface of rupture. These residual parameters may be determined by laboratory testing, but laboratory data should be verified by back analysis of the landslide if recurrence would have serious consequences.

There is also general agreement that low shear strength parameters should be used for intensely fractured clay-rich sediments. Swanson and Jones advocate using residual strength parameters, whereas Schnabel may also include a "reduced cohesion" component of strength. It should be noted, however, that Schnabel's laboratory-determined "residual" strength parameters may be somewhat higher than the residual shear strengths conventionally reported in the geotechnical engineering literature. Schnabel defines the "residual" strength as the shear strength after 1.5 in. (3.8 cm) of displacement along a precut shear plane. Data reported in the most recent definitive study of residual shear strength (Lupini and others, 1981) show that a minimum shear displacement of about 2.5 in. (6.3 cm) and typically on the order of 10 in. (25 cm) is required to determine residual shear strength for plastic clays. Limited test data obtained by the author on remolded specimens of Potomac Formation clays by using a repeated direct shear technique showed no cohesion intercepts on the strength envelopes.

Whether there is a cohesion intercept on the laboratory-determined strength envelope is probably irrelevant to the analysis of first-time slides in highly fractured and (or) sheared sediments because the laboratory test method does not simulate field conditions. Clay-rich Potomac Formation sediments are typically highly fractured in the uppermost 5 ft (1.5 m); these fractures commonly have rather rough surfaces, even where they are slickensided, and these surfaces are coated with washed-in clay in many places. These washed-in clays have usually been strengthened somewhat by drying. Thus, Schnabel's inclusion of a cohesion component of strength seems consistent with field observations. The only possible method for determining reasonable cohesion values uses back analyses of landslides.

This author believes, however, that extreme caution must be used in extrapolating information from back analyses. If no large body of data from previous studies of the Potomac Formation is available, it would seem judicious to use Swanson and Jones' method. The approach used by Schnabel, which considers many types of data (including laboratory strength data, Atterberg limits, landslide geometry, fracture intensity and orientation, and other geologic factors) obviously can be best used whenever the designer has accumulated much experience.

PARAMETERS FOR FIRST-TIME FAILURES IN "INTACT" SEDIMENTS

Both papers agree that effective shear strength parameters higher than residual should be used to evaluate the stability of slopes where there are no preexisting shear zones and in places where the soils are intact and not intensely fractured or jointed; failure is assumed to take place in drained shear. Swanson and Jones advocate using fully softened parameters. To justify using these parameters, much emphasis is placed on Skempton's (1970, 1977) research on the London Clay. In a similar vein, Schnabel advocates use of his "residual" strength parameters and inclusion of a cohesion component of the shear strength. For any given sample, it is not possible to compare directly the strength parameters that would be obtained by using Swanson and Jones' test method and those that would be obtained by using Schnabel's method.

Both methods described above are, in this author's opinion, empirical. The technique using fully softened strength parameters is the only one that approaches having a well-formulated theoretical basis; however, the method as developed and advocated by Skempton (1970, 1977) applies only to nonswelling soils, to soils having closed joints (joints so tight that ground water or surface water does not readily penetrate through them), and to slopes that do not fail because of seasonal variations in rainfall. Many of these conditions are not met in natural or cut slopes of the Potomac Formation. Discussion of the applicability of the fully softened method to Potomac Formation sediments is deferred to a later section, in which this author compares what he believes are relevant geologic details of the London Clay and of the clay-rich Potomac Formation sediments.

GROUND-WATER TABLE LOCATION

Well points are generally used to monitor the ground-water table in thick, permeable soils. In critical locations where thin, confined strata are suspected to have artesian pressure, it seems wise to use the method suggested by Swanson and Jones, which uses a closed low-volume-change system for measuring pressure.

The papers seem to disagree about the effectiveness of an underground drainage system in increasing the stability of a failed slope. Swanson and Jones think that, for design, it is realistic and reasonable to assume complete failure of the system, whereas Schnabel assumes that the water table is lowered to the invert of the subdrainage pipes.

Materials in the landslide mass above the surface of rupture are commonly highly fractured and quite permeable in the upslope part. Buried drains unquestionably lower the water table there and aid significantly in stabilization. However, there is some doubt about the effectiveness of the system, especially during prolonged wet seasons. For that reason, this author believes that Swanson and Jones' suggestion that the safety factor not be less than about 1.1, assuming failure of the drains, is reasonable.

SOME RELEVANT GEOLOGIC OBSERVATIONS

This author believes that the properties of natural slopes are fundamentally different from those of cut slopes and that these differences are important to understanding the stability of slopes in the Potomac Formation. For the great majority of design problems, however, he emphatically agrees with the consultants that the same methodology can safely be used to design both natural and cut slopes. This author distinguishes cut slopes from natural slopes because he believes that both vertical and subhorizontal fractures are important to stability; he also feels that the number of fractures is not necessarily as important as orientation, continuity, degree of softening, and amount of infilling of clay along the breaks.

It should also be emphasized that most landslides in cut and natural slopes of the Potomac Formation take place during intense rainstorms or after prolonged rainy seasons. Much, if not all, of the surface of rupture commonly traverses the zone of seasonal weathering, where surface water readily flows into many of the fractures.

NATURAL SLOPES

Within approximately the upper 5 ft (1.5 m) of the ground surface are many fractures (here called surface fractures). They typically form an anastomosing network of closely spaced breaks that make numerous, more or less continuous ruptures both vertically and subhorizontally. These fractures are best developed in highly plastic clays, where they may be slickensided, and are at random inclinations varying from vertical to subhorizontal.

Beneath these surface fractures and beneath other deposits such as colluvium and upland gravels that generally cover the Potomac sediments, there may be "joints," which typically are vertical or subhorizontal (approximately paralleling existing topography) breaks. These joints are best developed and common and can be continuous for large distances laterally in the massive and thick strata of clay-rich and plastic, silt-rich sediments. Both vertical and subhorizontal joints are more widely spaced and more poorly developed at increasing depths. The maximum depth for subhorizontal joints is usually less than 20 ft (6 m) beneath the unconformity separating the Potomac from younger materials, but the vertical joints extend much deeper in some places. Figure 18 in the first paper in this volume (p. 40) is a schematic depiction of these joints. Roots grow in and along joints in some places, water seeps from some joints, infilled and chemically softened soil is present along many of the joint surfaces, and there are some "point-to-point" contacts (localized points where hard soil is in contact along the joint). In some places, the joints are open as much as 0.1 in. (0.25 cm).

On the basis of numerous field observations, this author believes that the infilled and chemically weathered material along subhorizontal joints is commonly less consolidated than normal under the present overburden and that much of the overburden load is carried by the point-to-point contacts. He also believes that there has been enough small horizontal movement along many of the subhorizontal joints to accommodate wedging apart by tree roots and infilling by soil along vertical joints. The good match between opposite-facing irregular joint surfaces indicates that this horizontal movement is typically less than 0.25 in. (0.6 cm).

This physical setting is obviously so complex that no conceptual model can simulate precisely the mechanics of landsliding in nature. What, then, are the shear strength parameters that should be used for design problems, and what kinds of tests should be used to determine these parameters?

Some insight can be gained by using various assumptions to bound the problem. To obtain realistic lower bound shear strength parameters for first-time slides, let it be assumed that there are no point-to-point contacts along the subhorizontal joints and that the weight of the overburden is carried by the softened soil along a subhorizontal joint that is a critically oriented potential surface of rupture. If the entire surface of rupture is considered, the softened soil would be approximately normally consolidated under the total unit weight of existing overburden; an exception would be where the permanent ground-water table is at or very near the ground surface (an uncommon occurrence), in which case the softened soil would be approximately normally consolidated under the effective unit weight of existing overburden. Obviously, a landslide surface of rupture would pass along this joint with normally consolidated soil for much of its length, but would the drainage state be drained or undrained? This author believes it probable that much of the soil would fail in undrained shear, for several reasons: (1) landslides almost always take place during or soon after spring thaw, severe rainstorms, or prolonged wet seasons, (2) landslide movement is commonly rapid in the sense that the scarp at the head may move perceptibly in a day, and (3) the permeability of clays and plastic silts is very low, and the rate of dissipation of shear-generated pore pressure (ΔU_s) along subhorizontal joints must be extremely slow, especially for the deeper subhorizontal joints, where there are fewer vertical joints to relieve ΔU_s. Near the ground surface, however, the vertical surface fractures commonly intersect the subhorizontal joints at close intervals; these breaks would serve as "relief wells" and allow ΔU_s to dissipate. To summarize, at depths greater than about 5 ft (1.5 m), where surface fractures are rare, the lower bound shear strength would approximate the normally consolidated undrained shear strength; closer to the ground surface, the lower bound shear strength would be the normally consolidated drained shear strength. Where the shear surface follows the softened rupture, failure would be in direct simple shear; this shear strength is probably 20 to 30 percent lower than triaxial compression shear strength (Johnson, 1975). If factors such as the state of shear-generated pore pressure and the type of shear are considered, this lower bound shear strength through the deep joints is probably only slightly higher than the residual shear strength. This author, however, emphasizes his belief that, rather than being a common occurrence, the physical setting assumed in this lower bound problem is uncommon or rare and is most likely only on natural slopes where soils are intensely weathered. Furthermore, the irregularities of the subhorizontal joint surface would cause the strength to be somewhat higher.

Consider now a different model. Assume that the entire weight of the overburden is carried by the point-to-point contacts, which can be in one of two conditions—either crushed by the weight of the overburden or very strong and uncrushed. Because the contact area of the points is very small in either condition, any shear failure would occur primarily in drained shear. Thus, the average shear strength along any surface of rupture would approach the normally consolidated drained strength or would be higher (for uncrushed points); if the irregularities of the subhorizontal joint are considered, the average shear strength along any surface of rupture that followed the joint would be increased further and would probably become much higher.

To summarize this model, the shear strength must at least approach the normally consolidated drained shear strength.

For the great majority of slopes, however, other factors to be considered are that part of the overburden weight is carried by the softened soil and that the surface of rupture does not follow exactly a preexisting subhorizontal joint. The first factor would probably cause the shear strength to be less than the normally consolidated drained strength because of the tendency to fail in undrained shear. The second factor would cause the shear strength to exceed the normally consolidated drained shear strength and probably to greatly exceed that strength for most slopes.

It is virtually impossible, or at least impractical, to determine precisely the true shear strength parameters for a given slope. Thus, the designer is forced to make a decision based largely on judgment and previous experience. It is this author's opinion that, for the great majority of natural slopes on clay-rich Potomac Formation deposits that have never failed or have not experienced significant creep, first-time failure shear strengths approximately equal or exceed the normally consolidated drained shear strength; in rare instances, the first-time failure shear strengths approach being as low as the normally consolidated undrained shear strength, at depths greater than about 5 ft (1.5 m).

CUT SLOPES

For discussion purposes, cut slopes are excavations so deep that the possible surfaces of rupture go beneath the existing chemically weathered and infilled joints and fractures. Obviously, there is some overlap between natural and cut slopes, especially in excavations less than 10 to 20 ft (3–6 m) deep.

Using this definition for cut slopes in clays of the Potomac Formation, this author believes that the Potomac's physical setting closely resembles Skempton's (1970, 1977) research sites for cuts in the London Clay, where joints are tight and unweathered and surface water does not readily penetrate along them. Highly plastic clays of the Potomac Formation having liquid limits exceeding about 80 to 90 are an exception because they can swell and fracture and create slickensides in the material, probably within a few years after excavation in many places. For all clays except such highly plastic ones, a realistic lower bound shear strength appears to be the normally consolidated drained shear strength (normally consolidated under the overburden weight of the cut section after excavation). For highly plastic clays, this author believes that somewhat lower parameters should be used. Field observations indicate that highly plastic clays normally become intensely fractured and slickensided in the upper 5 ft (1.5 m) or more during the useful lifetime of an excavation and probably are somewhat fractured to much greater depths. Thus, the design methods used by the consultants represented in this volume seem generally consistent with geologic conditions.

SAND AND PERMEABLE STRATA

Instances have been reported where the surface of rupture passes through permeable sand-bearing strata in the Potomac Formation, where artesian pressures are high. This type of failure appears to take place almost exclusively in the area of the kaolinite-illite facies (although most surfaces of rupture in that facies certainly are in the clays and silts). Except in very unusual circumstances where there is a rather large cohesion component of strength, this author believes that failure of permanent slopes would be through the fine-grained materials.

HILLSIDE FILLS

A serious problem not addressed previously is failure of hillside fills placed over Potomac sediments. A construction practice sometimes used to combat this problem is to place better quality, stronger fill material directly on the natural Potomac Formation materials. Landslides may take place later because the fill has stopped drainage from permeable strata, fractures, and joints in the underlying Potomac Formation materials. This type of problem can be eliminated by using subdrains at the contact of the Potomac Formation and the fill material and by excavating very weak, weathered Potomac Formation materials before placing the fill.

DESIGN PROBLEM

For sites where expensive structures may be affected by landsliding or where there is a danger to people, all designers agree that very conservative strength parameters should be used for design. But the designer often faces a dilemma in planning a project such as a housing subdivision, where very conservative parameters would require so much earthwork that the project would not be economically feasible. This author believes that there are always uncertainties attendant on the use of any but the most conservative shear strength parameters in the clay-rich sediments and that these uncertainties change the design process from a matter of engineering judgment to a social or legal problem. This belief holds true especially for shallow excavations, the type most commonly used in housing subdivisions, because the most probable landslide sites would be in the weathered zones where evaluation of strength parameters is uncertain.

SUGGESTIONS FOR FUTURE STUDIES

Obviously, additional research is needed to determine appropriate design shear strength parameters. Because the Potomac Formation is so variable, both vertically and laterally, and because there are so many factors involved, such as joints, fractures, shear zones, artesian pressures, and swelling forces, extreme care would be required in selecting meaningful research sites, and extrapolating the data to other sites would be questionable in many cases. This author does not mean to discourage future studies but rather to urge that a very careful evaluation of the various factors be made before definitive conclusions are reached.

An especially fruitful area for research would seem to the effectiveness of subdrainage systems for different geologic settings. Another area would be correlations between the residual friction angle and the liquid limit for clays of the montmorillonite and kaolinite-illite facies. Such research could result in correlations much better than the one shown by Swanson and Jones (see their fig. 5), because the friction angle is very much dependent on clay mineralogy (Lupini and others, 1981).

Other items that clearly warrant research include the monitoring of long-term slope movements and potentiometric levels for both natural and cut slopes, detailed studies of the actual geometries of failure, a systematic laboratory evaluation of material strengths under the different stratigraphic and weathering conditions commonly present in the field (the evaluation should be done in terms of clay mineralogy, gradation, Atterberg limits, and consolidation state), and reconnaissance of active failures to determine how many are on man-modified slopes and how many are on natural slopes and how many are shallow and how many are deep. Also needed are better guidelines for designing slopes on small projects, where sophisticated exploration and testing costs are prohibitively expensive.

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