Slope Stability in the Washington, D.C., Area: Cretaceous Clays

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Slope stability analysis involving overconsolidated clay requires evaluating long-term effective shear strength. Strength parameters found in the laboratory, by using relatively small samples, have limited value because the applicable in-place shear strength depends on slippage along fissures or lenses within a large soil mass. Studies of full-scale landslides have therefore been used, and in-place shear strengths are estimated from back-analysis calculations. Four case studies are reported for landslides in the Washington, D.C., area. The clay soils are Cretaceous-age sediments of the Potomac Group, which are found along the East Coast of the United States from New Jersey to Virginia, but they are a noted problem only in the relatively hilly topography of southeast Washington, D.C., and the easterly area of northern Virginia immediately to the south. Data used for the back-analysis include complete soil profiles, topography at the time of failure, field locations of escarpments and uplifting, and some slope inclinometer readings. Estimates are made of groundwater level based on test borings, water observation wells, and field observations. Pore water pressures and seepage forces significantly affect the analysis, and estimated in-place shear strengths are reported for reasonable ranges of estimated groundwater and seepage conditions. The laboratory testing used includes complete soil identification and various methods of drained direct shear testing on undisturbed samples. The results of back-analysis calculations are correlated with laboratory test results and also compared with similar studies made by Skempton for overconsolidated London clay. Some differences are noted in comparison of studies of cuttings as reported by Skempton and the natural slope condition considered for some of the case studies reported in the paper. In addition, correlations are made based on Atterberg index values to further compare the findings with other available research data.

Analysis of the long-term stability of natural slopes in overconsolidated clay requires selection of in-place shear strength in the range between the peak and residual values. In his recent research involving back-analysis of slides in cuttings made in the overconsolidated London clay, Skempton (1) determined strength parameters relevant to analysis of first-time slides, which he calls the "fully softened" condition. For London clay, this value is reported to be approximated closely by laboratory test values obtained from samples that are remolded and normally consolidated. Comparisons based on laboratory testing indicate that the fully softened strength is much greater than the residual value and much less than the peak value.

To develop shear strength parameters for the overconsolidated Cretaceous-age clays of the Washington, D.C., area, laboratory shear strength tests have been conducted and back-analysis calculations have been made for existing slides. For the case studies reported, the slopes may be considered as essentially natural. Relatively small cuttings are involved; however, except for the specific cases noted, pore water pressures can be accurately estimated as hydrostatic with no excess pore water pressure.

Laboratory testing has been conducted primarily to determine the limit or residual shear strength value by using drained direct shear tests on precut samples. In recognition of the difficulties and limitations of testing small laboratory samples, existing landslides have been studied to determine in-place shear strength by back-analysis calculations. Laboratory testing has been used mainly to guide the selection of strength parameters.

It is generally accepted and demonstrated by laboratory testing that soil cohesion is greatly reduced as shear strength passes from the peak to fully softened and then to the residual state. Shear-strength parameters determined by Skempton (1) for brown London clay are summarized as follows (ϕ' = effective internal friction and c' = effective cohesion):

Case	ϕ' (°)	c' (lbf/ft^2)
Peak		
38-mm sample	20	280
250-mm sample	20	140
Back-analysis, first-time slide	20	20
Residual	13	20

For similar overconsolidated plastic clay, DeBeer (2) has used the following parameters, which he considers to represent a safe lower limit: $c' = 186$ lbf/ft^2 and $\phi' = 24.5°$.

The cohesion intercept is quite low for both the first-time slide and the residual cases, as reported above by Skempton. Generally, for applications in practice a zero cohesion value is used, and this is also recommended by the U.S. Army Corps of Engineers (3) to simplify the already complex and time-consuming direct shear test procedures necessary for developing effective residual shear strength. Accordingly, for our back-analysis calculations to evaluate in-place shear strength, we have assumed a zero cohesion intercept. Variations of effective internal friction are therefore determined. It should be recognized that values calculated in this manner may not accurately or even conservatively represent in-place shear strength for all slope stability studies. Estimated in-place shear strength as discussed in this paper should be used for conditions and applications similar to the case studies described--i.e., analysis for stability of natural or finished excavated slopes with gradients not exceeding about 3:1 (horizontal to vertical).

The research findings presented here include a description of the geology and soil properties, methods of stability analysis, a report of four case studies, and conclusions for in-place shear strength as applied to practical design analyses. Results and methods of laboratory testing are also discussed.

GEOLOGY, SOIL PROPERTIES, AND FAILURE MECHANISM

The overconsolidated clays studied here are Cretaceous-age Potomac Group outwash sediments. This is part of the general Middle Atlantic coastal plain deposits extending from the fall line to the edge of the continental shelf. Figure 1 shows this area and the approximate region of the Potomac Group outcrops extending from central Virginia north through New Jersey. These soils have been a noted problem related to slope stability, primarily along the relatively widespread belt of outcrops extending from Baltimore to northern Virginia and particularly in the areas of hilly topography in southeast Washington, D.C., and the easterly part of northern Virginia immediately to the south. The case studies of existing landslides reported here are in the latter areas of hilly topography. According to a recent report by Force and Moncure (4), the Potomac clay of these areas is relatively pure montmorillonite and montmorillonite-illite mixed layer material with a notable proportion of silt. At the

Figure 1. General geology: Cretaceous sediments of Middle Atlantic coastal plain region.

landslide sites studied, the general soil and geologic profile is as follows:

1. Pleistocene river terrace--Sand and gravel to an elevation of about 180 ft;
2. Recent colluvium--Mixture of sand, clay, and gravel about 5-10 ft in thickness along slopes extending below the original terrace sand and gravel; and
3. Cretaceous-age alluvial and deltaic sediments--Interbedded sand, silt, and clay with occasional isolated layers or lenses of gravel.

The lower stratum of Cretaceous-age sediments is the Potomac Group soil and is often referred to as "marine clay" in the local study area. Layers of clay soil are relatively persistent and continuous and usually thicker than other soil layers. The generally thinner layers of silt and sand are typically discontinuous. The subsoil profile is therefore fairly unpredictable, and the presence of permeable layers sandwiched within mostly clay soil also causes large variations in the groundwater level. Spring conditions occur frequently and, in slope stability studies, the relatively severe groundwater condition of steady seepage is often noted and must be anticipated in design. It is usually necessary to assume this severe condition along the lower portions of slopes and in considering relatively shallow slides with slip surface geometry approaching the infinite slope case.

Groundwater variations and equilibration of pore water pressures are considered by Skempton (1) to be the physical processes responsible for delayed failure of slopes. This is reviewed by Skempton in his studies of cuttings into the London clays. This process is normally initiated by reduction of overburden pressure due to excavations. Negative excess pore pressures result and, for clay soils, there is a slow return to equilibrium as pore pressures rise and effective friction is reduced. Excavated slopes can therefore fail many years after the initial excavation. For case studies of cuttings in the uniform clay soils of London, periods up to about 50 years have been noted from initial excavation to eventual slope failure.

In addition to the delayed effects of pore pressure changes, softened zones may develop due to perched water within pervious lenses and layers of silt and sand typically occurring in the Potomac Group soils. In addition, the weathering caused by alternate wetting and drying of these moderately expansive clays is accompanied by swelling and shrinkage near the ground surface. This can develop into closely spaced intersecting cracks. In this process, c' tends toward zero. In comparison with the dissipation of negative excess pore water pressures reported by Skempton, these latter processes are more significant for the case studies considered here. Besides the presence of pervious lenses for more rapid dissipation of excess pore water pressures, the cuttings considered in these case studies are relatively minor. Delay from excavation to failure has been noted to be about three years. Shrink and swell cycles, development of closely spaced intersecting cracks, and retrogressive failure with an eventual, fully developed continuous failure plane are believed to be the important delay factors.

Typical identifying properties of the Potomac clay considered here are given below:

Property	Value
Natural moisture content (%)	22-32
Liquid limit	65-80
Plastic limit	20-30
Plasticity index	40-50
Liquidity index	<0.2
Wet density (lb/ft^3)	117-129
Dry density (lb/ft^3)	96-100
Standard penetration resistance	≥15

This soil is typically the basal soil layer and the oldest sedimentary deposit in the study area. It is highly overconsolidated, generally about 10-15 ton-force/ft^2 in excess of the existing overburden pressure. The Atterberg index values, particularly the plasticity index, have been used to correlate our findings with other available research results. The clay soil studied would be described as moderately to highly plastic based on the plasticity index in the range 40-50. This should not be inconsistent with the reported predominance of montmorillonite in view of the silt proportions also reported in X-ray analysis.

GENERAL CONDITIONS AND METHODS OF ANALYSIS

Existing landslides selected for back-analysis calculations are in a predominantly clay profile, generally with only a fairly thin cover of fill, colluvium, and/or terrace sand and gravel. The failure surface is almost entirely within the Cretaceous clay considered in this study, and there is little or no effect of strength parameters for other soils. In-place shear strength is back-calculated for the estimated slope profile at the failure condition. The failures studied are all influenced by groundwater conditions, and movements are either initiated or accelerated by rainy periods.

Continuous records of groundwater level are not available, and the groundwater level at time of failure cannot be defined reliably. Because groundwater level, pore water pressures, and seepage forces are important in the effective stress analysis used, we have considered the possible extreme conditions. By using reasonable estimates of groundwater and seepage conditions for the cases studied, a fairly limited range of estimated in-place shear strength is developed that fits well with other research results for similar overconsolidated clay. The results obtained are believed to be suitable for design applications provided similar geometry is involved and similar drainage conditions are used in the slope stability calculations.

Back-analysis of shear strength is performed by using both circular failure arcs and general slip surfaces as shown on the slope sections. The back-analysis calculations are made to determine a friction angle required for equilibrium with an assumed value of cohesion of $c' = 0$. Cross sections used in the analysis are developed from detailed topographic surveys or a profile survey along the estimated axis of slope failure. Failure is either imminent or known to be progressing slowly and is generally affected by reduction of effective shear strength due to variations in the groundwater. Estimates of groundwater level are based on field measurements in borings, water-observation wells, and miscellaneous field observations as described.

CASE STUDIES

Huntington Station

A slope failure occurred in 1976 at the southeast corner of the parking lot at Huntington Station, part of the Washington, D.C., metro rail rapid transit system. The initial excavation slopes considered here were made based on proposed slopes varying from about 3.6:1 to 2:1 (horizontal to vertical). The plan and profile of this design and the eventual slope failure are shown in Figure 2. The original design included subdrainage extending to the east limit of the parking lot, but no subdrainage was used for an area of steeper 2:1 finished slopes just beyond the failure area. Redesign after the failure consisted of extending the subdrainage and regrading at 3:1 in the relatively deep-cut portion that had been designed at 2:1 slopes. As the depth of cut decreased, grading was held at the original 2:1 design. Approximate limits of the area of redesigned grading are delineated in the plan of Figure 2.

The back-analysis calculations discussed here are for the area graded at 3.6:1, where a failure occurred. This slope was excavated in a predominantly clay subsoil, as shown by the test borings VV43 and VV44 (Figure 2). The failure was apparently influenced by groundwater and surface runoff water observed during inspections of the failure area. The full range of possible groundwater conditions has been considered, from no groundwater to seepage at the ground surface. The failure area is on relatively high ground at least 40 ft above the toe of the natural slope. Accordingly, seepage forces are not included, and static water conditions are assumed in the final back-analysis.

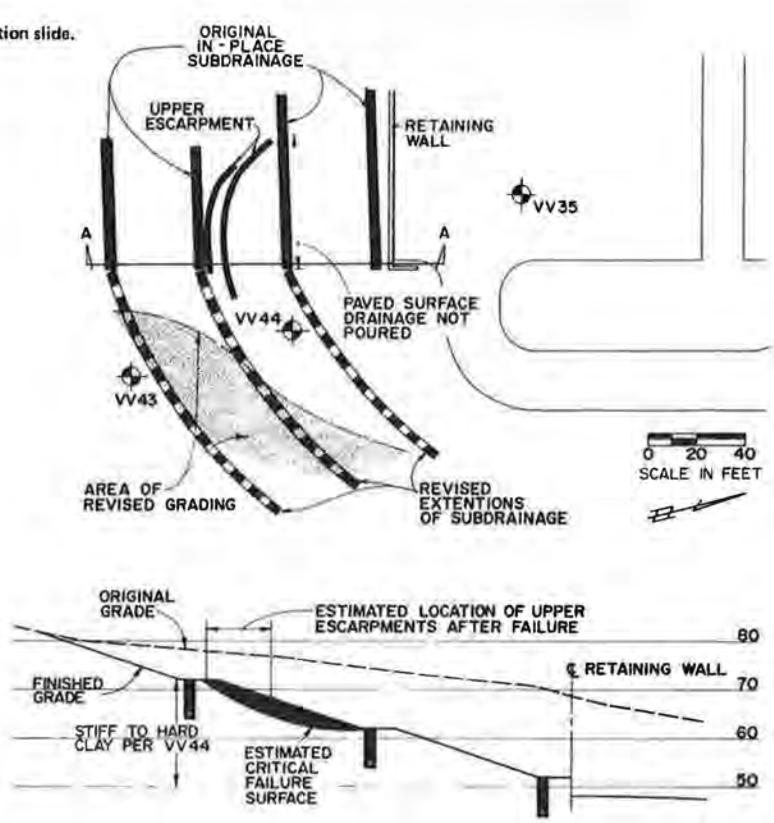
Based on an assumed cohesion $c' = 0$, the friction angle for factor of safety (fs) = 1.0 varies with the groundwater level as follows:

Groundwater Level	ϕ' Required for fs = 1.0 (°)
None	17
Intermediate ($r_u = 0.3$)	17.5
Ground surface	17.5

For the assumption of seepage based on a phreatic surface defined as $r_u = 0.3$ and ground surface, the required friction angles are 22° and 32°, respectively. These values are quite high, and an assumption of fully developed seepage forces should not be appropriate either for the back-analysis or a design with similar slope conditions.

For the intermediate groundwater condition, defined as $r_u = 0.3$, the term r_u is the pore pressure ratio. This term is defined in Figure 3. Field observations indicate perched water at least

Figure 2. Huntington Station slide.



up to a level defined by $r_u = 0.3$, and a friction angle of $\phi' = 17.5°$ should be a reasonable estimate from these data. Soil identification tests indicate a plasticity index ranging from 28 to 46 and an average of 40.

Beauregard Street

Construction of Beauregard Street along an area adjacent to the Newport Village apartments in Alexandria, Virginia, included excavation along the base of an existing slope. The original natural slope of 5:1 was cut at the toe, and the finished slope was 1.6:1. This initial excavation is believed to date back to about 1967 and the adjacent slope movements to 1971. Detailed investigations of the slope were initiated in December 1977 after various cosmetic repairs had been made, including filling at the upper escarpment. Slope movements were progressing intermittently, typically with variations in rainfall.

The slope cross section shown in Figure 4 was used for back-analysis of in-place shear strength. This section is based on a site survey made as part of the field investigation after the slope failure. It should represent the slope after movement to slightly beyond a stable condition, or fs > 1.0.

The slope failure is believed to have occurred under a more critical groundwater condition than that noted during the field investigation, in which the groundwater levels recorded were well below the estimated failure surface. Back-analysis calculations for different groundwater conditions, assuming $c' = 0$, result in the following required friction angles for the Cretaceous clay:

Groundwater Level	ϕ' Required for fs = 1.0 (°)
None	16
Intermediate ($r_u = 0.3$)	15.1
Ground surface	15.6

Strength properties of the overlying loose fill influence very slightly the total shear resistance along the estimated failure surface. Effective friction $\phi' = 25°$ and cohesion $c' = 0$ were used for the shear strength of this soil. This value, which may be high, was assumed in order to ensure a conservative estimate of shear strength for the underlying clay soils being studied.

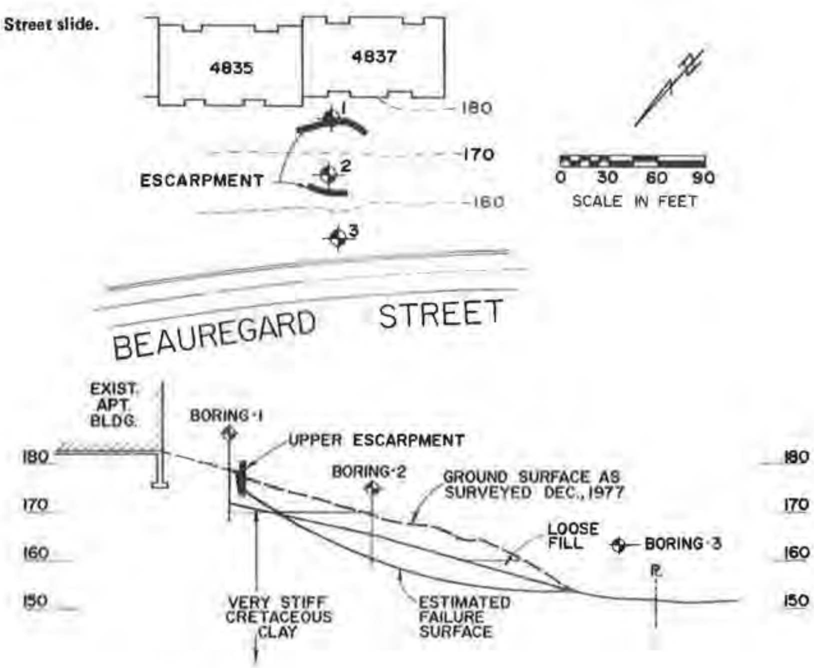
This analysis indicates an in-place shear strength of about 16°, based on the assumption of $c' = 0$. Atterberg limits tests indicate an average plasticity index of 39.

Mount Vernon Square

Mount Vernon Square is an apartment project located about 3 miles south of Alexandria, Virginia, in eastern Fairfax County. In 1971, a slide was noted in an area where borrow excavations had been made during 1967 along the base of a natural slope. The topography of the slide area and site observation data are as follows:

Figure 3. Definition of pore pressure ratio (r_u).

Figure 4. Beauregard Street slide.



Time Period	Topography
Spring 1964	Original, undisturbed topography
Late 1967 or early 1968	Topography after borrow at base of slope
1971	Movement noted but no site topography taken
December 1973	Topography of slide area at upper escarpment
August 1976	Topography of slide area

Analyses were made during 1971-1973 when the test borings were completed, as shown in Figure 5. The soils are predominantly stiff, fissured clay below a sand-and-gravel cap. The borrow excavations in 1967 extended through the sand-and-gravel upper layer, exposing the clay. The section in Figure 5 shows the original ground surface and the ground surface after the borrow excavations.

The eventual landslide area included ponded water, extensive gully erosion, saturated sandy soils, and cracking of exposed clay, which were apparent during the period 1971-1976 after the initial slope failure. Gradual movements were also noted but not accurately measured during this period.

To determine in-place shear strength of the overconsolidated clay soil, back-analysis calculations were made for the slope profile shown after completion of the borrow excavations. This is basically an existing natural slope for which drainage near the toe and along the slope was well established. Accordingly, seepage forces have been assumed in the analysis. The estimated groundwater level is based on readings at the test borings and on available records of site inspections.

The soil profile is predominantly clay along the estimated failure surface shown. Index properties of this soil are within the typical ranges given earlier. The upper sand-and-gravel soil layer is relatively thin and estimated at 6-ft thickness for the analysis. Shear strength has been taken as $\phi' = 30°$ and $c' = 100$ lbf/ft^2 for this soil, and variations of these parameters do not appreciably affect the in-place shear strengths calculated for the clay. The clay soil is considered a basically uniform soil layer whose strength properties are strongly affected by fissures and its generally blocky structure. For imminent failure, or an fs of 1.0, a required friction angle $\phi' = 17.4°$ and cohesion $c' = 0$ result. Soil identification tests show plasticity index ranging from 35 to 53 and an average value of 44.

Villa May East

During the period from 1970 through 1978, there was a series of movements and repairs of a slope along the bluffs overlooking the tidal flats of the Potomac River. This site is in northeastern Virginia just south of Washington, D.C. Special attention was given to the stability of the slope because three underground utilities were affected, as shown in Figure 6. Movements had propagated upslope to the house lines and endangered a gas line in addition to the sewers. Intensive studies of the site were initiated in about 1978. Pertinent topographic data available for the back-analysis calculations are as follows:

Time Period	Topography
August 1953	Original, undisturbed topography
June 1970	Cross section along estimated axes of initial failure
May 1976 and March 1978	Topography of failed slope area

Development in the eventual landslide area included

Figure 5. Mount Vernon Square slide.

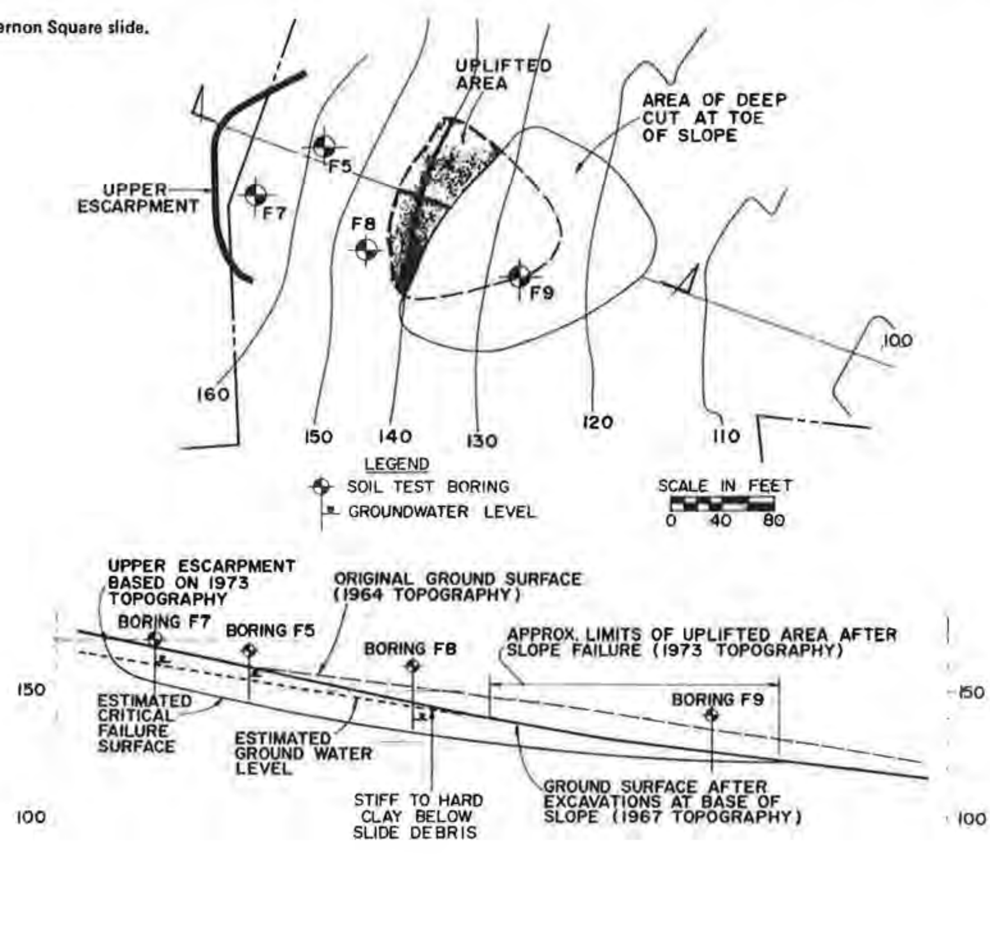
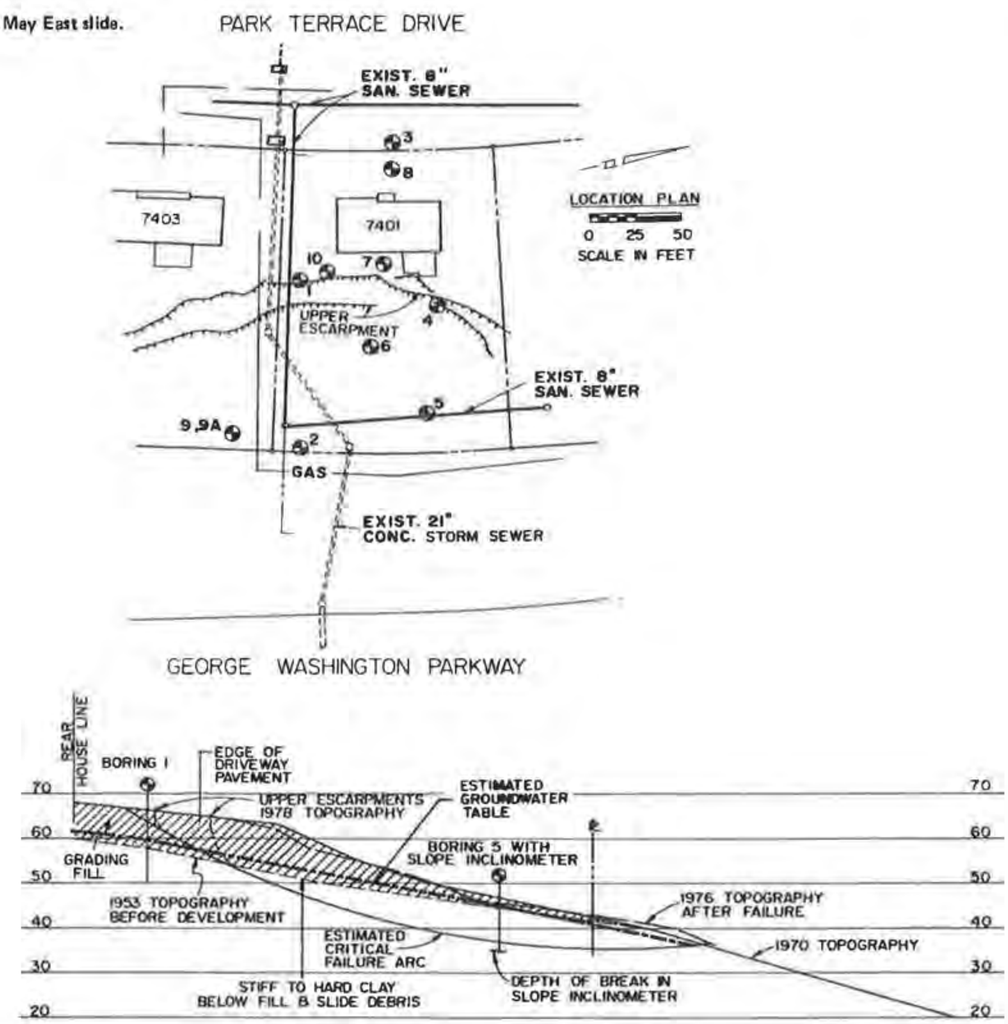


Figure 6. Villa May East slide.



fill depths of up to about 12 ft. The residential structures at the top of the slope are supported on drilled piers that extend below the fill. The original ground surface was at a maximum slope of about 5:1, and the finished slope was graded at about 3.5:1.

Movements noted during the 1970-1978 period were gradual and could generally be correlated with groundwater conditions. Large and relatively rapid displacements usually occurred after heavy rainfall. In some instances grading operations were completed shortly before slope movements occurred. This grading included excavations at the base to clear the roadway and also some filling at the upper escarpment. For the back-analysis calculations, we have used the slope cross section developed from the more accurate survey data obtained in 1970 and 1976. Subsoil profiles and groundwater levels are based on test borings and water observation wells.

The site topography and slope inclinometer readings at two stations were used in developing the estimated failure surface. The estimated critical failure surface is based on slope stability calculations with curve fitting, determined by using the slope inclinometer and topography data shown on the slope section. Back-analysis calculations were made by using a general slip surface program and the failure surface shown.

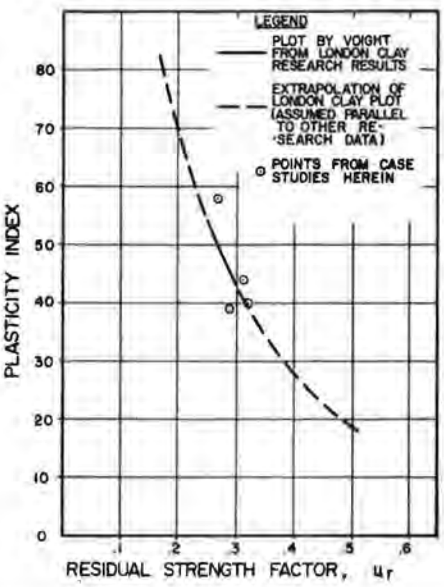
Records of field observations include initial evidence of slope movement consisting of a crack, apparently due to a horizontal displacement, near the upper escarpment. In-place shear strengths would tend to be altered thereafter because of water flowing into the initial opening. Continuing movements resulted in as much as 5 ft of vertical drop near the initial horizontal cracking. Movements after the initial failure obviously involved reduced shear strengths because of wetting, volume changes, and increased pore water pressures that occurred after the initial cracking. At the Villa May slide, this was complicated further by breakage of the storm sewer line as the failure progressed.

In our analysis, in-place shear strength for the first-time slide has been estimated by using the slope cross section given by the 1970 topography. The natural ground surface shown was determined from the test borings and the topography before site development. The failure surface has been drawn from the apparent upper escarpment to just above the inclinometer break and emerges at the apparent bulging along the base of the slope.

We examined primarily the clay shear strength properties, but calculations were also made to verify that properties of the fill do not appreciably influence the results. For final calculation of in-place shear strength of the clay, the following estimated soil properties were used:

Property	Value
Fill	
ϕ' (°)	25
c' (lbf/ft^2)	0
γ_W (lb/ft^3)	125
Clay γ_W (lb/ft^3)	120

Figure 7. Correlation of residual strength and plasticity.



For fs = 1.0, the friction angle of the clay soil varies with the groundwater level, as follows:

Groundwater Level	ϕ' Required for fs = 1.0
None	12.9
As shown in Figure 6	14.9

The groundwater level shown is estimated from the field investigation data and should be reasonably close to the conditions at the time of slope movements. After the 1976 topography was taken, there were intermittent movements until by 1978 a vertical drop of about 5 ft occurred at the upper escarpment. Periodic observations indicate that large and relatively rapid movements were typically accompanied by rainy periods. Based on our observations and analyses, we believe that the condition of no groundwater as listed above would represent an fs greater than 1.0. This case study indicates an in-place friction angle $\phi' = 14.9°$. With fairly limited Atterberg limits tests at this site and additional testing for nearby sites, we have used an estimated average plasticity index of 58 for this case study.

CONCLUSIONS

The following table provides a listing of in-place shear strengths and estimated average plasticity index determined for each of the four case studies:

Case Study	ϕ' (°)	Avg Plasticity Index
Huntington Station	17.5	40
Beauregard Street	16	39
Mount Vernon Square	17.4	44
Villa May East	14.9	58

The literature provides correlations of soil indices and shear strength parameters, including those presented by Karlsson and Viberg (5), Ladd and Foott (6), and Voight (7). Based on the supposition that residual shear strength varies primarily with mineral composition, which affects Atterberg index parameters, the plot developed by Voight can be used to extrapolate the back-analysis data in this paper to find residual shear strength over a range of plasticity index values.

Figure 7 shows the curve plotted by Voight for London clay with residual strength factor (u_r) plotted against plasticity. The curve has been extrapolated as shown, and data points are plotted from the case studies reported here. The residual strength factor (u_r) is equal to tan ϕ' for the cases reported with $c' = 0$. A limited number of data points are presented and, to our knowledge, other studies of the Potomac clay soils do not provide definitive estimates of shear strength parameters to supplement these data. The present available data fit very well with the curve based on Skempton's studies. The shear strength values obtained from Figure 7 should provide reasonably accurate values of residual strength for the clay soils studied, at least in the approximate plasticity index range of 35-60. Conservative applications require assumption of steady seepage and potential of a fairly high groundwater level. Some conditions may warrant less critical assumptions. Various factors must be included in this evaluation, including overall site topography, soil profile and presence of pervious layers, surface drainage conditions, and geometry of the potential failure surface being analyzed.

An important additional factor not considered in this study is the probable increased shear strength below a weathered zone. Typically, there are vertical and subhorizontal breaks within these overconsolidated clay soils that become more widely spaced and poorly developed with increasing depth. The in-place shear strengths reported in this paper are considered valid for an upper weathered zone with greater development of breaks. Obermeier of the U.S. Geological Survey has considered this factor in other reporting. Analysis for potential deep failure planes must include an assumption of an increased shear strength below some estimated depth of weathering. Although a specific depth cannot be applied for all sites, this factor can be included at least qualitatively in considering various potential failure surfaces. A variety of geologic and environmental factors must be included for evaluation of specific problems.

The general relation between plasticity index and shear strength, shown in Figure 7, has also been noted in laboratory shear-strength tests. Direct shear tests run on precut samples with drainage and stress reversal show shear strengths significantly lower and possibly in better agreement with ring shear testing reported by Lupini, Skinner, and Vaughan (8). Laboratory studies of the Cretaceous clays of the study area have consisted primarily of direct shear testing in consulting engineering laboratories. To our knowledge, no testing with ring shear or large samples has been attempted. Soil samples used are standard 3-in-diameter Shelby tube samples, and testing has been conducted on intact and precut specimens by using a constant strain rate, generally about 0.06 in/h. Except for the higher strain rates used, the procedure of precutting and repeated shearing is generally done according to the U.S. Army Corps of Engineers Testing Manual (3). Tests on the overconsolidated CH clay of the study area indicate residual angles of friction on the order of 8-15°. Somewhat coarser MH soils also occurred for the cases studied and show residual friction angles in the higher portion of this range. Lupini, using ring shear tests, has reported residual friction angles of about 7-11° for overconsolidated clay soils with a plasticity index in the 29-61 range. There is significant scatter of laboratory test results, which provides primarily an indication of the general trend; i.e., residual friction angle decreases with increasing plasticity index. We have not attempted to develop a correlation between laboratory residual friction angle and Atterberg index values.

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Design of Cut Slopes in Overconsolidated Clays

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The design of clay slopes for long-term stability is discussed in relation to (a) mode of failure, (b) soil test methods, (c) method of analysis, (d) selection of parameters, and (e) time dependency of stability. Failures of slopes in overconsolidated clays can be evaluated based on drained soil parameters with little groundwater drawdown below the cut face. The time dependency of slope failure is hypothesized to be a function of the Terzaghi hydrodynamic lag model. These failure criteria are evaluated for a case history of a cut slope in the western Allegheny Plateau region of New York State.

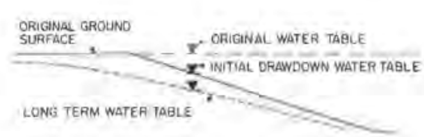
New York State cut-slope design procedures were previously published in the Highway Research Record series (1). It is the intent of this paper to discuss further the selection of design parameters for cut-slope design and to demonstrate, by a case history, the numerical methods used in evaluating a cut-slope failure. It is further intended to present a model to estimate the time to failure for unloading cut-slope soil conditions in the hope that other investigators will pursue the idea to develop a reliable estimation method.

DESIGN PROCESS

Seepage Force

Long-term cut-slope stability has been shown to be directly dependent on seepage force and, therefore, the ultimate groundwater level within the soils in the cut. As shown in Figure 1, the water table immediately after excavation is usually observed at the surface of the new cut slope. The free-water surface will usually then drop slowly to a stable zone at a variable depth below the new cut surface. This drawdown occurs rapidly for sand slopes but is very slow for clay systems. Although a number of investigators have attempted to mathematically model the rate and shape of the groundwater drawdown curves from cuts, none of these mathematical models has proved useful in correctly predicting the time or rate of drawdown of preconsolidated clays or clayey tills because the drawdown is a function of recharge, which has not been adequately modeled.

Figure 1. Seepage conditions in cuts.



A number of open-well piezometers were installed in cut slopes before and after excavation to improve predictions for clayey tills and layered silt and clay systems. The records have only been kept for eight years, but no measurable drawdown was observed (2-5 ft maximum). It is therefore generally assumed that the seepage forces that tend to cause failure are based on a free-water surface no more than about 2 ft below the surface of the cut for most clay or clayey till slopes in New York.

Shear Strength

The stability of the clay cut slope also depends directly on the shear strength of the clay. The shear strength of a clay, however, is not a constant. The in-place shear strength of an overconsolidated clay is a direct function of the prior load (P_p) if the clay is subjected to additional loading (fills) or short-term unloading conditions (temporary cuts). However, if the clay is subjected to long-term unloading conditions (permanent cuts), the strength of the clay no longer depends on the prior loading.

Figure 2 shows data from the case history. The strength of the clay under undrained (short-term) and drained (long-term) conditions is modeled from laboratory tests by use of the Mohr envelope. At the time the cut is made, the average shearing strength (at an average depth of failure plane 50 ft below the original ground surface) is 1850 psf (see Figure 3). After an indefinite time, the strength (at an average depth of failure plane 20 ft below the cut surface) is 610 psf, about one-third of the original strength.

This loss in shearing strength is generally attributed to the reduction in negative pore pressure after the excavation is made. This loss in strength