TRANSPORTATION
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Overconsolidated Clays: Shales

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NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
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Dedicated to

ALEKSANDAR S. VESIC
August 8, 1924 - May 3, 1982

As an administrator, researcher, teacher, and particularly as a scholar, Alex Vesic contributed to the advancement of the geotechnical engineering of the world. There will be a void in the work of the Transportation Research Board, as well as in our profession, with his passing.

As a researcher he developed a second generation of bearing capacity theories for both spread footings and piles. These supplement, and in some cases supplant, older, established theories. He extended these concepts to such diverse applications as evaluating anchors on the ocean bottom and predicting the craters produced by intense explosions.

As a teacher he inspired students to use advanced theory as a tool in solving engineering problems, both at the Georgia Institute of Technology from 1958 to 1964 and at Duke University since 1964. He was an Overseas Fellow, Churchill College, Cambridge, England (on leave from Duke) in 1971-1972. As Dean of Engineering at Duke since 1974, he emphasized scholarship and ingenuity in a period when numbers of graduates had become more important than quality in U.S. universities.

Alex was very active in advancing our profession. He became a member of the Transportation Research Board in the early 1960s and served on a number of its technical committees and as Chairman of the Committee on the Theory of Pavement Design. He also served as Chairman of the Committee on Deep Foundations of the American Society of Civil Engineers (ASCE) and Chairman of the Committee on In-situ Bearing Tests of the American Society for Testing and Materials. He was a noted lecturer at universities in the United States, Canada, Mexico, Eastern and Western Europe, South America, and the Far East. In addition, he had been a principal lecturer at several international and regional conferences on soil mechanics.

Alex received many awards and honors. The one he valued most was an honorary doctorate in science from the University of Ghent in 1981. The Highway Research Board presented him its Research Award in 1967 and ASCE its Thomas Middlebrooks Award in 1974. Duke University presented him its Engineering Alumni Award in 1981 and the Chi Epsilon Outstanding Civil Engineering Professor Award in 1967. He was appointed J.A. Jones Distinguished Professor of Civil Engineering in 1971.

He was a consultant on geotechnical engineering to the U.S. Army Corps of Engineers and the U.S. Navy Civil Engineer Corps. He also served as consultant on such diverse projects as bridge design, building and industrial plant foundations, exploration of the sea bottom, and port structures. He was coauthor of two books and author of more than 80 technical papers.

Alex was born in Yugoslavia. He received his BCE degree with highest honors at the University of Belgrade in 1950 and his Doctor of Science degree in 1956. He did post-doctoral study at the University of Ghent, Belgium; the University of Manchester, England; and at the Massachusetts Institute of Technology before taking up residence in the United States in 1956.

Alex married his classmate, Milena Sedmak, while both were students at the University of Belgrade. She survives him in Durham, North Carolina.

--George F. Sowers

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

J. SCHNABEL AND F. GREFSHEIM

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.

LEGEND FOR CRETACEOUS OUTCROP AREAS

LOWER CRETACEOUS, POTOMAC GROUP

UPPER AND LOWER CRETACEOUS, INCLUDES MONMOUTH, MATAWAN, RARITAN AND POTOMAC GROUP

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.

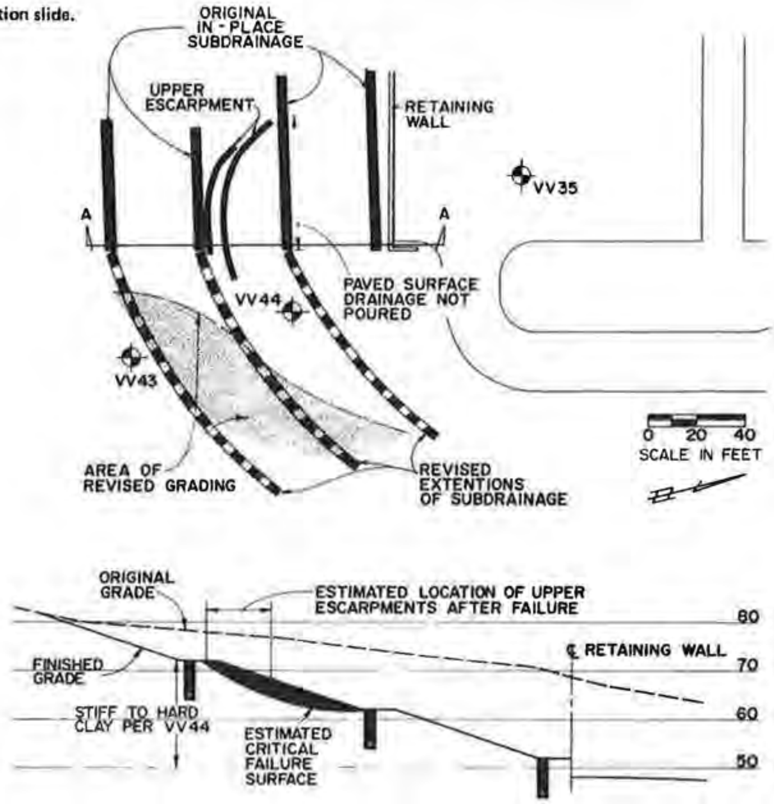
Based on an assumed cohesion $c' = 0$, the friction angle for factor of safety (fs) = 1.0 varies with the groundwater level as follows:

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).

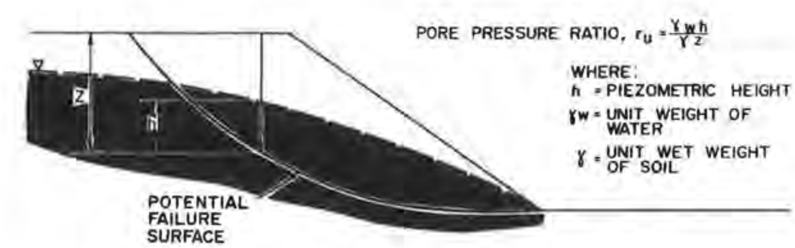
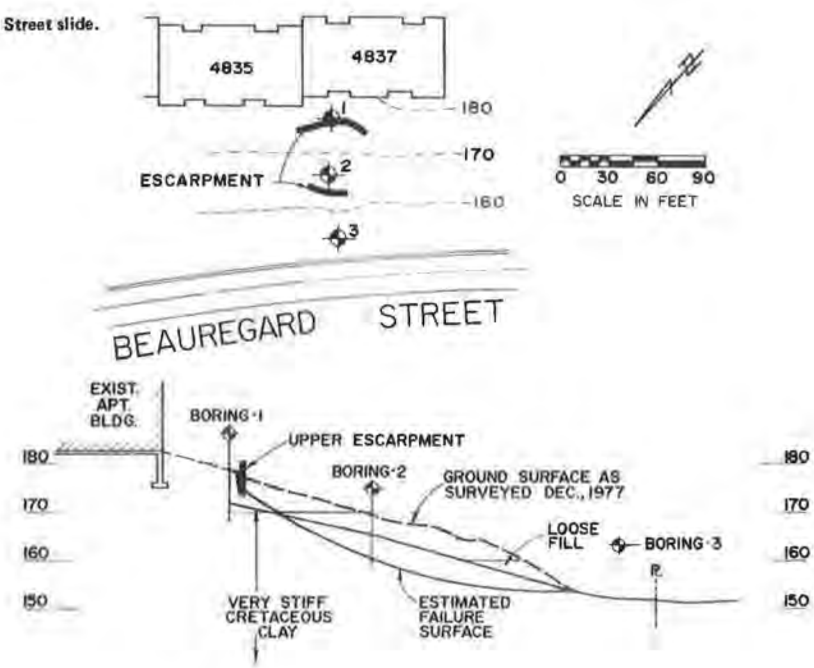


Figure 4. Beauregard Street slide.



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/ft² 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.

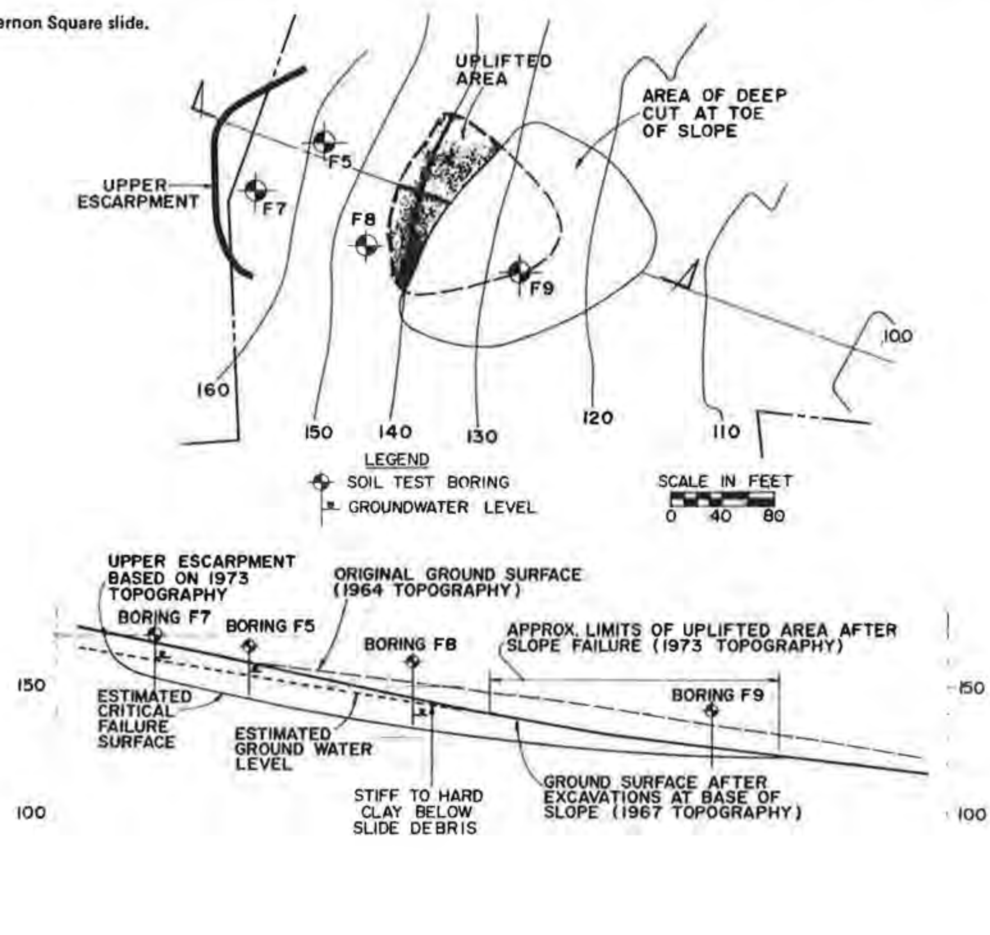
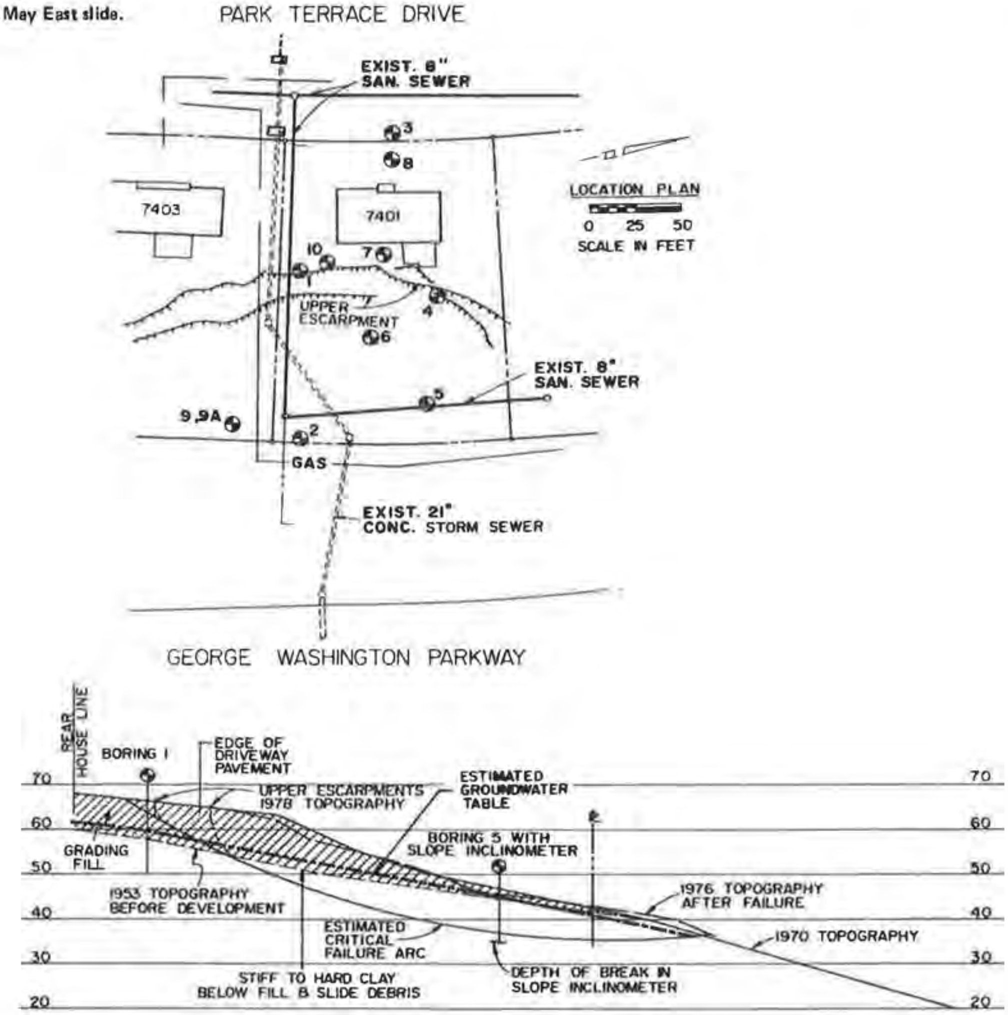


Figure 6. Villa May East slide.



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.

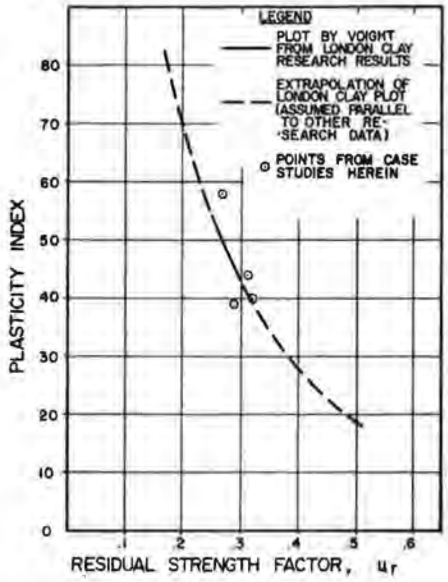
Records of field observations include initial evidence of slope movement consisting of a crack, apparently due to a horizontal displacement, near the upper escarpment. In-place shear strengths would tend to be altered thereafter because of water flowing into the initial opening. Continuing movements 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.

REFERENCES

1. A.W. Skempton. Slope Stability of Cuttings in Brown London Clay. Proc., 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 3, 1977, pp. 261-270.
2. E. DeBeer and E. Gocler. Stability Problems of Slopes in Overconsolidated Clays. Proc., 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 2, 1977, pp. 31-39.
3. Laboratory Soils Testing. Office of the Chief of Engineers, U.S. Army Corps of Engineers, EM 1110-2-1906, Nov. 30, 1970, Appendices IX and IXA.
4. L.M. Force and G.K. Moncure. Origin of Two Clay-Mineral Facies of the Potomac Group (Cretaceous) in the Middle Atlantic States. Journal of Research, U.S. Geological Survey, Vol. 6, No. 2, March-April 1978, pp. 203-214.
5. R. Karlsson and L. Viberg. Ratio c/p' in Relation to Liquid Limit and Plasticity Index, with Special Reference to Swedish Clays. Proc., Geotechnical Conference, Oslo, Norway, Vol. 1, 1967, pp. 43-47.
6. C.C. Ladd and R. Foott. New Design Procedure for Stability of Soft Clays. Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No. GT7, 1974, pp. 763-786.
7. B. Voight. Correlation Between Atterberg Plastic Limits and Residual Shear Strength of Natural Soils. Geotechnique, Vol. 23, No. 2, June 1973, pp. 265-267.
8. J.F. Lupini, A.E. Skinner, and P.R. Vaughan. The Drained Residual Strength of Cohesive Soils. Geotechnique, Vol. 31, No. 2, June 1981, pp. 181-213.

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Design of Cut Slopes in Overconsolidated Clays

VERNE C. McGUFFEY

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.

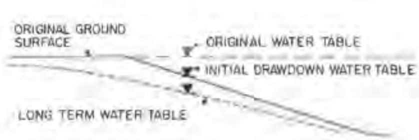
New York State cut-slope design procedures were previously published in the Highway Research Record series (1). It is the intent of this paper to discuss further the selection of design parameters for cut-slope design and to demonstrate, by a case history, the numerical methods used in evaluating a cut-slope 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

Figure 2. Shear strength for case history.

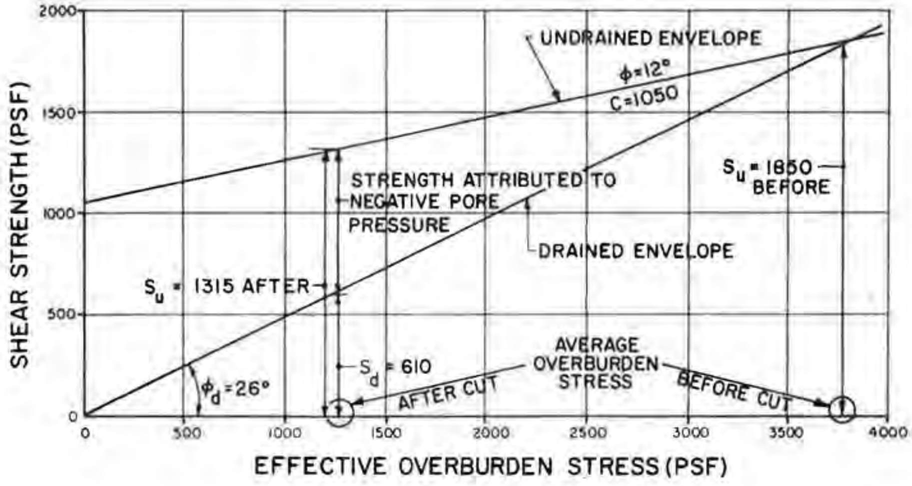
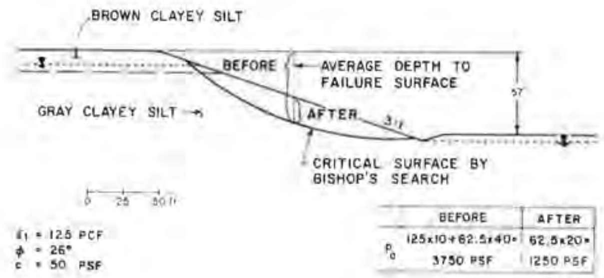


Figure 3. Cross section of slope failure.



has been observed to be a time-dependent function and appears to be related to the rate of dissipation of negative pore pressure.

Factor of Safety

The overall factor of safety of a slope can be evaluated separately for the time-dependent (a) decrease in seepage force (lowered groundwater) and (b) decrease in strength. Figure 4 schematically shows the two time-dependent functions and the corresponding factor of safety versus time function.

Since the water table does not draw down appreciably in most clay cuts in New York, the time dependency of the strength loss then usually controls the time to failure of a cut slope in clay soils.

Time-Dependent Strength Loss

Observations indicate that a large number of slopes more than 30 ft high have failed between 5 and 10 years after construction. Most of these failures occurred in clay or clayey till soils that had medium plasticity [with a plasticity index (PI) between 10 and 15] and a coefficient of consolidation (C_V) of approximately 0.1 ft²/day. Field observations indicate that the average depth of the failure surface through most of the clay systems was between 10 and 20 ft and the average depth was about 15 ft.

If it is assumed that the drained strength is reached at 90 percent consolidation, a simple calculation, using Terzaghi's theory of hydrodynamic lag (4), indicates that time for release of stress could be expressed as follows:

$$t = [(h)^2 \times T_{90}]/(C_v \times 365) = [(15)^2 \times 0.848]/[(0.1) \times 365] \quad (1)$$

where

h = drainage length = 15 ft,
T_{90} = time factor for 90 percent consolidation = 0.848, and
C_V = 0.1 ft²/day.

This simple calculation indicates that the time for stress release to occur, assuming the simple hydrodynamic lag model, would be in the order of 5.2 years. This seems to agree with the observed failures from the field.

An alternative to the simple hydrodynamic lag model is discussed below. It is hypothesized that the excess pore pressure (it would be negative in a cut) would be zero at the surface of the cut slope and it would also be zero at a point below the slope described by a one vertical on one horizontal line down from the top of the cut slope. Using this model, the estimated time to failure would be approximately 5.8 years:

$$t = (h^2\ T_{II}/C_v) = (15)^2\ 0.933/(0.1 \times 365) \quad (2)$$

where h = average distance from the slope face to the depth of the maximum negative pore pressure = 15 ft and T_{II} = time factor for 90 percent consolidation for "Case II" pore pressure distribution (4) = 0.933. Both of these models indicate values within the broad range of the average conditions.

CASE HISTORY

Site Conditions

Figure 3 shows a section through the middle of a cut failure on a section of NY-60 between Cassadaga and Fredonia in the glaciated Alleghany Plateau region of New York State. The soil in this cut was predominantly a silty clay with occasional silt layers and occasional stones. The soil has been overconsolidated, and the water table was near the surface. Construction of the road created two nearly identical cut slopes on opposite sides of the road.

History

The cut was made in 1958 at a slope angle of approximately 1:3 (vertical to horizontal) for a total vertical height of approximately 60 ft. Nine years later (1967) a drop at the top of the slope was noted with cracks occurring near the top of the slope on the east side. An investigation was started in 1969, and samples were taken through the natural materials from the top of the cut. By mid-1970, a drop of more than 15 ft had occurred on both east and west slopes.

Soil Parameters

Some test results are shown in Figure 5. Shear-strength test results are summarized in Figure 2. The samples were driven samples so that the undrained strength would be lower than the true strength, but the disturbance should have little effect on the drained tests.

One consolidated drained test and two consolidated undrained tests with pore pressure measurements were made. (Recent consolidated undrained testing with pore pressure measurements indicates that a good-quality test will yield drained friction angles almost identical with those obtained from good-quality long-term drained tests.) The test values obtained on the soil from the site were compared with the statewide curve of drained friction angle (ϕ_d) versus PI (see Figure 6). By correlating the moisture content and PI data, an average soil PI of 9.5 was obtained. At a PI of 9.5 (Figure 6), an average ϕ_d of 26° was obtained, and this value was used in this analysis. Prior studies have shown a residual cohesion of 50-200 psf for unfailed slopes at the time of failure. An estimate of 50 psf is used here. These values of strength parameters were assumed to best represent the soils on this site.

Stability Analyis

An approximate analysis of undrained failure was done. With an average shear strength of 1850 psf obtained from consolidated undrained strength tests (Figure 2), the factor of safety was $\geq$1.6 by stability charts, and therefore the undrained conditions do not represent the field performance.

Drained parameters were used in a series of modified Bishop stability analyses run on the cut slope. The groundwater table was estimated to be approximately 10 ft (from an observation well) below the surface at the top and to average about 3 ft below the slope face. The most critical failure circle gave a factor of safety of 0.96 at a drained angle of 26° and a cohesion of 50 psf. The most critical surface is shown in Figure 3.

An "infinite slope" analysis (2) was checked for ϕ_d = 26° (C = 0) for an h_w/H (height of water surface above the failure plane divided by depth to the failure plane) of 17 ft ÷ 20 ft = 0.85 (see Figure 7). The factor of safety for a 1:3 slope is 0.87 from Figure 7. If the slope is flattened to 1:4, the factor of safety from Figure 7 is raised to 1.16, which is acceptable.

Factor of Safety

It is common for the factor of safety to be in the

Figure 4. Change of factor of safety with time.

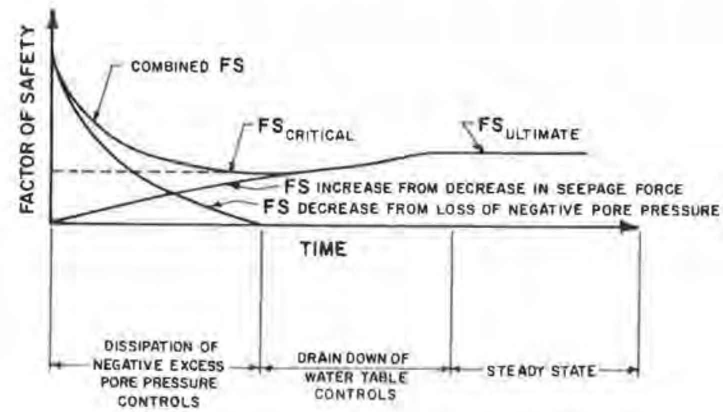


Figure 5. Subsoil conditions.

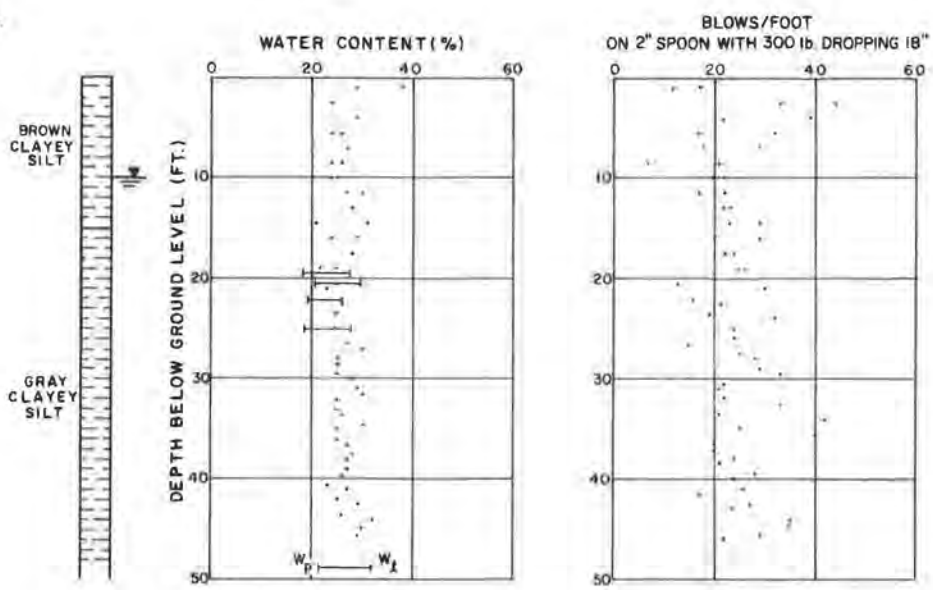
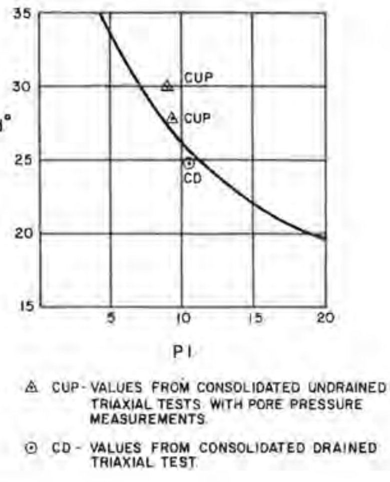


Figure 6. Drained friction angle versus PI for clay and till soils back-figured from failure.



range of 0.85-0.95 when back-figured from a cut-slope failure where movement in excess of 1 ft has occurred. This is assumed to be a result of a short-term higher water table, which produces a factor of safety appreciably below 1 and allows the system to overcome inertia and end restraint and to start movement. If the factor of safety was exactly 1, the soil would not start moving. After an initial movement of 1-3 ft occurs, the driving force of the soil is reduced slightly and the seepage force is also usually reduced as a result of the soil movement. Therefore, the landslide activity will cease and not recur until conditions worsen--for example, until another period of heavy rains, spring snow melt, or toe excavation.

Stabilization

The correction of the stabilization program was simple. The cut slope was trying to reach a stable equilibrium position of approximately a 1:4 inclination. With the help of the maintenance crews digging out the ditches and regrading the upper part of the slope, the slope was reworked to appoximately a 1:4 inclination with little further movement. This was predicted by additional stability analyses that used a 1:4 slope and the same drained parameters and high water conditions determined by modified Bishop analysis.

Estimate of Time to Failure

For this project, the C_v is 0.1 ft²/day and the depth to the failure plane is estimated to be an average of 20 ft below the slope surface (from the most critical surface). The time to failure was estimated to be 9.3 years by using the hydrodynamic lag model:

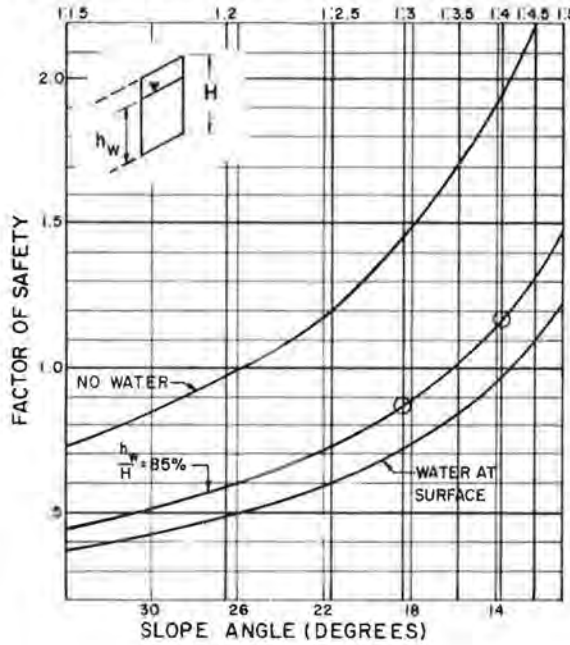
$$t_{90} = [T_{90} \times (h)^2]/(C_v \times 365) = 0.848\,(20)^2/0.1 \times 365 = 9.3 \qquad (3)$$

This agrees very well with the approximate 9 years to first observed major movement.

PROPOSED GUIDELINES

For interpretive purposes, the use of the following guidelines is proposed:

Figure 7. Infinite slope analysis for $\phi_d = 26°$.



1. If a clay slope with a factor of safety less than 1 for drained parameters has not failed within the time estimated by the simple hydrodynamic lag model, it will not fail.
2. If the performance of the slope is critical (if, for example, there is a hospital at the top of the slope), stabilization treatment is required if the factor of safety is less than 1.1 based on drained parameters and high water table.

CONCLUSIONS

Based on the research reported in this paper, the following conclusions can be drawn:

1. Long-term safety of overconsolidated clay slopes can be estimated by using the drained friction angle (plus a low residual cohesion) and seepage forces based on a high water table, determined by modified Bishop or a similar analysis.
2. Stress release in overconsolidated clays is a time-dependent function, and an estimate can be made from the assumption of the simple hydrodynamic lag model and a depth of failure plane based on the geometry of the most critical section.

REFERENCES

1. V.C. McGuffey. Earth Cut-Slope Design in New York State. HRB, Highway Research Record 457, 1973, pp. 1-8.
2. Infinite Slope Analysis. Soil Mechanics Bureau, New York State Department of Transportation, Albany, Manual 7.41-6, March 1972.
3. Computerized Analysis of the Stability of Earth Slopes. Soil Mechanics Bureau, New York State Department of Transportation, Albany, Manual SDP-1, Oct. 1970.
4. D.W. Taylor. Fundamentals of Soil Mechanics. Wiley, New York, 1948.

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Failure of Slopes in Weathered Overconsolidated Clay

JOAKIM G. LAGUROS, SUBODH KUMAR, AND REZENE MEDHANI

Small slides in backslopes in overconsolidated clay are only locally reported and documented, but they constitute a large maintenance expense item in transportation department budgeting. While these failures are usually attributed to drastic changes in water content caused by saturation, there are additional factors that are conducive to the loss of stability but are not considered adequately. These factors are related to reduction in strength and weatherability. The design of slopes uses shear-strength parameters, the values of which are conveniently obtained from triaxial compressive strength tests. However, these values reflect ultimate strength conditions whereas slope failures represent a state of reduced strength. The adjustment from ultimate to reduced strength can be effected by using the Webb technique. When this is done, a drop in the cohesion and interparticle friction values results and, in turn, a substantial reduction in the factors of safety governing slope stability. The weathering of material can be approximated in the laboratory by ultrasonic degradation tests. The data from these tests indicate that the clay content and the plasticity index of the soils in the slopes are higher than those indicated by the conventional standard tests. The slopes, acted on by water, develop higher pore water pressures and less resistance to the forces initiating sliding than predicted. Consequently, slope stability is further reduced and reaches failure or near-failure conditions.

Massive slides of slopes have been well documented in the geotechnical literature. On the other hand, small slides that occur quite frequently are only locally reported, yet they constitute a large maintenance expense item. Seldom are these slope failures subjected to a thorough and intensive investigation. Thus, this study concentrates mainly on small side-slope failures.

Within the topic of slope stability, overconsolidated clay slopes present interestingly unique features that have been described by Bjerrum (1) and Skempton (2) and recently in a comprehensive manner by Fragaszy and Cheney (3). The profound and critical problem of slope stability centers around the prediction of the "shear strength mobilized during undrained failure", since the design of slopes is based on static equilibrium and uses primarily the shear-strength parameters of cohesion and interparticle friction determined in the laboratory. Consequently, the state at which these parameters are evaluated is of paramount importance.

The studies by Bjerrum (1), Skempton (2), Fragaszy and Cheney (3), Gould (4), and Noble (5) emphasized the physical significance of the residual strength concept for overconsolidated material, such as shales. This study, however, is related to the failures that took place within the weathered material derived from Oklahoma shales. The unweathered material was not involved in the failure and therefore is not treated in this study. Whether the weathered material can be considered as definitely overconsolidated may be debatable. However, it is likely that it carries some structural features of its overconsolidation history.

To a great extent, soil structure owes its form and its permanence to the cementation bonds among the soil particles. Studies on the weatherability of shales by Laguros (6) revealed that, when the shales were excavated and put into use, their performance was substandard compared with that predicted through the conventional standard tests for soils. Further investigations (6,7) led to the application of the ultrasonic degradation test. Treatment of shales ultrasonically results in increases in the clay-size soil material and the plasticity index. These increases were attributed to the breakdown of the cementation bonds in the shale and are indicative of the propensity of the material to further weathering. This occurs rather slowly in nature. In the laboratory, however, it can be brought about in approximately 2 hours. Thus, the ultrasonic degradation test constitutes an accelerated simulation of the environmental and other influences in nature. It further provides a tool, at least qualitatively, for predicting the breakdown of the soil structure and the attendant lowering of the factor of safety for slope stability. In addition, the weathered material has a higher initial permeability than its parent shale material; thus, it becomes water saturated with greater ease.

In actual design, the stability of slopes is based on cohesion (c) and interparticle friction angle (ϕ), values obtained from undrained triaxial tests, and a predetermined minimum value of factor of safety. The c and ϕ values are chosen on the basis of an ultimate strength. Failures took place in the slopes that were designed in this manner.

Failures suggest that a reduction in shear strength has taken place. The Webb technique (8) is a method in which the peak strength values obtained from consolidated-drained triaxial tests are modified to give the "reduced" shear-strength values. Under the conditions of this study, c and ϕ values from consolidated-undrained triaxial tests were available, and it was felt that there might be an analogy between these failures and those in overconsolidated clays. Therefore, in an effort to find an explanation for the failures--aside from the fact that there was an augmentation in moisture content--the Webb technique was used.

This paper presents data on cohesion and interparticle friction for ultimate and reduced strength conditions as well as on the breakdown of the interparticle bonds. The use of the data and the analysis of slope stability led to the calculation of new safety factors that are significantly lower than those for which the slopes were designed.

SLOPE CHARACTERIZATION

Slope Failure Conditions

From a relatively large number of cases, the three basic slope failures selected for discussion represent typical occurrences. The common feature of these backslope sections is that they were cut through soil material that developed in place from the parent shale material.

The geometry of failure of the three slopes has the same pattern. Slope 3 is selected as representing this pattern, and its cross section is depicted in Figure 1. The events that preceded the slope failures and the field investigations following the failures indicate that a substantial augmentation of water content took place in the slope soil materials. In slope 1, ponding of water was observed on the top of the bench constructed to drain the water away from the cut, and for slopes 2 and 3 heavy rainfall is reported to have preceded the failure.

A very important feature common to all such slope failures, as shown in Figure 1, is that the weathered shale material was involved in the failure but not the more stable unweathered shale. Field investigations indicated a rotational block-type failure. The slip surface of failure coincided fairly well with the weathered-unweathered shale interface. Evidence of tension cracks was found in the material not involved in failure. Whether these cracks were there before failure is not known.

Since the failure took place soon after rainfall, strength mobilization seems to have occurred under undrained conditions.

Slope Material Properties

The shales from which the slope materials are derived are primarily marine deposits that at one time or another were subjected to high overburden pressures (6). Thus, the parent shales are classified as overconsolidated. Later on, the shales weathered, yielding the material found in the backslopes.

In the areas where the slides occurred, the in-place unit weights of the weathered shales vary from 1.5 to 1.8 Mg/m^3 and their natural moisture contents vary from 11 to 23 percent.

Table 1 summarizes the important properties of the slope soil materials obtained from Shelby tube samples. The geologic information was obtained from Sheerar (9). The engineering properties were determined by using the standard tests. The plasticity and clay content of the shales are reported in terms of two values. The first value is obtained by using the standard American Society of Testing and Materials tests; the second value is obtained by first treating the shale sample ultrasonically for 2 hours (6,7) and then running the standard tests for plasticity index and gradation. It is noteworthy that in all cases both the clay content and the plasticity index of the shales show increases after ultrasonic treatment. In fact, the augmentation of the clay-size material is quite marked, especially in the soil from slope 3.

Within a short time after failure, soil borings were obtained from the failed material. Consolidated-undrained (CU) triaxial strength tests, simulating the most probable field conditions, were run on the samples obtained from the field. Three different lateral pressures--68.9, 137.8, and 206.7 kPa (10, 20, and 30 psi)--were used. Stress-strain data to failure loads were recorded. Strength envelopes from these data are shown in Figure 2. By using the Webb technique (8), the shear-strength parameters at postfailure condition of reduced strength were calculated and are shown in Figure 3. The straightline envelopes of Figures 2 and 3 lead to the graphical determination of the c and ϕ values, both for the ultimate and the reduced strength conditions. These values are presented in Table 1.

For the ultimate strength, cohesion values varied from 83 to 159 kPa. For the reduced strength, they were between 28 and 55 kPa. The angle of friction values varied from 13° to 16° for the ultimate strength and from 11° to 12° for the reduced strength.

ANALYSIS OF FAILURE

The safety factors for slope stability were computed by using the circular arc method. The factors were found to range from 1.25 to 1.28 for the ultimate strength and from 1.04 to 1.08 for the reduced strength. Thus, it is unlikely that the ultimate strength values existed at the time of failure. The reduced strength values, calculated by using the Webb technique, indicate great potential for failure.

Another factor involved in failure stems from the geomorphology of these weathered shales, which have undergone further intensive weathering. This seems to have contributed to further reduction in the factor of safety.

To measure the change due to weathering and to substantiate it, the data obtained from the ultrasonic degradation test are used. These data indi-

Figure 1. Cut slope 3.

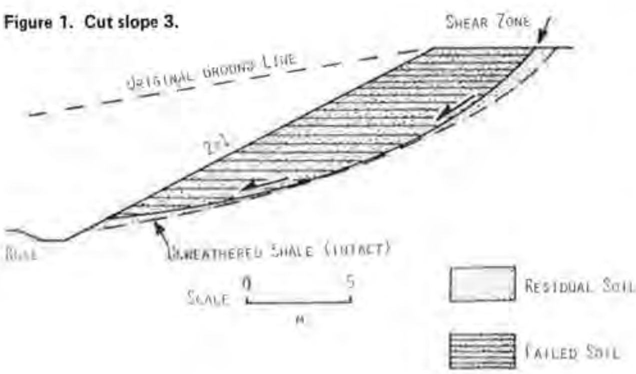


Table 1. Geotechnical and strength properties of slope materials.

Property	Slope 1	Slope 2	Slope 3
County	Pawnee	Love	McIntosh
Geologic system	Pennsylvanian	Cretaceous	Pennsylvanian
Physiographic region	Sandstone Hills	Red River	Prairie Plain
Geologic formation	Vamoosa (Kanwaka)	Caddo (Washita)	Senora (Senora)
Group character	Soillike	Soillike	Rocklike
Design slope	2:1	3:1	2:1
Depth of cut (m)	21.30	9.10	9.10
Year of construction	1963	1961	1963
Year of failure	1964	1964	1966
Annual rainfall (cm)	86	127	104
Clay minerals	Kaolinite, illite	Kaolinite, illite, montmorillonite	Kaolinite, illite, montmorillonite
<2μclay (%)			
Standard ASTM D424-63 (1972)	39	70	14
After ultrasonic test	48	84	65
Plasticity index			
Standard ASTM D424-59 (1971)	23	38	6
After ultrasonic test	25	40	22
Triaxial test			
Ultimate strength			
c_u (kPa)	118	159	83
ϕ_u (°)	13	14	16
Safety factor	1.25	1.28	1.25
Residual strength			
c_R (kPa)	35	55	28
ϕ_R (°)	11	11	12
Safety factor	1.04	1.08	1.04

Figure 2. Ultimate strength envelopes from triaxial test data.

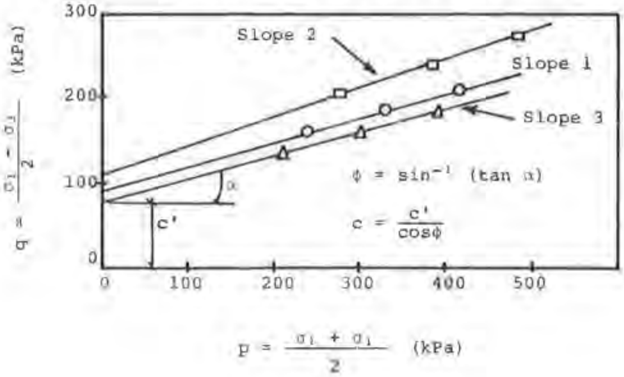
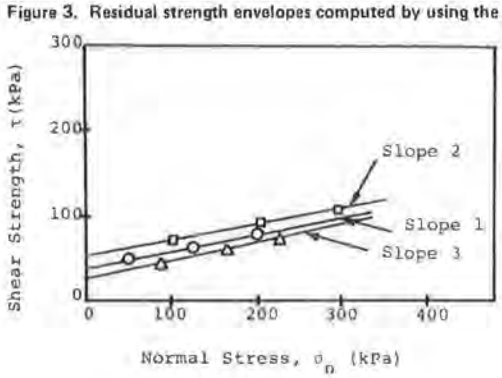


Figure 3. Residual strength envelopes computed by using the Webb technique.



cate that the 2-μm clay content increased 11, 14, and 51 percent for slopes 1, 2, and 3, respectively. The increase in the plasticity index was less pronounced for slopes 1 and 2 but substantial for slope 3. This augmentation in clay-size particles suggests that the soils, after slope construction and because of intensive weathering effects, may become more plastic and contain more clay-size particles than originally designed for. Higher plasticity and higher clay content result in lower permeability and the development of higher pore pressures under saturation; thus, the slope soil material is rendered more vulnerable to failure than anticipated during the design phase.

It should be pointed out that the ultrasonic test presents the soil material in its ultimately degraded state. That the slopes studied had reached that stage cannot be verified. However, it is certain that they had undergone some degree of degradation.

CONCLUSIONS

The stability analysis of slopes based on reduced strength, with the c_R and ϕ_R parameters computed from CU triaxial test data, predicts more realistically the critical field conditions. The potential of the slope soil material to develop high pore pressures and become less permeable in the field is strongly suggested by the increase in the 2-μm clay-size material and the plasticity index determined from the predictive ultrasonic degradation test data.

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REFERENCES

1. L. Bjerrum. Progressive Failure in Slopes of Over-Consolidated Clays. Journal of the Soil Mechanics and Foundations Division, Proc., ASCE, Vol. 93, No. SM5, 1967, pp. 3-49.
2. A.W. Skempton. Long-Term Stability of Clay Slopes. Geotechnique, Vol. 14, No. 2, 1964, pp. 77-101.
3. R.J. Fragaszy and J.A. Cheney. Drum Centrifuge Studies of Overconsolidated Slopes. Journal of the Geotechnical Engineering Division, Proc., ASCE, Vol. 107, No. GT7, 1981, pp. 843-858.
4. J.P. Gould. A Study of Shear Failure in Certain Tertiary Marine Sediments. Proc., Research Conference on Shear Strength of Cohesive Soils, ASCE, New York, 1960, pp. 615-642.
5. H.L. Noble. Residual Strength and Landslides in Clay and Shale. Journal of the Soil Mechanics and Foundations Division, Proc., ASCE, Vol. 99, No. SM9, 1973, pp. 705-719.
6. J.G. Laguros. Predictability of Physical Changes of Clay-Forming Materials in Oklahoma. Research Institute, Univ. of Oklahoma, Norman, Project 1677, Final Rept., 1972.
7. J.G. Laguros and others. A Comparative Study of Simulated and Natural Weathering of Oklahoma Shales. Journal of the Clay Minerals Society, Pergamon Press, Oxford, England, Vol. 22, No. 1, 1974, pp. 111-115.
8. D.L. Webb. Residual Strength in Conventional Triaxial Tests. Proc., 7th International Conference of Soil Mechanics and Foundation Engineering, Mexico City, Vol. 1, 1969, pp. 1521-1544.
9. L.F. Sheerar. The Clays and Shales of Oklahoma. Oklahoma A&M College, Stillwater, Vol. 3, No. 5, 1932.

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Standardized Tests for Compacted Shale Highway Embankments

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Economic considerations often dictate the use of shales in embankments. However, due to the nature of some shales, the embankment may deteriorate with time. Research that defined a series of laboratory tests and a numerical classification system (after Franklin) to be used to predict the performance of shales as embankment materials is described. The criteria for the tests are simplicity, economic and rapid evaluation, use of existing standard testing equipment, clear distinction between suitable and nonsuitable shales, and sufficient range to test most shales. Many tests, therefore, are simply standard tests modified for soft rock. The tests selected are Atterberg limits, five-cycle slake resistance, slake durability, point load strength, impact compaction-degradation, compaction moisture density, one-dimensional consolidation, and triaxial shear. Summarized procedures and discussions of each of these tests are presented.

During the building of the modern Interstate system, many embankments were constructed of shale. This was unavoidable because of the common occurrence of shale near the earth's surface. Where the shale was found to be hard, it was often used as a rock fill. This means that large fragments of rock were placed in thick lifts by being dumped from a truck and compacted minimally. A problem developed in that these shales were sometimes nondurable. Many cases can be cited in the literature of excessive settlements and sometimes failures of shale embankments due to slaking of the shale over time. Basically, the problem is one of the large fragments breaking apart and falling into the large voids that exist in a rock fill.

Because of these failures, several agencies sponsored studies to develop design and construction criteria for shale embankments. One of the largest of these was conducted at Purdue University, funded by the Joint Highway Research Project with the Indiana Department of Highways. The reports that were a result of this study are identified in Table 1.

The initial step involved in the design of any embankment is the sampling and classification of any shales that may be used as fill material. The sampling may be difficult due to the nature of the shale, and typical methods may not be useful. The classification should be based on the time-dependent characteristics of the shale. If the shale is classified as not being suitable material for a rock fill, then four additional characteristics must be ascertained: moisture density, compaction-degradation, compressibility, and shear strength.

EXPLORATION

The basic objective in sampling shale for embankment material is not only to define the complete shale section but also to obtain the quantity and quality of shale sample to run the classification and property tests. Table 2 lists the tests generally required as well as the quantity and the minimum chunk size of the sampled material. Core boring alone may be adequate to classify the material as to hardness and durability, but the layer would have to be both thick and generously sampled to get enough material. The rounded sides of a cored sample may also reduce the abrasion in the slake durability test and give misleading and unsafe results. In addition, it is often very difficult to obtain much intact material when coring in shale. Bailey (3) cites a case in which only 6 m (20 ft) of material was recovered from 15 m (50 ft) of boring, and of that there was only one piece more than 8 cm (3 in)

Table 1. Summary of Purdue University research on shale.

Report	Description
Deo (1)	Useful tests and classification system for determining shale behavior as an embankment material
Chapman (2)	Comparative study of certain test and classification systems by Deo, Gamble, Morgenstern, Eigenbrod, and Saltzman for determining shale behavior
Bailey (3)	Study of field and laboratory shale degradation due to compaction and its relation to point load strength
van Zyl (4)	Statistical analysis of existing shale data and storage and retrieval system for these data
Abeyesekera (5)	Study of stress-deformation and strength characteristics of a compacted shale by use of triaxial testing
Witsman (6)	Determination of effect of compacted prestress on compacted shale compressibility
Hale (7)	Development and application of a standard compaction-degradation test for shales
Surendra (8)	Study of additives that can be used to control slaking in compacted shale embankments
Oakland (9)	Battery of tests and a shale classification system useful in the design and construction of compacted shale embankments

Table 2. Tests and sampling requirements.

Test	Purpose	Quantity	Maximum Aggregate Size	Sampling Method
Atterberg limits	Classification	500 g	None	Auger
Five-cycle slaking	Classification	100-150 g	Intact	Test pit, possibly coring
Slake durability	Classification	450-550 g	10 pieces, 40-60 g each	Test pit
Point load strength	Classification	20 aggregates	25-mm diameter	Test pit, possibly coring
Compaction-degradation	Compaction control	9 kg	38.1 mm	Test pit
Moisture density	Compaction control	45 kg	38.1 mm	Test pit
1-D consolidation[a]	Settlement analysis	32 kg	19.0 mm	Test pit
Triaxial	Slope stability analysis	32 kg	19.0 mm	Test pit

Note: 1 kg = 2.2 lb; 1 mm = 0.039 in.
[a]One-dimensional.

long. Core borings can be used to define the soil and weathered shale depths, the thickness and inclination of the shale strata, and the layers draining into the shale embankment near the cut-fill transition (10). However, for classification purposes a test pit will very often be required. It is also helpful if the stratum can be traced to a nearby existing outcrop where unweathered material can be sampled in quantity.

CLASSIFICATION AND BEHAVIOR CHARACTERIZATION

As a result of the work done at Purdue University, a battery of tests has been developed that is useful in the design and construction of shale embankments for highways. These tests are Atterberg limits, five-cycle slaking, slake durability, point load strength, impact compaction-degradation, moisture-density relations, compressibility, and shear strength. Basically, these tests can be divided into two categories: those that classify the shale by its hardness and durability and those that give actual design parameters. The Atterberg limits, five-cycle slaking, slake durability, and point load strength tests are of the former type and the other tests make up the latter. It is not possible to completely describe these tests in a paper. However, this is done elsewhere (9).

The hardness and durability of a shale determine whether it should be placed as a rock or soil fill. If the shale is classified as being both hard and durable, it can be placed in thick lifts with relatively little compaction control. If it is not both hard and durable, then it must usually be thoroughly degraded and placed as a soil in thin lifts with strict compaction control.

Atterberg Limits

Although there is limited logic in applying a soil plasticity test to shale, Atterberg limits are used in several classification systems (11,12). Deo (1) describes the limitations of these tests for shales, and Abeyesekera (13) recommends that they be used only for classifying relatively soft shales that are to be degraded and placed as a soil fill. Franklin (11) reinforces this idea by developing a classification system that uses the Atterberg limits only for shales that have a slake durability of less than 80. Such shales are usually relatively soft, and standard testing procedures (ASTM D424-59) are suitable.

For any hard shale, degradation of the shale to perform the Atterberg limits tests becomes a tedious, time-consuming process. It is recommended that this effort be avoided. When for some reason this is not possible, ultrasonic devices can be used to facilitate degradation (14). However, such procedures are not suitable for routine use.

Five-Cycle Slaking Test

The five-cycle slaking test is an outgrowth of a slaking test that involves just one cycle of drying and wetting. The one-cycle test was found not to be severe enough to distinguish among the durabilities of various shales (1). The five-cycle test is useful in separating the "compacted" from the "cemented" shales (15) and can determine the shales that are obviously not durable when subjected to water. Cemented bonds will usually make the shales more durable (13).

The test, as originally described by Philbrick (15) and used by Deo (1), consisted of five cycles of drying a 50- to 60-g shale aggregate for 8 h and then submerging the aggregate in water (or another slaking fluid) for 16 h. As Philbrick describes the test, it is qualitative. The end condition of the shale is noted as being "fully slaked", "partially slaked", or "not affected". A description of the shape and size of the remaining shale fragments may also be given.

Surendra (8) and Chapman (2) later used a modified version of the test in which the soaking period was 24 h and the drying time at least 16 h. The degree of slaking was also measured quantitatively as the percentage of material able to pass a 2-mm (No. 10) sieve after each cycle. They also recommended that the appearance of the nonslaked fragments be described. However, the procedure used by Surendra and Chapman is too lengthy to be practical. The original periods of 8 h of drying and 16 h of soaking are recommended. It is also recommended that the slaking index be defined as the percentage of material retained on a 2-mm sieve in order to parallel the index of the slake durability test, which will be described later. Therefore, a shale that is not affected by water would be given a rating of 100 percent and a shale that totally slaked would be rated at 0.0 percent. The condition of the fragments retained should be recorded, since their shape and size can be a help with the durability prediction (13).

An important aspect of the test is the size and shape of the initial shale aggregate (13). In order to keep the tests as uniform as possible, the specific surface (surface area divided by the volume) of the shale aggregates should be as similar as practicable. Therefore, the weight of the essentially one-sized aggregates should be 50-60 g and the shape roughly equidimensional with no protruding corners.

The advantage of the test is its simplicity. It directly evaluates susceptibility to slaking, which is the prime evidence of nondurability. The disadvantages are that it is lengthy, involving a minimum of six days; it may not distinguish among the harder shales; and it does not model the stresses on a shale fragment confined in an embankment.

Slake Durability Test

To resolve the problems of the five-cycle slaking test--i.e., the length of time needed to run the test and the lack of ability to distinguish among the harder shales--an energy input is needed. One solution is the slake durability test developed by Franklin (11). The test adds a tumbling and abrasion action to the normal slaking process. Thus, the slake durability test requires only about two days, and it is more severe in evaluating durability. The disadvantages are that it requires a special piece of equipment and it does not model embankment confinement.

In this test, 10 fragments of shale are tumbled inside a rotating mesh drum that is partially submerged. The drum is made of 2-mm (No. 10) wire screen and rotates at 20 revolutions/min. As with the five-cycle tests, the charge should be closely controlled. The specific surface of the shale fragment is controlled by using shale fragments that weigh 40-60 g each and are roughly equidimensional with no protruding corners.

Another restriction to keep the samples similar is that the entire sample shall weigh 450-550 g.

The selection of the number of revolutions to be used for testing comes from research by Deo (1). He found that the greater the number of revolutions the greater was the contrast among durable and nondurable shales. However, beyond 200 revolutions values were not repeatable. A total of 200 revolutions is therefore recommended. The slake durability index is defined as the percentage of material remaining in the drum after it is rotated in the slaking fluid. Deo (1) also found that samples allowed to soak in water for 6 h before testing degraded more than samples that were tested immediately after being oven dried. In general, the soaked sample testing is too severe to be recommended. However, for the harder shales, specifically those that have an index greater than 85 percent, the soaked test may be used for further durability differentiation (1).

Deo (1) and Franklin (11) concluded that the slake durability index determined from the first cycle of the test gave inconsistent and nonrepresentative values. This may be due to loose material initially adhering to the specimen or easily broken protruding corners. It is therefore recommended that the slake durability index be determined from a second cycle of drying and 200-revolution testing.

Point Load Strength

The point load strength test normally produces a splitting or tensile failure that can be correlated with the rock hardness and compressive strength. Evaluating the shale in this manner is useful since the failure is like that which may occur under rolling or embankment weight. Again, confinement effects are not simulated.

The point load strength test is primarily advantageous for shales because preparation of cylindrical samples for uniaxial compression testing can be avoided. The cost is not the only prohibitive factor with shales. The normal preparation techniques of diamond bit coring and grinding require water as a coolant. This may cause slaking of the shale and may change the natural water content, which is an important factor. A review of test history and use is given by Hale (7).

Basically, the procedure is to place a shale aggregate between two axial contact points and load it to failure. The point load index (I_s) is defined as the load at failure divided by the square of the initial distance (d) between the platens. The value of I_s has the units of stress and is simply related to the stress on the plane of failure at failure.

The disadvantages of this test are that (a) there is considerable scatter in the results, (b) the index varies with the size of the specimen and so a correction factor must be used, and (c) the index varies with the shape of the specimen (7). The advantages include test speed, which allows the difficulty of variation in the results to be overcome by running many tests. In addition, the test device is portable so it can be conveniently used in the field. A third advantage accrues from the first two--i.e., the test can be conducted with very little change in the natural moisture content.

To account for the dependency of the point load index on the size of the specimen, a correction chart has been developed to adjust the index to that of a standard-sized specimen. Usually, the standard diameter is 50 mm; however, Hardy (16) quotes Bieniawski as suggesting that it be 54 mm, which is the diameter of an NX core (16). We favor the 50-mm standard size because of its common acceptance (11,17). Correction charts have been developed primarily from tests on sandstone, quartzite, and limestone (16). Abeyesekera (13) has found that these charts are not generally applicable to Indiana shales. The correction factor depends not only on the shale type but also on the sample shape and the orientation of the bedding planes of the specimen.

With sufficient testing, a unique size correction factor can be derived for each major shale member. Test samples should be bulky in shape and larger than a minimum dimension. Bieniawski recommends that the minimum dimension be 30 mm (16). Hale shows a disproportionately large increase in the point load index as the dimension goes below about 22 mm. This inconsistency may be explained by a certain amount of crushing at the contact platens, which spreads the load over a finite area. It is believed that the error is a function of the ratio of the finite area of contact and the volume of the specimen. This ratio would be small for larger samples but increases rapidly in smaller samples. It is therefore recommended that the samples tested be as large as possible, at least 25 mm.

Investigations of sample shape (18) have led to the prescription that the diameter of an irregular lump (direction in the axial direction of the platen) be 1.0-1.4 times the average width. This is an extremely difficult criterion due to the fissility of shale and especially because the shale is usually loaded perpendicular to the bedding planes. That is, in most cases the smallest dimension for a shale fragment is perpendicular to the bedding planes, yet the recommendation calls for it to be the largest. All shale fragments should be as close to equidimensional as possible.

Since point load strength depends on the direction of loading with respect to the bedding planes (13), more accurate and consistent results can be achieved if the samples are always loaded in the same direction with respect to the bedding planes. The shale aggregates will usually be loaded normal to the bedding planes in the fill. Accordingly, this direction of loading is selected for the point load test.

The water content of the shale at which the point load test is conducted critically influences the index. Bailey (3) considered the effect of three reference water contents on the point load strengths of Indiana shales: 100 percent saturation, 0 percent saturation, and the natural water content. He found it impractical to use 100 percent saturation since wetting the shales caused excessive slaking. At zero percent water content, there was considerable data scatter. Close inspection of the dried shale revealed that many small cracks developed during drying. On the other hand, Bailey was able to produce reasonable and reliable values at the natural water contents. Accordingly, it is recommended that the point load strength test be conducted at the natural water contents of shales.

CLASSIFICATION SYSTEMS

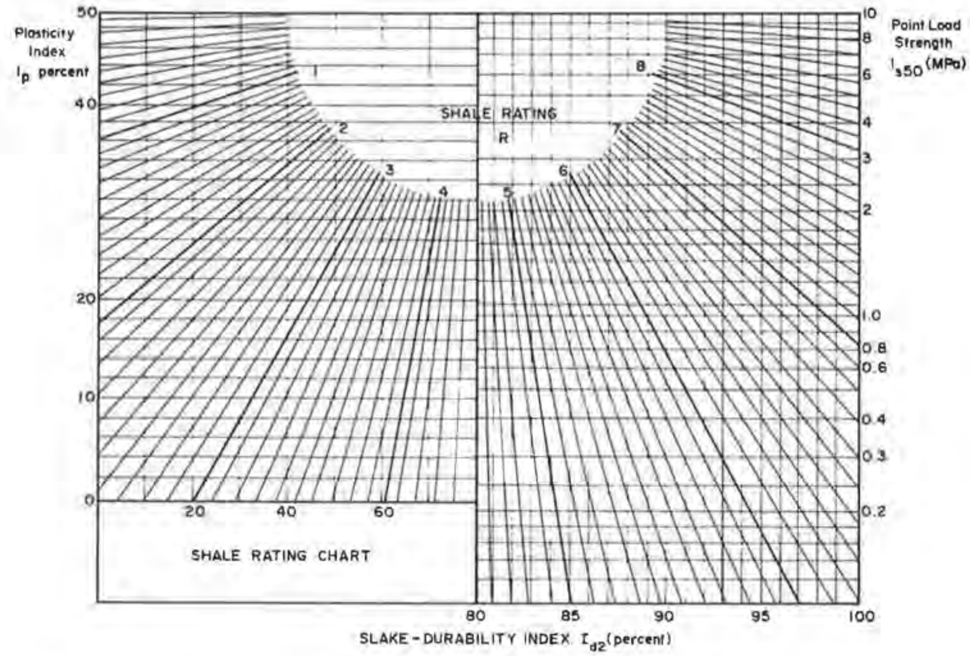
The first generation of shale classification systems was generally based on visual properties. These were classifications used by geologists to describe the shales and were only incidentally related to engineering behavior. Fissility is an obvious characteristic of most shales, and Alling (19), Ingram (20), and McKee and Weir (21) established scales of fissility.

Other classifications used variations in color, texture, and composition in the alternating laminae. Still another common system classified argillaceous rock by the sedimentary particle size (22). Certain adjectives have long been used to describe shales (15,23,24)--e.g., bituminous, oil, arkosic, micaceous, chloritic, and immature. Such classifications are of little help in predicting the behavior of compacted shale embankments.

A second wave of classification systems uses slaking behavior as the primary criterion. Such classifications are more suitable to engineering applications than those previously described.

Mead (25) separated shales into "cemented" and

Figure 1. Shale rating chart.



"compacted" categories. Compacted shales lack a significant amount of cementing agent such as calcite. The cemented shales are considered "rocklike", whereas the compacted shales are considered "soillike". A simple slaking test is used to make the distinction. Systems proposed by Deo (1), Chandra (26), and Hudec (27) extended this concept. Chandra and Hudec used slake durability test values to define the various levels of durability. Deo used multiple slaking tests and a soundness test to classify shales. Others have combined a slaking test with some other index. Gamble (12) and Morgenstern and Eigenbrod (28) use a slaking test and the plasticity index. Gamble used the slake durability test whereas Morgenstern and Eigenbrod developed a rate of slaking test. The system is best suited to softer shales that are easily degraded for the Atterberg limits tests (1).

Saltzman (29) geared his classification system toward the harder shales in using the Los Angeles abrasion test in combination with the Schmidt hammer. Both tests are too severe for Indiana shales.

The most modern systems are by Franklin (11) and Strohm, Bragg, and Ziegler (10). Franklin uses the slake durability test, the plasticity index, and the point load strength test. The point load index is applied to classify the more durable shales, and the plasticity index is used to rate the others.

In choosing a classification system, four criteria must be met:

1. The tests must be relatively simple, using only readily available and, if possible, easily portable equipment.
2. The test should be relatively rapid to permit classification soon after the shales have been excavated.
3. The system must be able to distinguish clearly among shales in the geologic population.
4. It is essential that the classification values be quantitative and numerical.

The Franklin system seems to meet all of the above criteria and should be used to classify embankment shales. Most laboratories already have the apparatus to conduct the Atterberg limits and point load strength tests, and the equipment for the slake durability test is relatively inexpensive. As was emphasized earlier, the tests are rapid, especially if a microwave oven is used for drying. The classification scale (R-value) is broad enough to cover all but the most exceptionally hard argillites. It ranges from zero to nine and thus affords a continuous numerical rating. The chart used to classify shales by this method is shown in Figure 1 (11): If the slake durability index is less than 80, it is used in conjunction with the Atterberg plasticity index; otherwise, the slake durability index and point load strength are used.

COMPACTION OR DEGRADATION TESTS

Since embankments of nondurable shales must be thoroughly degraded and tightly compacted in thin lifts, standard tests that rate the degradability and define the compaction relations are required.

Compaction-Degradation Test

Degradation of the shale will occur in all handling processes from excavation through final compaction. Correlations among rock classifications and methods of excavation are available (30). Predictions of degradability under field rolling are needed as well, and the laboratory compaction-degradation test is a first step in meeting this need.

The test is basically one in which the change of gradation produced by a standard compaction process is measured. Bailey (3) summarizes the various ways that gradation and change in gradation may be represented. In an extension of Bailey's research, Hale (7) selected the "index of crushing" as the standard measure of degradation. This index is the percentage change in mean shale aggregate size due to compaction. The compaction-degradation test is commonly needed only for harder shales. Indeed, it may be difficult to run for softer shales that tend to be cohesive and have the fragments bonded together when compacted. Fortunately, degradation measures are not needed for such shales.

The initial gradation--that is, the gradation before compaction--is one of the variables of the test. Ideally, the gradation should parallel that of the field gradation before compaction, but this is an unknown and varies widely with the project. A convenient gradation is afforded by the following expression:

$$P = 100(d/D)^n \tag{1}$$

where

- P = percentage by weight finer than size d,
- d = sieve size,
- D = maximum aggregate diameter, and
- n = 1.

This is a good approximation of the gradation of the material after crushing in a reciprocating jaw crusher, and wasted material is minimized. In cases where the field gradation is known to be significantly different from this relation, the gradation for the laboratory test can be modified accordingly.

To produce the proper gradation, the crushed material is separated in a nest of sieves and recombined by size fractions to fit the gradation. The sieved product exists in a discrete function and will imperfectly fit the continuous function of the gradation equation. The error is reduced by using more sieves, but this requires additional time to prepare the gradation. Since the test should be conducted at the natural moisture content, significant drying of the material may occur if the sieving process is too long.

In earlier work by Aughenbaugh (31) and Bailey (3), 10 sieve groups were used. Hale (7) enlarged the average range of each sieve group by increasing the maximum size aggregate but reduced the number of groups to 9. It is believed that the test can be further simplified by combining the smaller aggregate sizes into one sieve group. It can be shown that relatively large errors in the percentage of these fine materials will result in negligible changes in the gradation (28).

The maximum aggregate size also has a major effect on degradation. Hale (7) found that the larger the maximum size, the larger is the index. This is logical since the coarser gradation should have fewer aggregate contact points and hence higher contact stresses. The higher values of the index tend to make it easier to distinguish among different shales. It is therefore recommended that the maximum aggregate size be as large as is practical. Since a 15.2-cm (6.0-in) mold will be used, the maximum size that works well is 3.8 cm (1.5 in).

The type and effort of compaction are other important factors. Bailey (3) studied four types of compaction: kneading, gyratory, static, and impact. He favored the static and impact compaction methods. The static method has the advantages of being consistent and simple and uses a known compactive force. The impact method displayed almost as much consistency and had the advantage of being a widely accepted compaction mode. Hale (7) made a further study of the two methods and recommended the impact method for the reason of common acceptance.

Hale (7) also investigated various levels of nominal compaction effort. Based on many trials, a nominal effort of 790 kJ/m^3 (16 500 $lbf\cdot ft/ft^3$) was selected. This is obtained by compacting 0.002 m^3 (0.075 ft^3) in three layers with 25 blows/layer by using a 4.5-kg (10-lb) modified Proctor hammer with 43.5 cm (18 in) of free fall.

Compaction Control Test

Whenever possible, the compaction control should be generated in a test pad. It may be stated in terms of an end result, procedure, or combination thereof. The techniques that follow apply to the definition of a laboratory moisture-density curve that can be used in an end-result specification.

If the shale is soft and has good absorption characteristics, water content may be used as a variable in the usual way. Raising the water content weakens the shale and increases its degradability (7). Just as with soils, however, there is a point at which additional water hampers the compaction process and lowers the compacted dry density. Thus, an optimum water content and maximum density can be defined for a given shale and compactive effort.

The test is not well suited to hard shales because, in general, added water is not absorbed. Such materials must ordinarily be compacted at or near their natural water content. Density values for specification are defined by varying the compactive effort at constant water content either in the laboratory or in the field.

To achieve consistent and reproducible results, the moisture content throughout the sample should be as uniform as possible. Two techniques are recommended: (a) adding water in a spray and mixing thoroughly and (b) allowing the material to cure for two days. Bailey (3) found little or no benefit from curing more than two days.

Because the same basic procedures are used for defining a single point in compaction control and in the compaction-degradation test, the latter test may be used to produce a single point for the moisture-density curve.

Compressibility Test and Settlement

A compressibility test is needed whenever it is necessary to predict embankment settlement. The ordinary one-dimensional consolidation test apparatus can be used for the measurements. Example measurements by Witsman (6) are shown in Figure 2. As-compacted compressibility is shown by the initial curve. This curve will change in shape at the compaction prestress. After the shale has compressed under a pressure that approximates an embankment confinement, the sample is saturated and either settles or heaves. The saturation simulates the effect of the environment on the embankment in service and is represented by the vertical portion of the curve in Figure 2. Further loading would produce settlement according to the compressibility of the shale in a saturated condition.

Such laboratory data can be scaled upward from laboratory model to embankment prototype and used to predict the embankment settlement. Other data by Witsman (6) show that the as-compacted compression (under self weight) will occur as rapidly as the embankment is constructed.

To make the compression tests as consistent as possible with the other shale tests, the sample preparation is much the same as for the moisture-density test. It is recommended that the same gradation function used in the compaction-degradation test be used in this test. The mold for this test is only 10.2 cm (4.0 in) in diameter, and therefore the maximum particle size is reduced to 1.9 cm (0.75 in).

The water content selected for the test should be that predicted for the field application. This might be the natural water content or the optimum water content defined in a laboratory moisture-density test. Again, to achieve the most consistent results, the water content must be uniform throughout the sample. It is therefore recommended that the water be added as a spray and the sample be allowed to cure for at least one day. The curing

Figure 2. Collapse-consolidation test on New Providence shale.

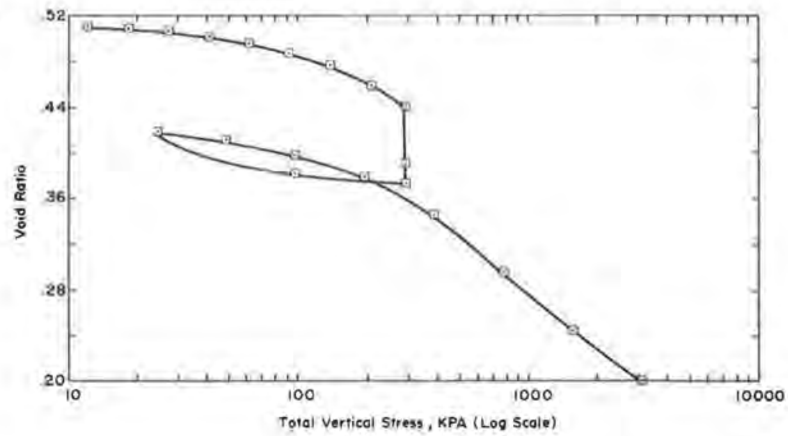
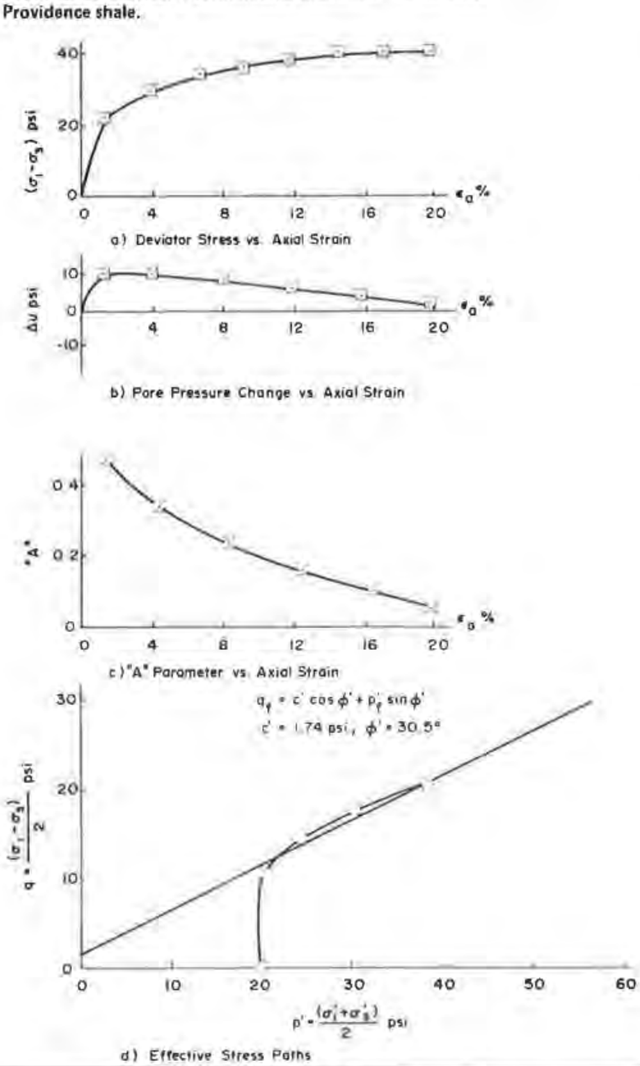


Figure 3. Isotropically consolidated undrained triaxial shear test on New Providence shale.



time for these samples is less than that for the compaction samples because both the maximum aggregate size and the sample size are smaller.

To approximate the field compaction, the laboratory kneading compaction is recommended. In this technique, kneading foot pressures are adjusted to match the density of the laboratory control curve at the desired water content. It is further recommended that these samples be compacted in five equal layers to achieve better sample homogeneity.

The test procedure will start by loading the partially saturated compacted sample in small load-increment ratios until the prestress point is defined. The second stage is to saturate the shale while monitoring the volume change. The confining load should correspond to a depth in the embankment prototype. It is necessary to conduct several tests to establish the settlement for the entire vertical profile of the embankment. Finally, the sample should be unloaded and reloaded to establish the rebound and loading relations of the saturated shale.

Abeyesekera's collapse test (5) gives an indication of what may happen if the shale is not well degraded and thoroughly compacted.

Shear Strength Test and Slope Stability

Strength tests are needed to estimate the likelihood of a slope failure. The preferred procedure is to extract intact cylindrical samples from a compacted test pad. Such samples can be confined to approximate various embankment positions. If these are sheared undrained, the as-compacted strength is defined and slope stability analysis can be undertaken by using a suitable computer program such as STABL2 (32).

If the samples are produced in the laboratory, the sample gradation and preparation are the same as for the compressibility test, except that the sample height is 21.6 cm (8.5 in) with a diameter of 10.2 cm (4.0 in). In order to keep the thickness of each layer of compaction the same as was used in the consolidation test, nine layers are needed. The calibration relating kneading foot pressure to density at a given water content may also be used in this test.

To determine the embankment stability in the long term, samples are consolidated to a range of values approximating the range of embankment heights and are back pressure saturated. It is possible to measure the volume change that takes place by monitoring the flow of water into and out of the sample during saturation. If an estimation of settlement or heave at isotropic stress is needed, it can be made from this stage of the test. The samples are then sheared undrained, preferably with pore pressure measurements.

Typical stress-strain, pore pressure-strain, A-factor-strain, and stress-path relations for the New Providence shale tested by Abeyesekera (5) are shown in Figure 3. These relations depend greatly on the confining pressure and its ratio with the compaction prestress. Strength is interpreted in terms of either total or effective normal stress (Mohr-Coulomb), and the long-term stability is assessed.

Values of the effective stress intercept (c') for all saturated samples at reasonable conditions of compaction are expected to be small, and the effective stress-strength angle (ϕ') is insensitive to compaction variables. Abeyesekera (5) found a ϕ' for Indiana New Providence shale of 28°-30°. In contrast, if the same shale was loosely placed, ϕ' decreased to 25°. Excess pore pressures at failure did vary considerably with the details of compaction and the confinement. Accordingly, the factor of safety against an undrained failure could vary significantly with the above compaction factors.

SUMMARY

Economic design and construction of shale highway embankment require three activities: exploration and sampling, classification, and testing for design parameters. Exploration determines the depth and lateral extent of the shale layers. Proper sampling produces the required quantity and maximum aggregate size of shale at the natural water content. Classification determines whether the fill is to be constructed as rock fill or a soil fill. The tests used for classification are the slake durability test (or the five-cycle slaking test if the former test equipment is not available) and either the Atterberg limits or the point load strength test. The Franklin system (11) is used to classify the shales. Finally, a design parameter suite of four tests is performed. A compaction-degradation test shows the degradability at natural water content, and the moisture-density test is the same type used for soils. Compressibility is evaluated for loading in an as-compacted condition, heave or settlement on confined saturation, and loading in a saturated condition in a one-dimensional (oedometer) test. Strength is evaluated with triaxial samples for both as-compacted and saturated samples. The tests are undrained and for the long-term case should include pore pressure measurements.

The tests have been selected after extensive review of the state of knowledge of compacted shale embankments and after 10 years of concentrated research in this area at Purdue University.

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REFERENCES

1. P. Deo. Shales as Embankment Materials. Purdue Univ., West Lafayette, IN, Ph.D. thesis, and Joint Highway Research Project Rept. 45, Dec. 1972, 202 pp.
2. D.R. Chapman. Shale Classification Tests and Systems: A Comparative Study. Purdue Univ., West Lafayette, IN, M.S.C.E. thesis and Joint Highway Research Project Rept. 75-11, May 1975, 90 pp.
3. M.J. Bailey. Degradation and Other Parameters Related to the Use of Shale in Compacted Embankments. Purdue Univ., West Lafayette, IN, M.S.C.E. thesis and Joint Highway Research Project Rept. 76-23, Aug. 1976, 209 pp.
4. D.J.A. van Zyl. Storage, Retrieval and Statistical Analysis of Indiana Shale Data. Purdue Univ., West Lafayette, IN, Joint Highway Research Project Rept. 77-11, July 1977, 98 pp.
5. R.A. Abeyesekera. Stress-Deformation and Strength Characteristics of a Compacted Shale. Purdue Univ., West Lafayette, IN, Ph.D. thesis and Joint Highway Research Project Rept. 77-24, Dec. 1977, 420 pp.
6. G.R. Witsman and C.W. Lovell. The Effect of Compacted Prestress on Compacted Shale Compressibility. Purdue Univ., West Lafayette, IN, M.S.C.E. thesis and Joint Highway Research Project Rept. 79-16, Sept. 1979, 181 pp.
7. B.C. Hale. The Development and Application of a Standard Compaction-Degradation Test for Shales. Purdue Univ., West Lafayette, IN, M.S.C.E. thesis and Joint Highway Research Project Rept. 79-21, Oct. 1979, 180 pp.
8. M. Surendra. Additives to Control Slaking in Compacted Shales. Purdue Univ., West Lafayette, IN, Ph.D. thesis and Joint Highway Research Project Rept. 80-6, May 1980, 304 pp.
9. M.W. Oakland. Classification and Testing of Shale to Be Used in Compacted Shale Embankments. Purdue Univ., West Lafayette, IN, M.S.C.E. thesis and Joint Highway Research Project Rept., Dec. 1981.
10. W.E. Strohm, Jr., G.H. Bragg, Jr., and T.W. Ziegler. Design and Construction of Compacted Shale Embankments: Volume 5--Technical Guidelines. Offices of Research and Development, FHWA, Dec. 1978.
11. J.A. Franklin. A Shale Rating System and Tentative Applications to Shale Performance. TRB, Transportation Research Record 790, 1981, pp. 2-12.
12. J.C. Gamble. Durability-Plasticity Classification of Shales and Other Argillaceous Rocks. Univ. of Illinois, Urbana-Champaign, Ph.D. thesis, 1971.
13. R.A. Abeyesekera and C.W. Lovell. Characterization of Shales by Plasticity Limits, Point Load Strength, and Slake Durability. Proc., 31st Annual Highway Geology Symposium, Austin, TX, Aug. 1980.
14. J.G. Laguros. Suggested Test Procedure for Ultrasonic Disaggregation of Clay Forming Materials. Univ. of Oklahoma Research Institute, Norman, Aug. 1972.
15. S.S. Philbrick. Foundation Problems of Sedimentary Rocks. In Applied Sedimentation (P.D. Trask, ed.), Wiley, New York, 1950.
16. H.R. Hardy. A Laboratory Manual for Rock Mechanics, rev. ed. College of Earth and Mineral Sciences, Pennsylvania State Univ., University Park, 1982.
17. Suggested Method for Determining the Point Load Strength Index. Commission on Standardization of Laboratory and Field Tests, International Society of Rock Mechanics, Lisbon, Portugal, Document 1, Part 2, Oct. 1972.
18. E. Broch and J.A. Franklin. The Point Load Strength Test. International Journal of Rock Mechanics and Mining Sciences, Vol. 9, No. 6, Nov. 1972.
19. H.L. Alling. Use of Microlithologies as Illustrated by Some New York Sedimentary Rocks. Bull., Geological Society of America, Vol. 56, 1945.
20. R.L. Ingram. Fissility of Mudrocks. Bull., Geological Society of America, Vol. 64, Aug. 1953.
21. E.D. McKee and G.W. Weir. Terminology for Stratification and Cross-Stratification in Sedimentary Rocks. Bull., Geological Society of America, Vol. 64, 1953.
22. P.E. Potter, J.B. Maynard, and W.A. Pryor. Sedimentology of Shales. Springer-Verlag, New York, 1980.
23. W.T. Huang. Petrology. McGraw-Hill, New York, 1962.

24. L.B. Underwood. Classification and Identification of Shales. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM5, 1969.
25. W.J. Mead. Redistribution of Elements in the Formation of Sedimentary Rocks. Journal of Geology, Vol. 15, 1907.
26. R. Chandra. Slake Durability Test for Rocks. Imperial College of Science and Technology, Univ. of London, London, England, M.S. thesis, Rock Mechanics Res. Rept., 1970.
27. P.P. Hudec. Development of Durability Tests for Shales in Embankments and Swamp Fills. Research and Development Division, Ontario Ministry of Transportation and Communications, Downsview, Res. Rept. 216, April 1978.
28. N.R. Morgenstern and K.D. Eigenbrod. Classification of Argillaceous Soils and Rocks. Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No. GT10, Oct. 1974.
29. U. Saltzman. Rock Quality Determination for Large-Size Stone Used in Protective Blankets. Purdue Univ., West Lafayette, IN, Ph.D. thesis, May 1975.
30. H.R. Thomas, J.H. Willenbrock, and J.L. Burati, Jr. CE 431 Supplementary Class Notes. Department of Civil Engineering, Pennsylvania State Univ., University Park, July 1979.
31. N.B. Aughenbaugh, R.B. Johnson, and E.J. Yoder. Degradation of Base Course Aggregates During Compaction. School of Civil Engineering, Purdue Univ., West Lafayette, IN, May 1963.
32. E. Boutrup. Computerized Slope Stability Analysis for Indiana Highways: Volumes I and II. Purdue Univ., West Lafayette, IN, M.S.C.E. thesis and Joint Highway Research Project Repts. 77-25 and 77-26, 1977, 512 pp.

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Slaking Modes of Geologic Materials and Their Impact on Embankment Stabilization

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The results of a study of the impacts of slaking on surface mine spoils in the Appalachian Basin are discussed. Representative samples were collected for laboratory analyses from core borings near the active highwall and from test pits excavated in fresh and 2-, 5-, and 10-year-old spoils from four selected mine sites. Extensive qualitative and semiquantitative data were collected on the behavior, causes, and effect of slaking materials. Three slaking modes were identified in the field: (a) slaking to constituent grain size; (b) chip slaking to thin, platy fragments; and (c) slab or block slaking to large, approximately equidimensional fragments. These modes are dependent on inherent material properties such as bedding, cementation, and grain size and may affect overall spoil pile or embankment stability. Slaking to inherent grain size promoted surface crusting, reducing infiltration and accelerating sheet erosion. Stability problems in embankments and spoil piles can be anticipated by observing the rate, degree, and modes of slaking.

The problem of understanding the impact of the slaking process is underscored by the fact that there is no universally accepted definition of the term "slaking". It has been defined as "the crumbling and disintegration of earth materials when exposed to air or moisture" (1), the "disintegration of rocks by water immersion" (2), or the "disintegration of mudstones upon alternate drying and wetting" (3). These definitions fail to consider the element of rate of breakdown and changing stress or strength conditions within the material.

An alternative definition of slaking has been proposed that focuses on short-term dynamic stress and strength conditions and the fundamental mechanisms involved (4).

The proposed definition is as follows: Slaking is the short-term physical disintegration of a geologic material following removal of confining stresses. Breakage may result either from the establishment or the occurrence of sufficient stresses within the material or from the decrease in structural strength. The significance of the disintegration rate depends on the specific engineering consideration; for definition, "short-term" may be taken to mean less than several years.

Since slaking by definition involves the degradation of geologic materials, it follows that the properties of these materials play an integral part in the slaking phenomenon. These properties can be broadly characterized as stratigraphic or structural.

Two stratigraphic components related to durability of a given material are lithology and mineralogy. Lithology, such as sandstone or shale, considered in conjunction with bedding characteristics, can generally be related to durability; e.g., argillaceous materials have a higher slake potential than arenaceous materials. Mineralogy also affects durability by defining the composition and stability of individual grains and cementing agents.

Clay minerals, because of their water absorption and ion adsorption properties, can exert physicochemical stresses within rock materials, a process that can lead to slaking. In addition, mineralogic properties of cementing agents, including bond strength and chemical activity, may influence slakability.

Megascopic and microscopic structures, as they relate to planes of weakness, influence durability by controlling the entry and mobility of pore fluids, residual stresses, and fragment size. Megascopic geologic structures, especially faults and joints, relate to zones of weakness created during earth movements. Microscopic structures (rock fabric) describe spatial relations between individual grains and among grains. Fabric can directly affect rock integrity by influencing interactions between grains and their reaction to imposed stress. This is especially prominent where a strongly anisotropic fabric is present.

STUDY METHODOLOGY

A detailed field program was implemented as part of a research program sponsored by the U.S. Bureau of

Mines to document the nature, occurrence, distribution, and effects of slaking in surface mine spoils. The results of the laboratory portion of this study are presented in the paper by Withiam and Andrews in this Record. The field program consisted of five phases:

1. Preliminary site selection,
2. Site reconnaissance and final site selection,
3. Highwall exploration,
4. Mine spoil exploration, and
5. Interviews with mine and regulatory personnel.

Preliminary site selection was based on consideration of mining technology, duration of operations, and overburden lithologies of candidate sites. Further screening was provided by consideration of mining and reclamation techniques, geographic location, production characteristics, and minable extent of seams mined at each site as well as uniformity of overburden materials on a site-specific and regional basis.

Following the preliminary site-selection process, a one-day field reconnaissance was performed at seven potential sites. Site-specific information was obtained to assess the compatibility of each site with study objectives and to collect additional data on mining and reclamation methods, overburden composition, and slaking characteristics. Based on the reconnaissance results, four sites that provided an optimum mixture of mining methods and geologic materials as well as good spatial distribution within the study area were selected for detailed field studies (see Figure 1).

A boring was drilled near the active highwall to evaluate in situ overburden characteristics at each site. Rock coring was conducted through the full section of overburden (stratigraphy overlying the coal) to the lowest mined coal seam by using truck-mounted drill rigs and clear water as the drilling fluid. Rock core was described and visually classified by using a standard format that included color, lithology, relative hardness, depths and thickness of rock units, recovery and rock quality designation, and other significant features. The logs were correlated with the exposed highwall, and any additional pertinent highwall observations were incorporated.

A test-pit exploration program was conducted to evaluate slaking and associated effects in spoil 0 (recent), 2, 5, and 10 years old at each site. The investigation included

1. Identification of lithologies undergoing slaking,
2. Identification of inherent lithologic properties influencing slaking,
3. Evaluation of time effects on slaking, and
4. Evaluation of mining and reclamation techniques on slaking.

Four test pits on spoil of each age, a total of 16 per site, were excavated to a depth of approximately 2 m after a detailed inspection of surficial characteristics. Preliminary logs were prepared, and two representative test pits were selected and logged in detail. Geologic and agronomic descriptions used for characterization are given in Table 1. Bulk density and water content determinations were made at the surface and within two selected subsurface layers in one detailed test pit. A bulk sample was collected from one subsurface layer, and bag samples of major lithologies were collected from fresh spoil for laboratory testing.

Informal interviews were conducted with mining and regulatory personnel to supplement information collected during other portions of the program. Data on the following subjects were collected:

1. Rates, modes, and degrees of slaking of spoil materials;
2. Environmental effects of slaking;
3. Control measures to alleviate adverse environmental effects of slaking;
4. Mining techniques;
5. Geologic features; and
6. Final reclamation practices, including water and erosion control, fertilization, seeding, and plant methods.

FIELD OBSERVATIONS

Slaking Modes

Slaking in mine spoils is a function of several variables, including material properties, mining techniques, and reclamation procedures. Three modes of slaking were observed to occur in mine spoils: slaking to inherent grain size, chip slaking, and slab or block slaking. Slaking to inherent grain size results in the destruction of original rock structure and production of a sediment mass consisting of fine-grained particles. Disintegration of rocks by this slaking mode can occur in a time period that ranges from a few days to several years or longer.

Chip slaking results in the production of flat fragments that range in thickness from 0.6 to 2.0 cm and in length and width from 2.5 to 15.0 cm. Usually, the chips are relatively stable and resist further degradation. Initial breakdown occurs along subparallel planes of weakness that may not be apparent during visual inspection of fresh rock core. However, the existence of thin interbeds of contrasting mineralogy or chemical composition was found to be a reliable indicator of incipient chip slaking.

Slab or block slaking results in the production of thick slabs or approximately equidimensional blocks that range in size from about 7.5 cm to 1.8 m or more. Breakdown commonly occurs along natural or blasting-induced fractures. Once broken into blocks or slabs, the fragments are generally resistant to further breakdown or deteriorate slowly over time.

Inherent rock properties are a major factor in determining mode, rate, and degree of slaking. Lithology, bedding, and mineralogy were observed to influence the slaking characteristics of mine spoils. Mudstones, siltstones, and shales were found generally to be the most slakeprone lithologies. Slaking of sandstone and limestone was variable but generally minor in occurrence. Bedding characteristics primarily influence the mode of slaking; i.e., rocks that exhibited thin bedding were found to undergo chip slaking, whereas massive rock units were prone to slab or block slaking or slaked to their constituent grain size.

The percentage content of 2- to 20-mm-sized coarse fragments, which shows little variation with depth at all sites, seems to suggest that slaking of large coarse fragments can take place rapidly but that the rate of degradation may decrease substantially with smaller particle size. Hence, an equilibrium point may exist for many geologic materials when the 2- to 20-mm-sized fraction is reached. Slaking of large coarse fragments was observed to occur principally along natural zones of weakness such as bedding planes, fractures, and weakly cemented grains. It is believed that few zones of structural weakness exist in smaller fragments, which substantially reduces the rate of slaking.

Readily identifiable minerals also correlated with various slaking characteristics. For example, carbonate cemented clastic rocks often disintegrate rapidly when placed in spoils. Lithologies with interbeds of contrasting mineralogy, such as concentrations of iron oxides or coarse mica flakes, provide zones of weakness that augment the slaking process.

General relations between slaking modes and inherent rock properties are summarized in Table 2.

Physical Characteristics of Spoils

To understand the various parameters associated with deterioration of spoil materials, the field program examined variations in density and grain size among spoils of various ages. Bulk density of spoils is a function of placement methods, slaking characteristics, lithologic content, and age of spoils. Typical in situ bulk density and moisture content data are shown in Figure 2. Bulk density generally de-

Figure 1. Appalachian and Illinois Basins and approximate location of study sites.

Table 1. Descriptors used in characterization of mine spoil.

Location	Variable	Descriptor[a]
Recent and 2-, 5-, 10-year aged spoil areas	Spoil geomorphology	Outslope, bench, backfilled highwall, valley fill, rolling upland, other[b]
	Slope	Length, gradient, configuration, uniformity
	Slope stability	Surface creep, bulges, scars, failures
	Surface hydrology	Type (e.g., seeps, springs, ponded water), areal extent, flow, pH, depth
	Hydrologic structures	Type (e.g., sediment pond, diversion ditch), dimensions, drainage basin, topographic position, rock riprap
	Erosion	Type (e.g., sheet action, rills, gullies), rock pavement
	Surface condition	Disc marks, bulldozer or rubber tire tracks, other[b]
Test pit		
Surficial features	Vegetation	Species, cover, distribution, litter
	Surface condition	Rock mulch, desiccation cracks, crusting, heaving
	Surface materials	Lithotype, hardness, structure, color, mode of slaking
Subsurface features		
A	Stratification	Depth, thickness, coarse fragment content (percent), lithotypes (percent of each), texture (grain size), special features
B[c]	Stratification[d]	Pores and voids, roots, moist color (Munsell designations for matrix and mottles), structure, moist consistency, boundary, special features, bulk density, water content[e]

[a]Unless noted, any descriptor that required quantification (i.e., gradient) was estimated by visual observation. Soil descriptors were characterized according to standard U.S. Department of Agriculture methodology.
[b]When the descriptor did not fall within the preselected terminology, further definition was required.
[c]In one pit of each age spoil, samples were taken for point load testing, water contents, and bulk grain sizes.
[d]Compiled for two pits in each age spoil.
[e]Measured in one pit in each age spoil.

creases with depth in the fresh and 2-year-old spoils and increases with depth in the 5- and 10-year-old spoils. The higher surface densities in the younger spoils, particularly in the 2-year-old spoils, are a result of more intensive materials-handling activities that have crusted and compacted the surface layer, whereas the lower surface densities in the older spoils are believed to be a result of less machinery traffic. Ten-year-old spoils received minimal grading compared with younger spoils. Increasing density with depth may also indicate that settlement of the spoil materials has taken place.

Spoils exhibit a broad range of grain sizes, from boulders to clay. Because slaking, by definition, involves the physical degradation of rocks, a net decrease in grain size occurs. However, since slaking rates vary greatly, even spoils that have undergone extreme slaking of a particular lithology usually retain some coarse fragments of slake-resistant lithologies. A typical grain-size analysis is shown in Figure 3. Material properties apparently control grain-size distribution in these spoils. Generally, the proportion of fine-grained material increases with increasing spoil age. Younger spoils contain fewer coarse fragments due to increased handling, which, in turn, relates to the increased grading associated with restoration to approximate original contour. Older spoils generally required less grading.

Slaking was observed to occur as a function of depth in spoils of all ages, and, generally, evidence for the occurrence of slaking was negligible below 5 ft. Total coarse-fragment content generally increases with depth as does the proportion of larger fragments. The proportion of smaller coarse fragments typically decreases with depth whereas the distribution of intermediate-sized fractions is more variable.

Table 2. Relation of slaking modes to rock properties.

Slaking Mode	Associated Rock Properties
Slaking to grain size	Generally occurs in mudstones and occasionally in sandstones; rocks are usually massive and may be carbonate cemented
Chip slaking	Generally occurs in shales and siltstones, occasionally in thin bedded sandstones; bedding may be indistinct to well expressed and often micaceous
Slab or block slaking	Generally occurs in sandstones and limestones; rocks are usually massive and may be micaceous

Excavation and Placement Techniques

Mining techniques were observed to affect slaking characteristics in two ways. First, the type of mining used (e.g, area mining versus haulback) influences slaking. Second, the increase in equipment size, machinery traffic, and spoil handling associated with all mining methods has produced an attendant increased impact on slaking. These impacts include greater crushing and abrasion of spoil fragments as a result of increased material handling, more controlled placement, and more extensive final grading.

Blasting methods depend on both the mining method and the number of seams mined. Blasting directly affects slaking in at least three ways: total disintegration of the rock into its constituent particles, partial breakdown of the rock into smaller

Figure 2. Dry bulk density and water content versus depth for spoils of different ages.

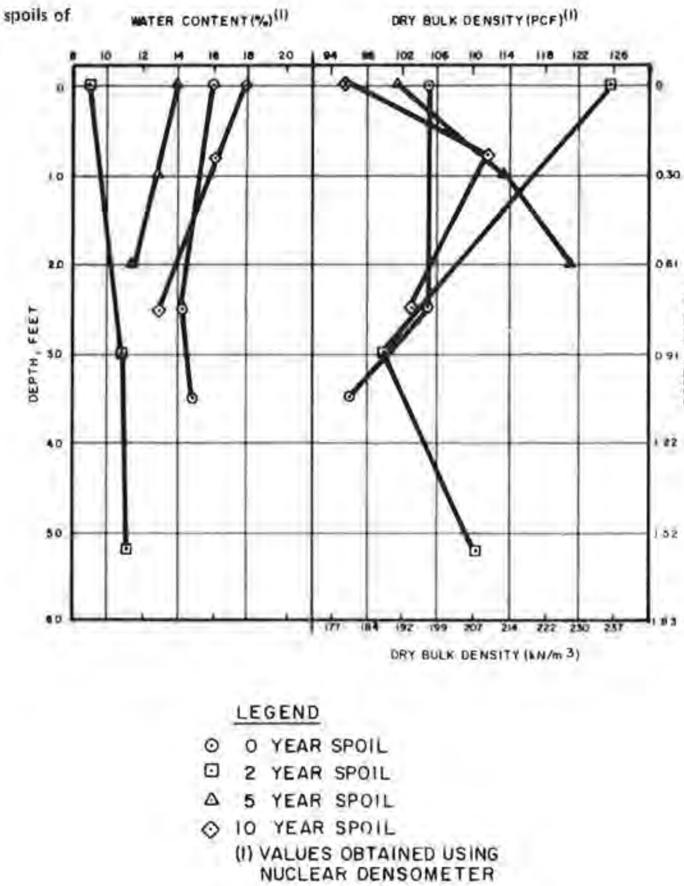
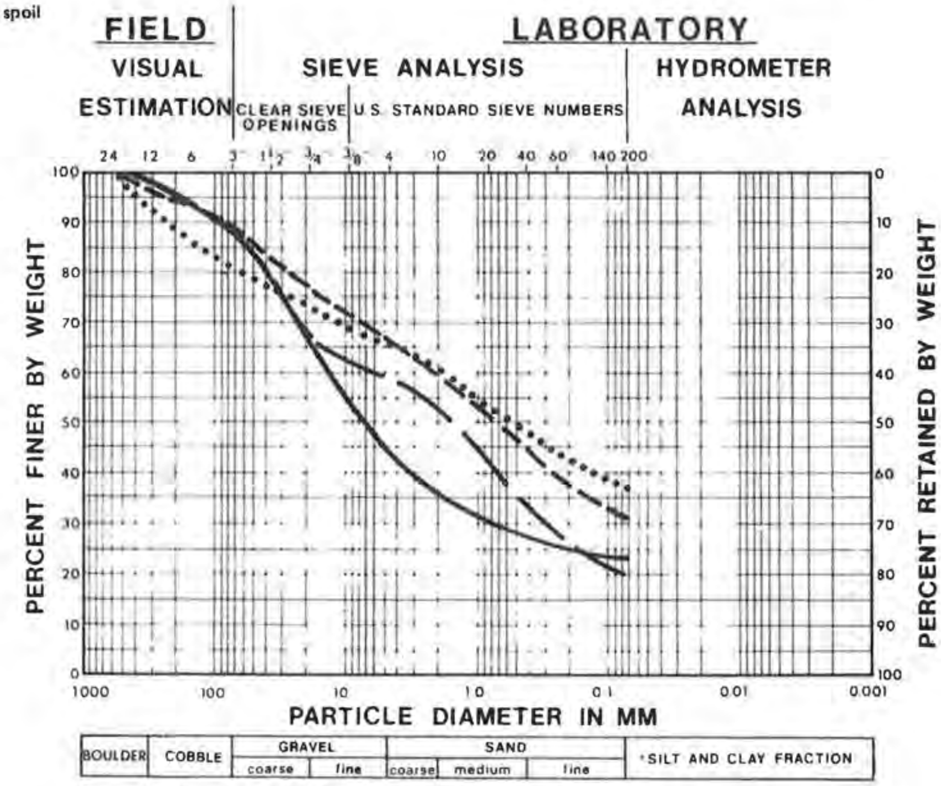


Figure 3. Grain-size distribution in a spoil pile.



SYMBOL	SPOIL AGE YEARS	DEPTH IN	DEPTH CM	COMPOSITION
——	0	36	91	15 % SANDSTONE ; 25 % SHALE ; 60 % MUDSTONE
— —	2	60	152	20% LIMESTONE ; 30% SHALE ; 60% MUDSTONE
----	5	36	91	10 % BLACK SHALE ; 30 % SHALE ; 60 % MUDSTONE
••••••	10	48	122	10 % SANDSTONE ; 20% SHALE ; 70 % MUDSTONE

NOTE: COMPOSITION IS BASED ON PARTICLE DIAMETERS > 5/64 INCH (2mm)

fragments, and disruption and weakening of bonds within the rock without a decrease in fragment size. The first mechanism is responsible for the production of dust, whereas the remaining two mechanisms provide suitable conditions for slaking to proceed during excavation, placement, and reclamation activities.

Excavation and placement techniques also affect the slaking process. Spoil placement in dragline operations consists of dumping in parallel ridges with successive lifts compacted only by the weight of the overlying spoil. The porous character of the spoil ridges tends to be open, accessible to air and water; therefore, slaking can be initiated after placement.

Spoil placement in haulback and mountaintop removal is usually accomplished by using large trucks. Spoil loading, transport, and dumping can cause crushing and abrasion of materials. In addition, placed spoils located in access areas sustain heavy truck traffic, which results in further crushing, grinding, and compaction. Therefore, the spoils that result from this type of placement are often denser and less susceptible to atmospheric agents below the surface.

Final grading to approximate the original contour often involves considerable reworking of the spoil and may have the greatest impact on slaking of the entire mining and reclamation program. This conclusion is based on inspection of graded and ungraded fresh spoil and comments from mining personnel. Ungraded fresh spoil appeared to have a coarser particle-size distribution than graded materials. In addition, rocks were observed to be broken, cracked, or crushed during actual grading. Finally, when topsoiling is conducted, machinery passes further accelerate material breakdown.

Effects of Slaking

Slaking of mine spoils produces a variety of associated effects. Some, such as accelerated erosion, may adversely affect local environmental quality. Slaking-related environmental effects observed in the field program included the following: pebble pavement formation on the spoil surface; development of a surface crust; variations in sheet, rill, and gully erosion; vegetation impacts; variations in slope stability; and changes in spoil hydrology.

The presence of a pebble pavement on the spoil surface was characteristic of all sites studied. The size of fragments ranged from about 0.5 to 50 cm but was commonly 1-8 cm. The pebble pavement usually consisted of fragments of slake-resistant lithologies such as limestone or strongly cemented sandstone. Pebble-pavement cover ranged from as little as 30 to virtually 100 percent areal cover. Total or near-total pebble cover appeared to assist erosion control, moisture conservation, and seed germination.

Spoils that had not been topsoiled commonly exhibited a crusted layer immediately beneath the pebble pavement. Crusts usually consisted of materials that had slaked to their inherent grain size and were from 1 to 13 cm thick. The thickness and compaction of the crust are a function of the proportion of materials present that slake to constituent grain size, machinery traffic, and moisture content. Development of a surface crust appeared most pronounced where mudstone materials composed a large amount of the surface spoil. The presence of durable fragments appeared to partly offset compaction and crust formation, as did the establishment and continued growth of vegetation with vigorous root systems.

Sheet, rill, and gully erosion were observed to occur to varying degrees on all spoils. Spoils containing a high proportion of materials that slake to their inherent grain size exhibited the most severe erosion. Conversely, spoils with a high percentage of slake-resistant rocks were least affected by erosion. The presence of a durable pebble pavement was observed to be effective in controlling sheet erosion. Concentrated surface flow created numerous rills and occasional deep gullies, particularly on spoils that slake to their inherent grain size. Riprap that is composed of stable materials prevents down-cutting, whereas slakable riprap typically yields V-shaped ditches with active down-cutting. Off-site damage can occur as a result of slaking in the form of increased sediment loads and stream siltation, particularly when materials slake to their inherent grain size.

Slaking may produce both adverse and beneficial effects on vegetation. Adverse effects include dense crusting of the surface, which impedes seed germination, root proliferation, and moisture infiltration. Beneficial effects include the production of fine particles, which results in a suitable blend of coarse and fine materials to provide physical support to plants, allow easy root proliferation, and store sufficient amounts of plant-available water. It is also probable that slaking increases the levels of plant-available nutrients. Slaking involves a decrease in fragment size, which increases the surface area so that more material is exposed to chemical weathering and subsequent mobilization of plant nutrients.

Slaking can affect embankment stability. Major causes of embankment instability include excessive lift thickness, which results in high void ratios that can enhance the amount of future settlement; inadequate compaction; physical and chemical deterioration (primarily of argillaceous materials); expansive characteristics related to the mineralogy of the fill materials; inadequate drainage; and excessive side slopes near the angle of repose (5). In our study, slope stability was also found to be related to the mode of slaking. Little or no stability problem was found where slab or block slaking dominated. Where chip slaking was dominant, the mass appeared to be relatively stable. The chips form an interlocking matrix that is resistant to bulk movement. When slaking to inherent grain size was found to be the primary mode, stability problems were observed, as evidenced by slips, slides, and similar features. Degradation processes that control settlement and stability behavior may also be influenced by excavation and placement techniques. In general, methods that involve some mechanical crushing and compacting will most likely result in less deterioration following placement than those that involve disposal by simple dumping. Increased crushing and compaction increase bulk density, thereby reducing contact of the geologic materials with air and water. In conjunction with compaction, reclamation procedures that involve proper drainage and revegetation will minimize the influence of slaking on stability and settlement.

Spoil hydrology and its interaction with slaking were observed to be a complex process. Contrary to the belief that spoils are highly permeable and free draining and retain little water, spoils were found to be effective reservoirs. Perched water tables were found in several test pits within 1.5 m of the surface. Within spoil profiles, water moves by three means: capillary action, gravitational drainage, and vapor transport. Vapor transport could not be directly observed in the field, but evidence of capillary and gravitational movements was seen. The presence of a wetting front was noted in several test pits excavated immediately after a rain shower. Gravitational drainage was observed in large pores and voids in numerous profiles.

Precipitation in the Appalachian Basin is fairly well distributed throughout the year. Consequently, cyclic wetting and drying appear to be confined to approximately the upper 1.5 m of the spoil profile, and extreme desiccation is limited to the top 0.5 m. Below this zone, the moisture regime in mine spoils appears to be relatively stable. The depth of cyclic wetting and drying appears to coincide with the depth of observed slaking in many spoil profiles.

SUMMARY AND CONCLUSIONS

Slaking is a short-term, physical disintegration process that may occur in some geologic materials following excavation. The rate and degree of disintegration are directly related to material characteristics and local environmental conditions. Argillaceous materials with closely spaced bedding planes, principally shales and siltstones, may deteriorate rapidly to small chips when exposed. Soft, weakly consolidated argillaceous materials, principally mudstone, may experience almost complete disintegration to constituent particles shortly after exposure. Many sandstones are strongly cemented and composed of relatively inert particles that weather slowly in relation to slaking processes. Sandstone disintegration is therefore typically not a significant factor in slaking and is generally limited to the production of slabs or blocks. Argillaceous limestones with a significant number of planes of weakness may also be susceptible to disintegration through chip, slab, or block slaking, but solution-weathering phenomena that are predominant in more massive crystalline limestone are relatively slow and are not considered to be a part of the slaking process.

Although slaking is caused by many complex and interrelated variables, certain factors are important in determining the slaking potential of geologic materials; these factors include inherent particle size, mineralogy, rock fabric, spoil fragment size, and local hydrologic conditions. Inherently fine-grained sediments appear to be more susceptible to breakdown and at higher rates than coarse-grained sediments. In conjunction with particle-size effects, the mineralogical composition of argillaceous sediments affects the nature, degree, and rate of slaking. Small amounts of active clay minerals can influence material behavior because of interlayer water absorption and the type of ions adsorbed on exchange sites. Environmental effects associated with slaking include embankment instability, erosion and sedimentation, vegetation, and hydrologic impacts. However, these impacts can be alleviated through application of appropriate control measures that minimize the adverse effects of slaking.

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REFERENCES

1. American Geological Institute. Dictionary of Geological Terms. Anchor Press, Doubleday and Co. Inc., Garden City, NY, 1962, 545 pp.
2. A.F.S. Nettleton. An Investigation into a Physico-Chemical Aspect of the Slake Durability Weathering Test. Univ. of New South Wales, Kensington, New South Wales, Australia, UNICIV Rept. R-128, 1974, 3 pp.
3. N.R. Morgenstern and K.D. Eigenbrod. Classification of Argillaceous Soils and Rock. Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No. GT10, 1974, pp. 1137-1156.
4. D.E. Andrews, J.L. Withiam, E.F. Perry, and H.L. Crouse. Environmental Effects of Slaking of Surface Mine Spoils: Eastern and Central United States. Bureau of Mines, U.S. Department of Interior, Denver, CO, Final Rept., 1980, 247 pp.
5. J.H. Shamberger, D.M. Patrick, and R.J. Lutton. Design and Construction of Compacted Shale Embankments: Volume 1--Survey of Problem Areas and Current Practices. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Rept. FHWA-RD-75-61, Interim Rept., 1975, 288 pp.

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Statistical Analysis of Shale Durability Factors

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The results of a study performed to develop durability tests for shales for use in embankments and swamp fills are presented. Forty-three shale samples, representing all exposed shale units in Ontario, were collected. The samples were subjected to a variety of tests, some standard soundness tests, some recently developed shale durability tests, and some special tests devised by the author. The tests included freeze-thaw durability, Franklin slake test, wet-dry deterioration, modified "rate of slaking", water adsorption at controlled humidities, water absorption, abrasion, and dry bulk density. The results of the tests were statistically analyzed. The principal analytic method was multivariate stepwise regression analysis, but other statistics were also obtained. The stepwise regression analysis picks the testing procedures that have the greatest influence on the desired property and produces a multitest equation that can be used to determine that property. A series of seemingly unrelated tests can thus be used to determine the durability of shales. For instance, wet-dry deterioration is related to and can be calculated from water absorption, freeze-thaw, and abrasion results. The results obtained apply to the variety of shales found in southwestern Ontario. A larger data base may be necessary to extend the conclusions to all shales as a group.

Shale as a construction material has always been shunned. Its tendency to "slake", or turn to mud, is too well known in the construction industry. However, not all shale slakes equally, and some shale is rocklike--i.e., very little affected by weathering. Shale exists in many different forms, ranging from soft mudstone to indurated hard slate. The behavior of shale cannot be predicted from its composition, since the main constituent of any shale is clay. The amount of clay, the degree of natural compaction, cementation, and organic content all have a bearing on the physical properties of shale.

The purpose of the research described in this paper, supported by the Ontario Ministry of Transportation and Communications, was to develop durability tests for shale material and to classify Ontario shales as to their durability. Ontario shales are no less varied than shales found elsewhere. Some are highly susceptible to weathering, whereas others are rocklike in all aspects. All varieties of exposed shale across the province were collected--some from brick quarries, some from fresh road cuts, some from weathered road cuts. Although shales from weathered road cuts are not as desirable, they were included because no other examples of shales in that locality were available. The 43 different samples obtained represented all formational shale units found in the province. The formational units sampled and the number of samples representing the unit are given below. The total number of samples permits statistical treatment of data, which is the main purpose of this paper.

Shale Unit	No. of Samples
Upper Devonian, Kettle Point (black shale)	1
Middle Devonian, Hamilton Group (grey shale)	4
Mid-Low Silurian	
Clinton-Cataract Group	7
Decew dolomite	
Cabbot Head shale	
Rochester shale	
Billings shale	
Upper Ordovician	
Queenston formation (red shale)	10
Georgian Bay formation	11
Dundas shale	
Blue Mountain shale	
Meaford shale	
Whitby formation	3
Precambrian (shale and slate)	7
Rowe formation	
Gunflint formation	
Sibley shale	

CAUSES OF SHALE DETERIORATION

Shale and water interaction, with or without attendant freezing, can be considered as the main cause of shale deterioration. The actual processes involved in the breakdown are still not fully understood; however, it is known that shale-water interaction causes expansion of the shale. This expansion may be due to a number of factors: (a) "structuring" of water on clay surfaces, (b) osmotic pressure, and (c) frost action.

The structuring of water in clay-water mixtures causes the well-known thixotropic effect, in which the viscosity of the water increases as the water becomes oriented and more structured as the clay surface is approached. Shales have a high internal surface area, which is measured in terms of square meters per gram. Water molecules adsorbed by these surfaces tend to completely fill the internal pores. Some believe that the adsorbed water may become strongly structured under the influence of the surface and may increase in specific volume. Volume expansion of water in turn causes volume expansion of the rock.

Expansion may be due to osmotic pressures generated within the rock. The pore size of shale is extremely small, less than 1 µm in diameter. Water in such pores has a lower vapor pressure over it than does bulk water. Water, therefore, is impelled to enter the rock, which creates pressure within the pore and causes the pore and the rock to expand. This mechanism differs from the one above only in the method of expansion; both rely on surface-held water to initiate the force.

Saturating the clay with water causes the clay to expand. Conversely, drying the clay causes it to contract as the capillaries filled with water get progressively smaller and exert increasingly higher tensional stress on the pore walls. Alternate wetting and drying of some shales (and some argillaceous rocks) causes their disintegration.

Freezing and thawing can be considered principally a wetting and drying process, since very little contained water in the small shale pores actually freezes. However, cooling the pore water to below freezing temperatures does significantly lower its vapor pressure, possibly below the vapor pressure of ice at equivalent temperature, and causes water vapor transfer from water in larger pores, ice, or bulk water to the smaller pores, setting up expansion. Expansion of cement gel under slow cooling conditions (without any or only minor freezing) has been observed. Cement gel as a system is analogous to a clay-shale system.

Based on the above, it is evident that cyclic wetting and drying of shale or freezing and thawing will result in cyclic application of tensional and compressional stress between pore walls and between clay particles. As the shale forms, the clay particles are often simply pressed together and dewatered by the weight of the overlying sedimentary column. The removal of the confining pressure and immersion in water result in the complete disintegration of the shale particles to the original mud.

The presence of cement containing $CaCO_3$, SiO_2, and various hydrous iron compounds tends to prevent the particle dispersal on unloading. However, in some shales, the tensional stress of water-clay interaction is great enough to overcome the tensional strength of such cement, and the shale disintegrates.

Shales subject to metamorphism are often indurated and recemented and tend to be quite resistant to water-induced stresses. However, such shale, because of extreme alignment of clay particles or their metamorphic equivalents, exhibits pronounced parallel and subparallel cleaving. Such material is no longer shale but slate.

SELECTION AND DESCRIPTION OF TESTS

Based on the above considerations, any test that accurately measures the disintegration of shale due to cyclic wetting and drying (and freezing and thawing) or a test that measures the amount of the tightly held surface, pore, and capillary water will, therefore, be a direct indicator of the durability of the shale. Statistical evaluation will show the relation between tests and the contribution of each test to the overall durability of the rock.

The following tests were carried out in this research:

1. Franklin slake,
2. Rate of slaking,
3. Wet-dry deterioration,
4. Water adsorption (at 45 and 95 percent relative humidity),
5. Abrasion,
6. Density, and
7. Freezing and thawing resistance.

A very brief description of each of these tests is given below. In all tests, the sample was crushed to -19-mm, +9.5-mm fragments. In the Franklin slake test and the wet-dry deterioration test, a five-cycle sequence was performed and losses were determined at the end of each cycle. One-, three-, or five-cycle results were used in later statistical analyses.

Franklin Slake Test

The Franklin slake test was first described by Chandra (1) and Franklin and Chandra (2) and was adopted by the Commission of Standardization of Laboratory and Field Tests of the International Society of Rock Mechanics.

The test uses a steel mesh drum with 2-mm-diameter openings. The sample is placed in the drum, and the drum is rotated while submerged in water for 10 min and 200 revolutions. The slaking products fall through the mesh and the remainder is removed, dried, and weighed to determine the lost mass.

Wet-Dry Deterioration Test

The process of wetting and drying is the most commonly observed cause of deterioration of shales. The simplest way to determine the resistance of shale to wetting and drying is to do just that--alternately wet and dry the shale for a given number of cycles. In this case, 500 g of crushed and sized sample was placed in mason jars, saturated for 8 h, decanted, and dried for 16 h at 65°C. Drying at 110°C was avoided to prevent possible alteration of the clay structure. Loss was determined on the 9.5-mm sieve.

Rate of Slaking

The rate of slaking test, first developed by Morgenstern and Eigenbrod (3), consists of saturating the sample contained in a funnel with a filter paper. The water absorbed by the shale (after free water has filtered through the paper) is determined. The second part of the test consists of determining the Atterberg limits on the slaked portion and the crushed and sieved unslaked portion of the sample. Only the first part of the test--i.e., the absorption--was done. The test, whether in its original or modified form, is misnamed, since no time measurement is involved. It is simply an absorption test.

Adsorption at 45 and 95 Percent Relative Humidity

Shales rapidly adsorb water on their internal surfaces when exposed to moisture in the air. The adsorbed water fills the very small pores and partially fills the larger capillaries. Expansion of the shale due to adsorbed water alone has been noted. Thus, a test designed to measure the amount of adsorbed water will indicate the proportion of fine pores in the shale. The two humidities used give some indication of pore-size distribution. The adsorption test does not directly indicate the size

Table 1. Test results.

Sample No.	FRZTHW12 (%)	FRZTHW25 (%)	SLAKE1 (%)	SLAKE3 (%)	WETDRY1 (%)	WETDRY3 (%)	SLAKEADS (%)	ADSGLYC (%)	ADSSULF (%)
1	31.58	64.36	0.62	1.70	0.00	1.31	3.12	0.34	0.45
2	36.16	69.77	0.55	1.28	0.00	0.18	2.97	0.26	0.31
3	36.18	73.63	0.68	1.35	0.04	0.42	1.88	0.23	0.34
4	89.89	98.47	8.04	40.53	16.89	67.80	8.04	0.50	0.63
5	49.04	74.95	0.91	4.47	1.18	4.62	2.63	0.25	0.38
6	97.54	99.32	16.31	57.73	22.83	84.41	9.21	0.48	0.70
7	99.33	100.00	43.04	84.99	31.24	81.95	10.82	0.78	
8	38.67	75.04	0.80	1.89	0.00	0.62	3.09	0.31	0.41
9	56.23	79.76	0.96	2.52	0.00	0.79	3.13	0.38	0.43
10	98.09	100.00	35.95	78.18	43.47	81.37	9.63	0.62	0.95
11	96.44	99.90	28.78	82.63	38.48	80.54	10.72	0.69	0.91
12	34.05	62.72	1.43	3.45	0.00	1.15	4.48	0.24	0.37
13	31.95	66.67	0.86	1.84	0.01	3.26	3.24	0.32	0.45
14	46.00	90.67	1.05	2.40	0.07	4.21	3.30	0.32	0.46
15	29.19	64.57	0.89	2.19	0.00	0.80	2.88	0.32	0.42
16	3.82	9.14	0.22	0.60	0.00	0.96	1.63	0.14	0.09
17	99.45	100.00	62.82	93.72	82.50	93.38	25.55	0.33	
18	96.72	100.00	47.03	94.91	89.92	100.00	23.18	0.36	0.74
19	100.00	100.00	71.74	99.06	98.04	100.00	23.31		
20	1.78	2.31	0.24	0.57	0.00	0.00	1.66	0.09	0.14
21	96.16	100.00	54.08	89.13	82.41	95.32	20.66	0.68	0.58
22	92.06	96.16	57.65	63.11	63.39	68.77	17.20	0.13	0.21
23	97.76	99.30	68.30	85.21	71.80	83.33	25.97	0.37	0.52
24	91.43	97.06	39.91	59.73	35.53	50.11	20.27	0.32	0.67
25	68.91	81.77	8.23	17.50	2.65	6.61	5.44	0.19	0.28
26	34.29	54.64	0.83	1.97	0.03	0.23	2.09	0.13	0.21
27	100.00	100.00	81.76	97.36	98.98	100.00	37.72		
28	100.00	100.00	81.30	99.76	100.00	100.00	36.02		
29	48.73	87.12	1.08	2.70	0.03	0.32	3.33	0.32	0.47
30	100.00	100.00	89.04	100.00	100.00	100.00	32.28		
31	18.17	37.29	0.56	1.59	0.00	0.30	3.74	0.59	0.85
32	85.64	96.38	4.15	21.39	11.20	54.49	6.60	0.77	1.27
33	97.51	100.00	3.14	9.32	1.15	9.91	3.09	0.77	0.96
34	96.85	100.00	5.86	17.34	7.28	18.73	4.06	0.83	0.92
35	86.18	98.82	2.43	18.18	3.60	32.14	5.09	0.63	1.03
36	62.89	94.07	1.28	4.16	1.70	5.58	4.37	0.81	1.11
37	22.66	40.92	0.31	0.56	0.00	0.11	5.01	0.67	0.92
38	24.66	59.08	0.18	0.65	0.00	0.18	2.77	0.93	1.04
39	19.26	65.32	0.16	0.34	0.00	0.25	3.54	1.30	1.50
40	40.06	85.97	0.31	0.69	0.00	0.11	1.39	0.36	0.45
41	47.06	88.69	0.55	2.32	0.00	1.46	2.43	0.47	0.55
42	11.37	21.21	0.01	0.17	0.00	0.00	2.27	0.82	1.00
43	22.86	52.28	0.63	1.34	0.00	0.00	2.39	0.51	0.60

of pores as does mercury intrusion porosimetry; it does, however, differentiate between pores of less and greater diameter than approximately 5 μm.

The test consists of placing a dry shale (dried for 24 h at 65°C and cooled over a dessicant) into a chamber where humidity is maintained at desired level. The humidity is controlled by saturated salt solutions. After 72 h, near equilibrium is established; that is, the sample has adsorbed almost all the moisture it can hold at that humidity. The weight gain is the measure of adsorbed water.

Abrasion

The strength of the natural cement holding the clay particles determines, in part, how well the shale is able to withstand the expansive forces of wetting. Any test designed to physically "pry" the particles apart will measure the relative strength and amount of the cement. Abrasion can be considered as prying apart surface particles from the body of the rock. To simulate abrasion, the Franklin slaking machine was used. The dry shale sample, along with five steel balls of 19-mm diameter, was rotated at 20 rpm for 400 revolutions. Loss of weight was a measure of abrasion. Although the test is similar to the Los Angeles abrasion test, the intensity of abrasion is approximately one magnitude lower than in the Los Angeles abrasion test.

Density

Since shales consist of clay particles as principal constituents and clays have similar specific gravity, the density of the sample determines the degree of compaction and/or induration with cementing materials. Because of the slaking nature of the shale, water cannot be used as a displacement medium for volume determination. A mercury displacement technique with a specially designed mercury volumeter was used.

Freeze-Thaw Resistance

Shale is exposed to freezing and thawing in the northern latitudes and at higher altitudes. However, before freezing can do any damage the shale must be wet. Thus, a component of wetting and drying deterioration is incorporated into the freeze-thaw loss. However, freezing does contribute to the deterioration of the shale.

A freeze-thaw test designed by the author consisted of placing the sample in a mason jar, saturating it in 5 percent sodium chloride solution (to accelerate frost action) for 2 min (to minimize slaking), and draining. The jar was capped to prevent moisture loss, frozen at -20°C for 15-17 h, and thawed for 5-7 h. Loss of mass on the 9.5-mm sieve was determined after 12 and 25 cycles, respectively.

TEST RESULTS

The following designations for tests are used throughout the paper:

FRZTHW12 = 12-cycle freeze-thaw test, loss of mass (%),

ADSSLAKE (%)	ADS45 (%)	ADS95 (%)	ABRASION (%)	DENSITY (g/cm^3)
0.31	0.37	1.01	0.36	2.61
0.24	0.31	0.79	0.32	2.57
0.26	0.29	0.70	0.39	2.65
0.66	0.84	1.93	0.53	2.58
0.30	0.43	1.00	0.36	2.65
0.80	0.92	2.07	0.53	2.59
1.03	1.07	2.41	0.83	2.48
0.35	0.36	0.86	0.37	2.57
0.36	0.38	0.93	0.39	2.59
0.76	0.94	2.41	0.55	2.51
0.75	0.94	2.54	0.67	2.53
0.22	0.25	0.68	0.71	2.49
0.32	0.31	0.77	0.44	2.65
0.33	0.35	0.87	0.45	2.63
0.41	0.35	0.67	0.36	2.63
0.15	0.22	0.64	0.42	2.36
0.43	0.93	2.71	0.62	2.42
0.35	0.94	2.74	1.23	2.43
	1.00	3.03	0.72	2.45
0.09	0.14	0.37	0.34	2.43
0.49	0.97	3.17	0.77	2.40
0.21	0.61	1.86	0.91	2.78
0.32	1.33	3.47	2.73	2.45
0.44	0.07	1.99	2.04	2.65
0.23	0.44	1.12	0.83	2.65
0.13	0.14	0.36	0.43	2.46
	0.71	1.95	0.63	2.49
	0.74	2.00	0.75	2.56
0.30	0.33	0.80	0.77	2.48
	0.81	1.93	0.71	2.29
0.56	0.00	1.52	0.36	2.63
1.07	1.16	1.89	1.31	2.67
0.82	0.84	1.32	0.95	2.68
0.94	0.92	1.49	0.65	2.57
0.91	0.87	1.63	1.00	2.41
0.91	0.92	1.80	0.48	2.32
0.76	0.84	1.57	0.59	2.32
1.08	1.05	1.55	0.41	3.19
1.45	1.07	2.54	0.62	2.60
0.39	0.37	0.62	0.24	2.71
0.48	0.49	0.85	0.36	2.65
1.05	0.89	1.74	0.26	2.72
0.56	0.50	0.88	0.40	2.66

FRZTHW25 = 25-cycle freeze-thaw test, loss of mass (%),
SLAKE1 = 1-cycle Franklin test, loss of mass (%),
SLAKE3 = 3-cycle Franklin test, loss of mass (%),
WETDRY1 = 1-cycle wet-dry test, loss of mass (%),
WETDRY3 = 3-cycle wet-dry test, loss of mass (%),
SLAKEADS = rate of slaking absorption test, gain (%),
ADSGLYC = glycerine solution slaked, adsorption at 45 percent relative humidity (%),
ADSULF = sulphate solution slaked, adsorption at 45 percent relative humidity (%),
ADSSLAKE = water slaked, adsorption at 45 percent relative humidity, gain (%),
ADS45 = adsorption on original sample at 45 percent relative humidity, gain (%),
ADS95 = adsorption on original sample at 95 percent relative humidity, gain (%),
ABRASION = dry abrasion in Franklin slaking machine (%), and
DENSITY = mercury immersion density (g/cm^3).

The test results are given in Table 1.

STATISTICAL ANALYSIS

The results obtained were analyzed by using both simple and multivariate statistical techniques. The purpose of the statistical analysis was to establish relations between tests and, through multivariate statistics, to determine the factors that affect the durability of shales. The summary statistics for the tests are given in the table below. The mean obtained is not particularly instructive, and the standard deviation simply implies the range or dispersal of the results about the mean.

Variable	N	Mean	Standard Deviation
FRZTHW12	43	61.3	33.7
FRZTHW25	43	78.8	26.1
SLAKE1	43	19.2	28.3
SLAKE3	43	31.4	39.1
WETDRY1	43	23.4	35.7
WETDRY3	43	33.4	40.9
SLAKEADS	43	9.4	10.2
ADSGLYC	39	0.5	0.3
ADSSULF	37	0.6	0.3
ADSSLAKE	39	0.5	0.3
ADS45	43	0.6	0.3
ADS95	43	1.6	0.8
ABRASION	43	0.7	0.5
DENSITY	43	2.6	0.2

Correlation Matrix

The "best-seven" correlation matrix for the tests is given in Table 2. Each test is taken in turn as a dependent variable, and the seven other tests with the best correlation are ranked against it. The confidence level number is given as a ratio and should be multiplied by 100 to give percentage confidence limits. Thus, 0.0001 indicates that 0.01 percent or less of the variation is unexplained.

In Figures 1-9, some of the tests that showed high correlation are graphed to show the nature of the relation between tests. The equation of the line of best fit through the points was calculated and used to derive classification limits for each test. The equations are given in Table 3. The classification is somewhat arbitrary, in that more or less arbitrary limits were established for the five-cycle Franklin slake test, and these limits then were substituted into the equations to obtain equivalent limits for the other tests. The Franklin test limits are given in Table 4 and the derived limits in Table 5. The tests given in Table 5 are arranged in order of reliability or applicability. The Franklin and wet-dry tests, however, are roughly equivalent in results and reliability and can readily substitute for each other.

Multivariate Statistics

The bivariate relation between tests described in the previous section can compare the relations between the two tests involved. However, the shale, like any material, has an inherent set of properties that respond somewhat differently to each test. Or to put it in another way, each test emphasizes a different property, or a mix of different properties, of the shale. To determine the interrelation of the tests, a multivariate statistical approach was used. The best stepwise regression technique was chosen (since this method involves choosing a dependent variable property of a test), against which other tests are compared. The test with the greatest correlation is chosen first. Other tests are chosen that, in turn, improve the correlation or the "model". In this stepwise fashion, an equation is developed that relates the tests to each other and to the dependent variable.

Two methods of stepwise regression were used. In one, all the tests were compared with the selected dependent variable. In the second, the closely related tests were excluded to better evaluate the contribution of nonrelated tests to the model. The

Table 2. Correlation matrix.

Test	Item	1	2	3	4	5	6	7
FRZTHW12	Test	FRZTHW25	WETDRY3	SLAKE3	SLAKE1	WETDRY1	ADS95	SLAKEADS
	Correlation coefficient	0.889	0.851	0.826	0.722	0.720	0.698	0.697
	Confidence limit	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	No. of samples	43	43	43	43	43	43	43
FRZTHW25	Test	FRZTHW12	WETDRY3	SLAKE3	ADS95	SLAKE1	WETDRY1	SLAKEADS
	Correlation coefficient	0.889	0.645	0.619	0.541	0.531	0.531	0.520
	Confidence limit	0.0001	0.0001	0.0001	0.0002	0.0002	0.0002	0.0004
	No. of samples	43	43	43	43	43	43	43
SLAKE1	Test	WETDRY1	SLAKEADS	SLAKE3	WETDRY3	FRZTHW12	ADS95	FRZTHW25
	Correlation coefficient	0.976	0.969	0.942	0.894	0.722	0.701	0.531
	Confidence limit	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0002
	No. of samples	43	43	43	43	43	43	43
SLAKE3	Test	WETDRY3	WETDRY1	SLAKE1	SLAKEADS	FRZTHW12	ADS95	FRZTHW25
	Correlation coefficient	0.979	0.946	0.942	0.903	0.826	0.802	0.619
	Confidence limit	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	No. of samples	43	43	43	43	43	43	43
WETDRY1	Test	SLAKE1	SLAKEADS	SLAKE3	WETDRY3	ADS95	FRZTHW12	FRZTHW25
	Correlation coefficient	0.976	0.963	0.946	0.915	0.729	0.720	0.531
	Confidence limit	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0002
	No. of samples	43	43	43	43	43	43	43
WETDRY3	Test	SLAKE3	WETDRY1	SLAKE1	SLAKEADS	FRZTHW12	ADS95	FRZTHW25
	Correlation coefficient	0.979	0.915	0.894	0.866	0.851	0.810	0.645
	Confidence limit	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	No. of samples	43	43	43	43	43	43	43
SLAKEADS	Test	SLAKE1	WETDRY1	SLAKE3	WETDRY3	FRZTHW12	ADS95	FRZTHW25
	Correlation coefficient	0.969	0.963	0.903	0.866	0.697	0.670	0.520
	Confidence limit	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0004
	No. of samples	43	43	43	43	43	43	43
ADSGLYC	Test	ADSSLAKE	ADSSULF	ADS45	ADS95	SAMPNO	FRZTHW25	DENSITY
	Correlation coefficient	0.958	0.940	0.687	0.493	0.481	0.176	0.153
	Confidence limit	0.0001	0.0001	0.0001	0.0014	0.0019	0.2829	0.3509
	No. of samples	39	37	39	39	39	39	39
ADSSULF	Test	ADSSLAKE	ADSGLYC	ADS45	ADS95	SAMPNO	FRZTHW25	FRZTHW12
	Correlation coefficient	0.953	0.940	0.696	0.555	0.504	0.259	0.250
	Confidence limit	0.0001	0.0001	0.0001	0.0004	0.0015	0.1222	0.1352
	No. of samples	37	37	37	37	37	37	37
ADSSLAKE	Test	ADSGLYC	ADSSULF	ADS45	ADS95	SAMPNO	FRZTHW12	FRZTHW25
	Correlation coefficient	0.958	0.953	0.706	0.469	0.440	0.205	0.199
	Confidence limit	0.0001	0.0001	0.0001	0.0026	0.0050	0.2106	0.2240
	No. of samples	39	37	39	39	39	39	39
ADS45	Test	ADS95	ADSSLAKE	ADSSULF	ADSGLYC	WETDRY3	FRZTHW12	SLAKE3
	Correlation coefficient	0.797	0.706	0.696	0.687	0.588	0.583	0.530
	Confidence limit	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0003
	No. of samples	43	39	37	39	43	43	43
ADS95	Test	WETDRY3	SLAKE3	ADS45	WETDRY1	SLAKE1	FRZTHW12	SLAKEADS
	Correlation coefficient	0.810	0.802	0.797	0.729	0.701	0.698	0.670
	Confidence limit	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	No. of samples	43	43	43	43	43	43	43
ABRASION	Test	ADS95	FRZTHW12	SLAKEADS	SLAKE3	SLAKE1	WETDRY3	FRZTHW25
	Correlation coefficient	0.557	0.528	0.477	0.462	0.456	0.449	0.423
	Confidence limit	0.0001	0.0003	0.0012	0.0018	0.0021	0.0025	0.0047
	No. of samples	43	43	43	43	43	43	43
DENSITY	Test	WETDRY1	SLAKE3	SLAKEADS	WETDRY3	SLAKE1	ADS95	FRZTHW12
	Correlation coefficient	−0.338	−0.337	0.322	−0.322	−0.319	−0.227	−0.227
	Confidence limit	0.0266	0.0269	0.0352	0.0355	0.0373	0.1427	0.1439
	No. of samples	43	43	43	43	43	43	43

Note: The seven test variables ranged across the table are from greatest to least correlation.

Figure 1. Slake loss versus wet-dry loss.

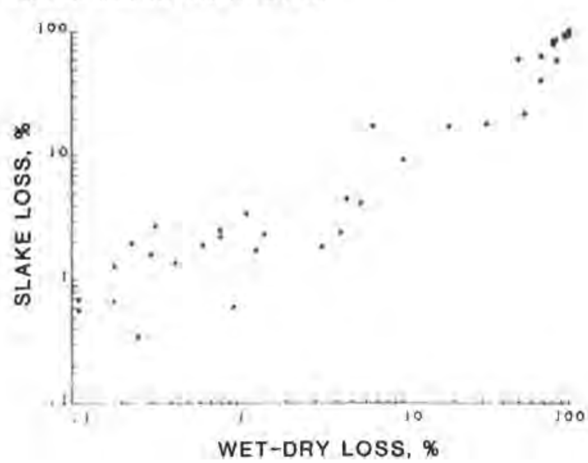
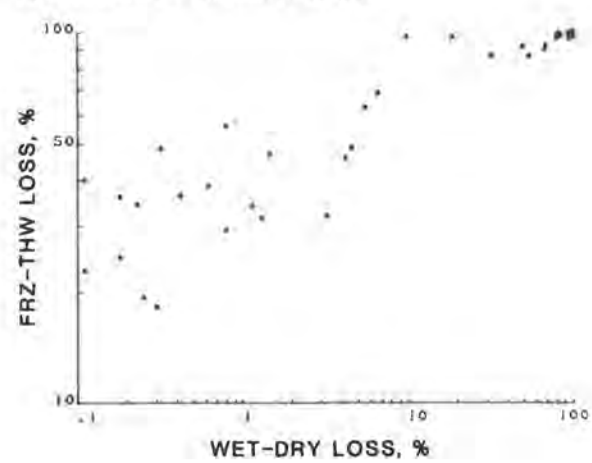


Figure 2. Freeze-thaw loss versus wet-dry loss.



results of the first are given in Table 6, the second in Table 7.

DISCUSSION OF RESULTS

The results of statistical analysis show that the Franklin slake test and the wet-dry test give very similar results. Correlation between them is very high. Multivariate stepwise regression indicates that, for instance, results of the three-cycle Franklin slake test can be determined from one- and three-cycle wet-dry test results by the following relation:

$$\text{Franklin SLAKE1} = 1.553 + 0.486(\text{WETDRY1}) + 0.556(\text{WETDRY3}) \qquad (1)$$

Figure 3. Slake loss versus absorption.

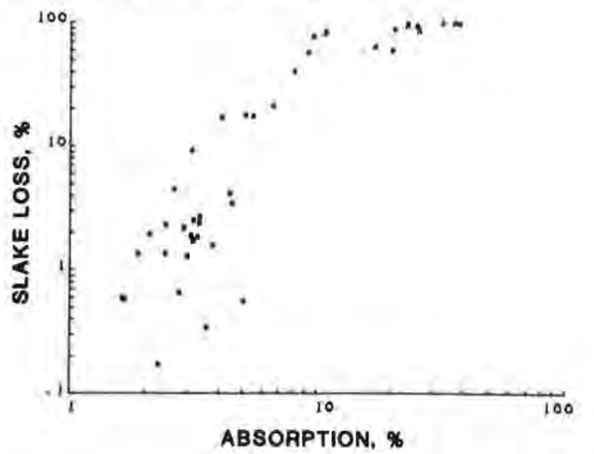


Figure 4. Freeze-thaw loss versus slake loss.

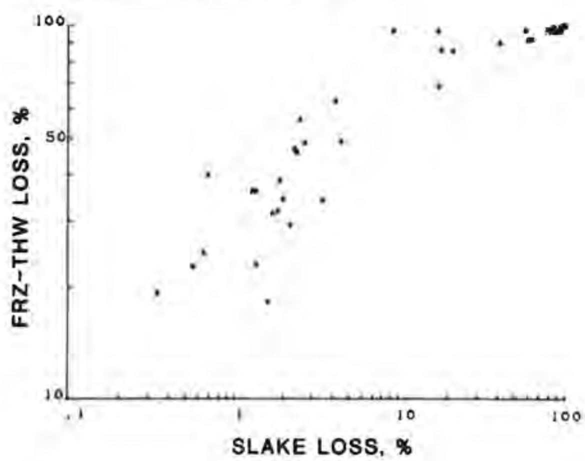


Figure 5. Adsorption at 95 percent relative humidity versus absorption.

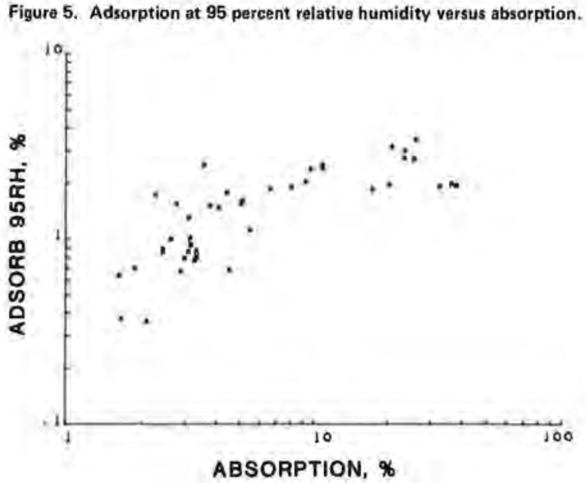


Figure 6. Adsorption at 95 percent relative humidity versus slake loss.

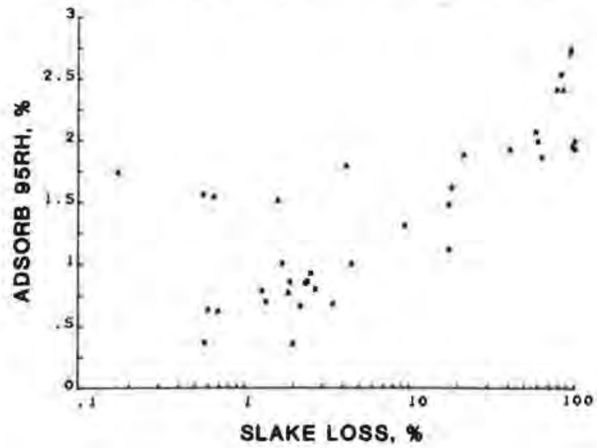


Figure 7. Freeze-thaw loss versus absorption.

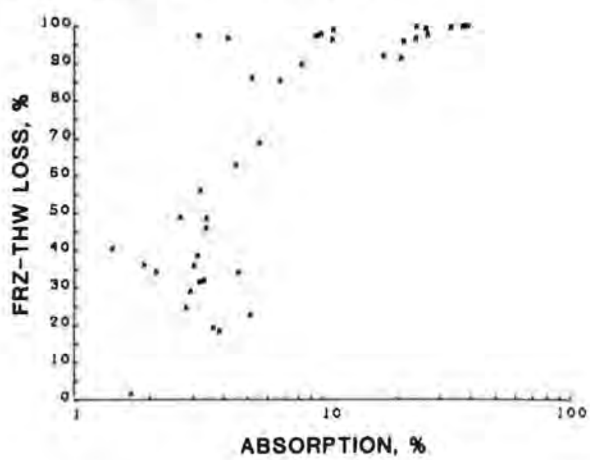
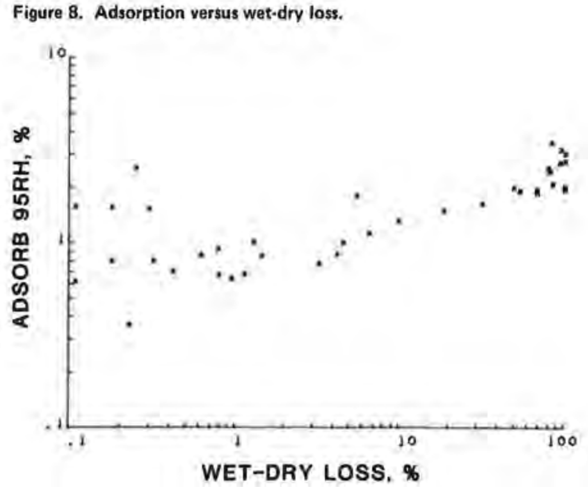


Figure 8. Adsorption versus wet-dry loss.



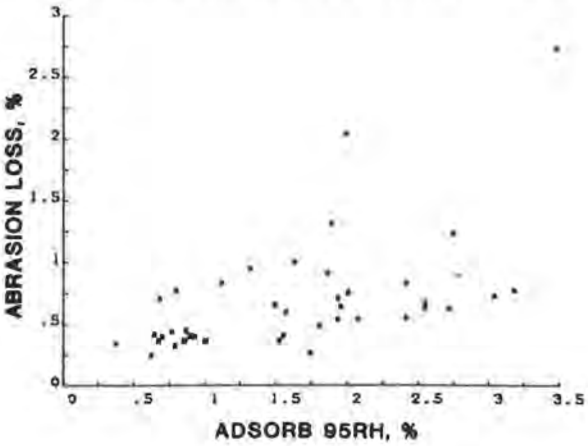
Likewise, three-cycle wet-dry results can be obtained from the Franklin slate test by

$$WETDRY3 = 0.239 - 0.544(SLAKE1) + 1.375(SLAKE3) \quad (2)$$

Adsorption at 45 percent relative humidity enters both relations as a third significant variable, improving them slightly. The improvement is not significant but simply points to the importance of adsorbed water as a factor in the deterioration of shales.

Resistance to freezing and thawing is related to the results of many tests (as in the case of the 12-cycle freeze-thaw test in Table 6). The partial F's indicate that results of the WETDRY3 test have the strongest influence on freezing and thawing results.

Figure 9. Abrasion loss versus adsorption at 95 percent relative humidity.



Abrasion and water absorption are also important but less significant. SLAKE3 results show a marginal effect on the relation.

When the strongly related tests are excluded from the analysis (Table 7), it is possible to determine the relations of other tests to Franklin slake and wet-dry tests. As can be expected, the above two tests are influenced by much the same tests and in much the same order of importance. What is different, however, is the degree of influence exhibited by the independent variables. The Franklin SLAKE3 test, for instance, is strongly related to water absorption of shale and less so to FRZTHW12 and ABRASION results. The WETDRY3 test shows proportionately less dependence on water absorption and more on FRZTHW12 and ABRASION properties.

CONCLUSIONS

The results of statistical analysis have indicated that some of the tests used to measure shale durability are strongly interrelated. The following can be concluded:

1. Wetting and drying degradation can be equally well tested by simple alternate wetting or drying or by the more sophisticated Franklin slake test.
2. Wetting and drying deterioration is influenced by the ability of the shale to absorb water and, to a lesser degree, by its freeze-thaw and abrasion resistance.
3. Freezing and thawing resistance is strongly related to wetting and drying resistance and, to a lesser degree, to abrasion resistance and absorption.
4. The relations are valid for the shale group studied. A wider sampling and a larger sample base are required to derive similar relations for shales as a group.

Table 3. Linear regression equations for Figures 1-9.

Figure	Dependent Variable (Y)		Slope (M)		Independent Variable (X)		Intercept (on Y)	R
1	Slake loss	=	0.91	x	Wet-dry loss	-	0.40	0.967
2	Freeze-thaw loss	=	13.16	x	ln (wet-dry loss)	+	34.20	0.915
3	Slake loss	=	34.7	x	Absorption	+	34.2	0.914
4	Freeze-thaw loss	=	17.6	x	ln (slake loss)	+	20.7	0.850
5	Adsorb 95RH[a]	=	0.7	x	Absorption	+	0.33	0.850
6	Adsorb 95RH[a]	=	0.016	x	Wet-dry loss	+	0.97	0.809
7	Freeze-thaw loss	=	28.2	x	Absorption	+	11.58	0.803
8	Adsorb 95RH[a]	=	0.31	x	Slake loss	+	0.84	0.706
9	Abrasion	=	0.32	x	Adsorb 95RH[a]	+	0.17	0.560

[a]Adsorption at 95 percent relative humidity.

Table 4. Durability classification: five-cycle slaking loss, +13-mm sample size.

Classification	No. of Samples	Loss Limit (%)	Nature of Loss
Rocklike	20	0-5	No discernible effect
Low loss	6	5-25	Minor spalling along bedding planes
Intermediate loss	3	25-60	Spalling and disintegration
High loss	14	60-100	Disintegration into mudlike consistency

Table 5. Durability limits of shales.

Test	Rocklike	Low Loss	Intermediate Loss	High Loss
Slaking loss (%)	0-5	5-25	25-60	60-100
Wet-dry deterioration (%)	0-6	6-28	28-66	66-100
Rate of slaking, H_2O absorbed (%)	0-2.6	2.6-4.7	4.7-12.9	12.9-40[a]
Freeze-thaw loss, 12 cycles (%)	0-49	49-70	70-88	88-100
H_2O adsorption at 95 percent relative humidity, 30°C (5)	0-1.0	1.0-1.4	1.4-2.1	2.1
Abrasion loss	0-0.47	0.47-0.72	0.72-1.0	1.0

[a]Extrapolated upper limit.

Table 6. Stepwise regression of all test data.

Variable	B-Value	Significance Level	F	R^2
Three-Cycle Slaking Conditions				
One-variable model			507.5	0.935
Intercept	0.594			
WETDRY3	0.876	0.01	507.5	
Two-variable model			490.5	0.967
Intercept	1.553			
WETDRY1	0.486	0.01	31.5	
WETDRY3	0.556	0.01	76.1	
Three-variable model			334.3	0.968
Intercept	3.558			
WETDRY1	0.469	0.01	29.2	
WETDRY3	0.593	0.01	73.6	
ADS45	−4.395	20.15	1.7	
Three-Cycle Wet-Dry Conditions[a]				
One-variable model			507.5	0.935
Intercept	0.865			
SLAKE3	1.068	0.01	507.5	
Two-variable model			317.0	0.949
Intercept	0.239			
SLAKE1	−0.544	0.48	9.1	
SLAKE3	1.375	0.01	155.5	
Three-variable model			240.5	0.956
Intercept	−4.328			
SLAKE1	−0.477	0.91	7.6	
SLAKE3	1.283	0.01	133.6	
ADS45	9.635	2.63	5.4	
Four-variable model			194.3	0.960
Intercept	−2.434			
SLAKE1	−0.549	0.30	10.3	
SLAKE3	1.278	0.01	141.7	
ADS45	18.741	0.60	8.7	
ADSGLYC	−13.670	7.54	3.4	
Twelve-Cycle Freeze-Thaw Conditions[a]				
One-variable model			68.3	0.661
Intercept	37.397			
WETDRY3	0.760	0.01	68.3	
Two-variable model			37.1	0.685
Intercept	31.586			
WETDRY3	0.633	0.01	37.9	
ABRASION	12.166	11.21	2.7	
Three-variable model			28.8	0.724
Intercept	31.118			
WETDRY3	0.961	0.01	30.8	
ABRASION	27.637	1.02	7.4	
SLAKEADS	−2.616	4.03	4.6	
Four-variable model			21.9	0.732
Intercept	31.990			
WETDRY3	0.658	0.01	23.5	
ABRASION	30.795	0.67	8.4	
SLAKEADS	−3.629	2.94	5.2	
SLAKE3	0.504	32.60	1.0	

Note: Addition of more variables does not improve the model significantly.
[a]All variables allowed.

Table 7. Stepwise regression of selected data.

Variable	B-Value	Significance Level	F	R^2
Three-Cycle Wet-Dry Conditions[a]				
One-variable model			91.2	0.723
Intercept	−5.428			
SLAKEADS	4.487	0.01	91.2	
Two-variable model			80.9	0.826
Intercept	−21.200			
SLAKEADS	2.919	0.01	32.3	
FRZTHW12	0.468	0.01	20.3	
Three-variable model			76.2	0.874
Intercept	−15.467			
SLAKEADS	4.262	0.01	53.1	
FRZTHW12	0.502	0.01	30.8	
ABRASION	−24.403	0.13	12.5	
Four-variable model			67.3	0.894
Intercept	−1.698			
SLAKEADS	3.959	0.01	50.1	
FRZTHW12	0.834	0.01	27.2	
ABRASION	−24.400	0.06	14.3	
FRZTHW25	−0.400	2.04	5.9	
Five-variable model			62.4	0.910
Intercept	−6.240			
SLAKEADS	3.943	0.01	56.7	
FRZTHW12	0.743	0.01	23.1	
ABRASION	−25.085	0.02	17.2	
FRZTHW25	−0.382	1.83	6.2	
ADS45	14.715	2.59	5.5	
Three-Cycle Slaking Conditions[b]				
One-variable model			163.3	0.823
Intercept	−6.763			
SLAKEADS	4.338	0.01	163.2	
Two-variable model			120.8	0.877
Intercept	−17.000			
SLAKEADS	3.320	0.01	71.8	
FRZTHW12	0.303	0.05	14.7	
Three-variable model			127.4	0.921
Intercept	−12.017			
SLAKEADS	4.488	0.01	113.9	
FRZTHW12	0.333	0.01	26.2	
ABRASION	−21.209	0.02	18.2	
Four-variable model			112.6	0.934
Intercept	−1.860			
SLAKEADS	4.264	0.01	113.6	
FRZTHW12	0.578	0.01	25.5	
ABRASION	−21.199	0.01	21.1	
FRZTHW25	−0.295	1.71	6.3	

Note: Addition of more variables does improve the model significantly.
[a]WETDRY1, SLAKE1, SLAKE3 excluded.
[b]SLAKE1, WETDRY1, WETDRY3 excluded.

REFERENCES

1. R. Chandra. Slake Durability Test for Rocks. Imperial College of Science and Technology, Univ. of London, Master's thesis, Rock Mechanics Res. Rept., 1970.
2. J.A. Franklin and R. Chandra. The Slake Durability Test. International Journal of Rock Mechanics and Mining Sciences, Vol. 9, 1972.
3. N.R. Morgenstern and K.D. Eigenbrod. Classification of Argillaceous Soils and Rocks. Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No. GT10, Oct. 1974.

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Relevance of Durability Testing of Shales to Field Behavior

JAMES L. WITHIAM AND DAVID E. ANDREWS

Geologic materials exhibit a wide range of behavioral responses following excavation and replacement in a new environment. This is a result of various mechanisms induced by variations in moisture and stress regimes or other environmental aspects of the materials. The physical disintegration of such geologic materials caused by fundamental changes in stress conditions or strength characteristics is referred to as slaking. The most distinctive aspect of the slaking process is a relatively rapid decrease in fragment size of the material. To develop an understanding of the slaking process, a comprehensive evaluation of existing data was undertaken that included a detailed literature review of geotechnical, agronomical, and geochemical test procedures used to identify potentially problematic slakable materials. The findings from this review are incorporated into a comprehensive laboratory testing program that forms the basis for a proposed classification system and spoil management program. It is concluded that a relatively simple series of tests can be used to assess the probable impact of slaking of fine-grained materials on stability, settlement, and erosion potential. Comparisons are made between laboratory and field observations to support this conclusion.

To develop an understanding of the slaking process as it relates to mine spoils, a comprehensive evaluation of existing laboratory techniques was undertaken. The principal objectives of previous laboratory durability testing programs have centered around (a) assessing the applicability of accelerated rock weathering tests to help predict long-term performance, (b) quantifying the effects of time-dependent rock degradation caused by various slaking mechanisms, and (c) finding additional tests that can quickly identify problem materials.

A summary of previous research efforts and general survey studies associated with durability testing programs is presented in Figure 1 [full citations for the studies included are presented in the report by Andrews and others (1)]. Tests are classified according to their nature and applicability as identification (physicochemical and mineralogical), durability, and strength tests. Many of these consist of standard and modified geotechnical, agronomical, and geochemical tests. Most programs include both identification tests, such as grain size and mineralogy, and behavioral tests, such as rate of slaking. Ideally, the simplest and most reliable indicators are sought for a given purpose, although more sophisticated tests often serve a useful purpose.

Physicochemical tests have been incorporated into several durability-related testing programs because much useful data can be collected quickly, at low cost, by using relatively standard equipment and test procedures (2). As Figure 1 indicates, those tests that show the greatest success in correlations with observed durability behavior are water content and, to lesser degrees, grain size and Atterberg limits. The successful performance of these tests is principally related to the fact that most low-durability geologic materials are fine-grained and often active clay mineralogies that absorb water following stress relief. Because durability is fundamentally related to mineralogical composition, applications of mineralogical tests also have formed the basis for several slake durability studies. Of those techniques identified in Figure 1, X-ray diffraction has been applied most successfully, although its high cost makes other less quantitative procedures, such as the methlyene blue absorption (MBA) test, attractive substitutes.

Figure 1. Summary of laboratory tests associated with slaking.

	IDENTIFICATION TESTS										DURABILITY TESTS										STRENGTH TESTS			
	PHYSICO-CHEMICAL							MINERAL-OGICAL																
STUDIES(3)	WATER CONTENT	GRAIN SIZE	ATTERBERG LIMITS	UNIT WEIGHT	SPECIFIC GRAVITY	pH	CHEMICAL ANALYSIS	PETROGRAPHIC EXAMINATION	X-RAY DIFFRACTION	METHYLENE BLUE ABSORPTION	JAR SLAKE(1)	CYCLIC WET-DRY(2)	RATE OF ABSORPTION	RATE OF SLAKING	SLAKE DURABILITY	WASHINGTON DEGRADATION	ULTRASONIC DEGRADATION	L. A. ABRASION	FREEZE-THAW	SWELLING TESTS	UNCONFINED AND TRIAXIAL COMPRESSION	POINT LOAD	SCHMIDT HAMMER	SHORE HARDNESS
Gamble	●	○	○	●					○		○	○			●					○				
Haley and MacIver	○	○	○	○					○		○	○								○				
Deo		○	○	○					○			●	○		●			○						
Eigenbrod	●	○	●						○			○		●							○			
Laguros		●	○		○	○		○	●		●						○			●				
Aufmuth															●							○	●	
Goodman, Heuze, Thorpe, and Chatoian	○			○											●							○		
Nettleton			○							●					●									
Reidenouer, Geiger, and Howe		○	○	○				○	●		●	○	○			●	●	●	○					
Chapman	●		●						○		●	●		●	●	○	○	○					○	
Augenbaugh and Bruzewski	●		○										●		○					○	○			○
Bailey	○	●									○											○		○
Rodrigues											○				○									
Lutton	○					○			○		●				●									○
Noble							○		●		●	●			○									
Strohm	○	○	○									○			○							○		
Strohm, Bragg, and Ziegler			○			○					○	○		○	●							○		
Hudec	●			○								●	●	○	●					○				

(1)Water, ethylene glycol, hydrogen peroxide, or sulfuric acid used as slaking fluids.
(2)Water and sodium sulfate solutions used for slaking wet-dry cycles.
(3)Study citations are omitted for brevity.

LEGEND:
○ - Test conducted.
● - Fair to good correlation with durability.

Several types of tests have been developed or modified to provide qualitative and/or quantitative assessments of the slake potential of geologic materials. As illustrated in Figure 1, several of these have shown moderate to very good success. Others, because of severe test conditions, show little apparent value. In general, those durability tests that show the better correlation with field response also represent the more realistic models of slaking processes. By most accounts, slaking of geologic media results from physical breakdown by wetting and drying, chemical action by leaching, and other processes following stress relief. Based on these criteria, the first five durability tests identified in Figure 1 represent the most realistic models of field behavior. They have also shown the greatest success during previous investigations.

The jar slake test is the simplest of all durability tests and consists of placing a rock fragment or gradation of rock fragments into a beaker containing water or some chemical solution. After immersion, sample breakdown is observed for some period of time ranging from hours to days. Cyclic wet-dry durability tests subject rock fragments to conditions more consistent with observed field response. These tests impose several cycles of alternating wetting and drying by soaking samples in water or chemical solutions (e.g., sodium sulfate) and air or oven drying over a range of temperatures. These test conditions are consistent with the cyclic wet-dry, rate of absorption, and rate of slaking durability tests. Finally, the slake durability test (3) uses the additional feature of mechanical agitation to accelerate slaking under laboratory conditions. To date, the slake durability test is the only procedure acknowledged as part of an accepted industry standard (3). Most of the other reported durability test procedures represent unusually severe conditions that frequently result in rapid and complete sample deterioration. Such complete sample breakdown provides neither a reasonable physical model nor a reliable model of field behavior. This is especially true for geologic media of an intermediate durability range.

The principal purpose in relating strength to durability has been for various classification schemes, although strength tests have also been used to directly identify rock durability or to infer future settlement or stability behavior. Several attempts have been made to relate the strength tests identified in Figure 1 to durability. These efforts have generally failed because of a lack of sensitivity in distinguishing ranges of slaking response, bias imposed by a need to test specimens prepared to certain tolerances, and a need to perform large numbers of tests to provide a statistical basis for test conclusions.

FIELD PROGRAM

To examine the nature and impact of slaking at coal surface mine sites in the eastern and central United States, detailed field and laboratory programs were designed and implemented. The field program was developed to document the nature, occurrence, distribution, and effects of slaking at sites in this region. A detailed account of the field program followed at four sites is provided in the paper by Perry and Andrews elsewhere in this Record.

Of particular importance to the laboratory program were highwall and mine spoil exploration programs, for these would yield undisturbed highwall and bulk spoil samples for laboratory evaluation. Highwall exploration was accomplished by drilling near the active highwall and using standard wireline techniques that provided NQ-sized core (1.875 in in diameter). The rock core was then placed in specially designed boxes that used weather stripping and heavy sheet plastic to minimize change in moisture content. In addition, bag samples of each major lithotype were collected from the fresh spoil area at every site to supplement the rock core samples obtained for the laboratory program.

To evaluate the mode, degree, and extent of slaking in mine spoils and the associated environmental effects, a thorough test-pit exploration and visual reconnaissance program was conducted by examining spoil recently placed as well as 2-, 5-, and 10-year-old spoils. After the completion of test-pit logging, bulk samples weighing approximately 100 lb were taken from a selected subsurface layer for subsequent determination of bulk grain-size distribution.

LABORATORY PROGRAM

As discussed previously, several slake testing procedures are available and these provided the framework for the laboratory program implemented for this study. In addition, several supportive analytic procedures were used for classification or identification purposes. The laboratory program was designed to be efficient and provide information needed to evaluate various parameters that affect the degradation of geologic materials. Incorporated into the design were those procedures and techniques that would allow anticipated key relations to be addressed.

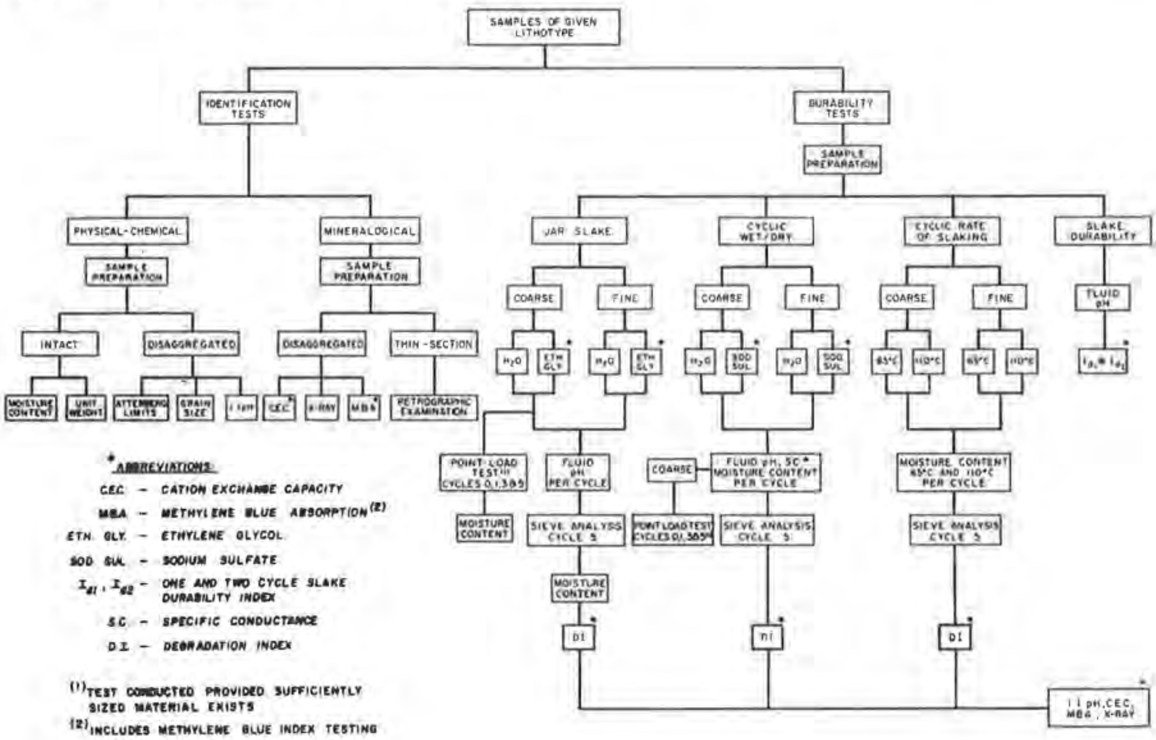
The selected laboratory tests were grouped in three categories: durability, identification, and ancillary testing. These categories were found useful for quantifying, classifying, and predicting the long-term durability characteristics of geologic materials. A flow diagram of the laboratory testing program is shown in Figure 2. Because space does not permit a detailed description of the test procedures used, the interested reader is referred to the completed report (1).

The lithotypes that represented major portions of the overburden or exhibited unusual field behavior were used for the testing program. Whenever possible, geologic materials obtained from test borings were selected because they were collected and preserved as closely as possible to the in situ conditions. Because siltstone, mudstone, and shale generally make up a high percentage of overburden and typically exhibit more severe slaking characteristics, the majority of samples for this study were fine-grained. Limestones were only tested with the slake durability apparatus because limestone seldom exhibits large amounts of physical deterioration. When an insufficient quantity of rock core was available, given lithotypes were supplemented with recently spoiled material. Occasionally, due to fragment size or material availability, insufficient quantities of a given lithotype were collected for the completion of the full series of tests.

The major portion of each sampled lithotype was used for the durability testing program. To determine the effects of fragment size on slaking, both coarse and fine components were selected for the jar slake, cyclic wet-dry, and cyclic rate of slaking durability tests. The coarse component consisted of 1.5- to 2-in fragments; the fine component ranged in size from 0.25 to 0.75 in in diameter. Samples prepared for the physicochemical testing were generally crushed at the natural water content to the required grain size (approximately No. 60 mesh size). Samples prepared for mineralogical analysis (X-ray diffraction) were dried and crushed to less than a No. 200 mesh size.

Durability tests used to measure the slaking po-

Figure 2. Generalized flow diagram for laboratory testing.



tential of the various highwall lithologies were the slake durability, jar slake, cyclic wet-dry, and rate of slaking tests. These tests were selected because they represent a wide range of conditions (e.g., sample fragment size, wetting and drying cycles, temperature, and slaking fluids) and had some degree of proven success.

Nine identification tests were used in the laboratory program. Moisture content, disaggregated grain size, Atterberg limits, and unit weight tests were performed in general accordance with standard testing procedures. Petrographic examination of thin sections of rock fragments was performed on coarse-grained sedimentary materials. The principal purpose for using this tool was to examine in detail the composition of matrix cements and rock fabric as they relate to the observed durability behavior. The remaining identification tests [1:1 pH, cation exchange capacity (CEC), MBA, methlyene blue index (MBI), and X-ray diffraction] were included to identify the chemical or mineralogical composition of the sampled lithotypes. X-ray diffraction and CEC were intended to provide an understanding of the mineralogical composition of the samples that could be used as a reference for comparing the results of other less costly and time-consuming test procedures (e.g., CEC versus MBA or MBI).

Additional tests that were expected to give insight into the durability characteristics of the sampled lithotypes included point load tests, analyses of the slaking fluid (pH and specific conductance), and fragment moisture contents. Point load strength tests were used to monitor the degree of strength reduction during the "slaking" tests and were performed as part of the coarse-fragment portion of jar slaking and cyclic wet-dry tests. Fragments were periodically selected for testing to determine incremental strength deterioration. Fluid pH, specific conductance, and fragment moisture contents were determined at key stages in the program.

The durability tests selected for study consisted of the jar slake, cyclic wet-dry, cyclic rate of slaking, and slake durability tests. These tests were selected because the influence of a wide range of parametric effects could be closely examined by using procedures that had previously proved successful. The parametric factors that could be considered from this series of tests include (a) lithotype; (b) sample size; (c) slaking fluid (i.e., distilled water and solutions of ethylene glycol and sodium sulfate); (d) temperature (i.e., 65° and 110° C for cyclic rate of slaking test); and (e) energy input. Because each of these may control, to some degree, the slaking response of geologic materials, the testing program represents a comprehensive attempt to identify the key factors that affect this behavior.

To quantify the deterioration of samples subjected to the jar slake, cyclic wet-dry, and cyclic rate of slake durability tests, a degradation index (DI) was used in accordance with a procedure suggested by Bailey (4). The DI is defined by using mean equivalent mesh sizes from sieving operations before and after slake testing as weighing factors to produce an index that is a measure of the amount of sample breakdown. The limits for this index range from zero (no breakdown) to 100 percent (complete breakdown). The slake durability index (I_d) was used to define the slake durability test results (3). Numerical subscripts to I_d (e.g., I_{d2}) are presented to indicate the number of testing cycles used to determine the index value. For purposes of comparison with the other durability test results, however, the term $(1 - I_{d2})$ is used to define a DI for the slake durability tests.

LABORATORY TEST RESULTS

Durability Tests

As shown in Figure 3, 128 individual durability tests were conducted by using 14 lithologic units

Figure 3. Summary of durability test results.

SITE	LITHOTYPE	DURABILITY TEST(1)																	
		JAR SLAKE(2)				CYCLIC WET/DRY(2)				CYCLIC RATE OF SLAKE(2)				SLAKE DURABILITY	
		COARSE		FINE		COARSE		FINE		COARSE		FINE			
		WATER	ETHYLENE GLYCOL	WATER	ETHYLENE GLYCOL	WATER	SODIUM SULFATE	WATER	SODIUM SULFATE	65°C	110°C	65°C	110°C	I_{d_2}(3)	$1-I_{d_2}$
A	Brown Sandstone	0	2.7	0.5	0.5	2.0	4.9	0.8	0.3	0	0	0.8	0.3	99.0	1.0
	Gray Shale	11.7	14.8	0.4	0.3	31.6	34.2	0.7	0.7	40.9	52.5	1.2	1.8	87.6	12.4
	Green Mudstone	●	●	47.5	51.9	76.6	■	56.6	30.7	●	●	48.7	58.6	87.1	12.9
	Black Carbonaceous Shale	21.8	26.2	0.5	0.2	42.1	52.1	2.1	0.8	■	35.2	1.9	1.2	97.6	2.4
	Gray Limestone	●	●	●	●	●	●	●	●	●	●	●	●	99.3	0.7
B	Gray Siltstone	3.7	0	1.6	0	75.2	72.2	26.5	16.4	0	76.2	12.1	24.3	98.5	1.5
C	Yellow Sandstone	0	0.3	●	●	0	0	●	●	0	0	●	●	97.1	2.9
	White Sandstone	0	0	0.3	0.3	0	0	0.6	0.7	0	0	1.2	0.5	97.7	2.3
	Gray Siltstone	31.4	25.4	●	●	24.4	25.7	7.5	3.0	26.8	31.6	●	●	94.3	5.7
D	Brown Sandstone	5.4	3.4	●	●	0	18.1	●	●	10.4	7.5	●	●	99.5	0.5
	Gray Shale	38.3	32.6	13.5	16.9	41.9	64.4	42.0	32.6	80.5	87.5	45.7	57.6	98.3	1.7
	Green Mudstone	13.3	9.7	19.3	10.9	20.6	21.8	27.4	15.3	2.3	24.2	21.2	22.6	75.7	24.3
	Red/Green Mudstone	85.6	●	●	●	92.7	●	●	●	●	●	●	●	63.4	36.6
	Gray Limestone	●	●	●	●	●	●	●	●	●	●	●	●	99.7	0.3

(1) All numbers expressed as percent.
(2) Numbers in columns represent Degradation Index (DI) values.
(3) 2-cycle slake durability index.

LEGEND
● - No test performed.
■ - Test performed; sample crushed for X-ray prior to sieving.

collected from four different active surface mining sites. These lithologic units included four sandstones, two limestones, two siltstones, three mudstones, and three shales.

The size of sample fragments is important for almost all cases studied. In general, the coarse samples developed greater breakdown for all fluids and drying temperatures. This was most noticeable for thinly bedded, anisotropic sediments, such as siltstones and shales. Consequently, for complete slake potential identification, samples of sufficient size must be tested. The results of these tests suggest that a minimum fragment size of 1 in be used.

For the samples tested, the application of solutions of sodium sulfate and ethylene glycol led to slight differences in breakdown compared with the use of distilled water. Because the distinctions are small, the use of chemical slake fluids probably can be ignored.

With the exception of the slake durability test, cyclic wetting and drying provided the greatest assurance of breakdown. Of the cyclic tests, the cyclic rate of slaking procedure at 110° C is the most severe, which indicates that short soaking periods that cause only partial saturation are more effective for the tested samples. An advantage of this method is the shorter soaking observation period, which reduces the overall time required for testing. The range of DI values obtained for the jar slake test was narrower than for the cyclic tests, which makes the analysis more difficult and subjective. Therefore, cyclic testing is preferred to the static jar slake test.

The results of the cyclic rate of slaking test provided an opportunity to study the relation between drying temperatures and slake durability behavior. These tests indicate that a reduction in durability occurs with increasing drying temperatures. This pattern is stronger for the coarse size fraction, which also demonstrates the effect of particle size on the durability of the tested samples.

With the possible exception of the slake durability tests, greater degrees of breakdown were realized with test procedures that involved the greatest energy effort. This effort was developed by several means, including fluid infiltration, oven drying, chemical attack, and mechanical agitation. Of the durability tests considered for study, both the cyclic wet-dry and cyclic rate of slaking tests at 110° C provided the broadest range of breakdown (DI values ranging from 0 to 93 percent) and sufficient sensitivity to characterize relations within these boundaries.

Identification Tests

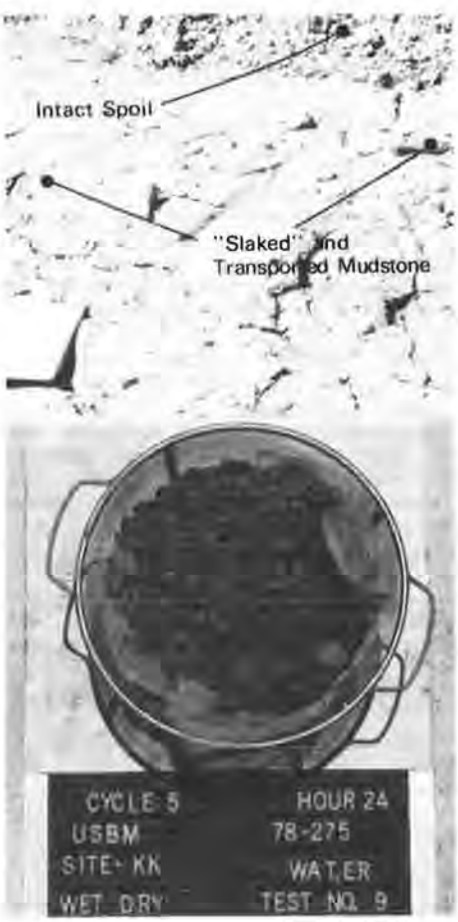
Knowledge of the physical, chemical, and mineralogical composition of geologic materials can provide corroborative information regarding their slake durability behavior. The major goal of these tests, therefore, was to identify procedures that can be confidently used as part of an overall slake durability predictive model.

Except for moisture content determinations taken at the end of test cycles, none of the geotechnical index tests provided any reliable basis for predicting durability behavior. Grain-size analyses and Atterberg limits tests of disaggregated samples revealed a small range of values that bore little relation to the observed durability behavior. These results were generally confirmed through X-ray diffraction analysis, which indicated relatively inactive clay mineralogies. A similar small range of initial moisture content values was measured for undisturbed highwall samples collected from all sites. The range of in situ moisture contents was generally unrelated to durability behavior, and no correlations could be made between unit weight and durability.

Variations in moisture content with increasing cycles of durability testing generally follow two trends. For sandstones and more durable siltstones and shales, the moisture content increased to an equilibrium value within one or two cycles. All other fine-grained sediments exhibited an increasing moisture content with increasing numbers of cycles. Accordingly, these patterns can be correlated with

Figure 4. Comparison of field and laboratory behavior of gray siltstone from site B.

Figure 5. Comparison of field and laboratory behavior of mudstones from site D.



durability. In a similar fashion, moisture contents for samples consisting of the fine fragment size always exceeded the coarse-fraction moisture content for corresponding samples and solutions. The smallest increases were realized for sandstones and the largest for mudstones. The principal factor controlling the moisture-content behavior of coarse and fine fragments is the relation between surface area and volume.

Of all other identification tests, only 1:1 (soil) pH and CEC showed any relation to durability test results. Application of this observation is described subsequently.

Ancillary Tests

Additional testing was performed concurrently with the durability testing program. This included point load tests and analyses of pH and specific conductance of the slake fluid. These various techniques were relatively simple to perform and were thought to possibly provide additional data for understanding the slaking phenomenon.

Although much effort was spent to test the suitability of the point load test as a predictor of durability, only limited trends could be discerned. The point load strength index (I_s) generally increased with increasing durability, but the results are biased toward the more durable rock fragments that remained at the end of various test cycles. For lithotypes with very low durability (e.g., mudstone), the lack of samples of sufficient size precluded the possibility of testing. Measurements of slake fluid pH and specific conductance provided no recognizable trends.

COMPARISON OF FIELD AND LABORATORY PROGRAMS

Some previous studies of slaking have failed to correlate laboratory test results with observed field behavior. This has often led to incomplete recognition of several of the prime factors that control the breakdown of geologic materials. Furthermore, because of the similitude and time-effect problems associated with laboratory programs, the implications of several factors (e.g., sample size and accelerated weathering) in relation to field behavior are not completely understood. Accordingly, field observations of durability and modes of slaking were generally substantiated by, and in agreement with, the laboratory program. Two examples at different sites are summarized to show the similarity of field and laboratory response.

At site B, the gray siltstone is initially massive with high rock quality designation (RQD) values and no apparent bedding. When subjected to slaking stresses, however, this siltstone undergoes chip slaking, which was seen in both the field and the laboratory (see Figure 4). This behavior is strongly controlled by structure (thin bedding planes) that originates during deposition of the sediments. The coarse fractions subjected to laboratory testing showed high DI values, whereas the fine fractions were more durable and had DI values equal to approximately one-third those of the coarse samples. This is generally consistent with the observed field behavior in that the fine chips appeared to resist further degradation.

At site D, although the lithology of the red-green mudstone and green mudstone is similar, the rate of deterioration was far more rapid for the red-green mudstone, as evidenced by the general lack of this material in even recent spoil piles. This was supported by the very rapid disintegration that developed in the laboratory following immersion. However, the impact of this lithotype within the spoils is negligible because it represents less than 5 percent of the highwall materials. The RQDs for the red-green mudstone were extremely high. This suggests that RQD and similar classification systems do not accurately reflect slaking potential. Similar results were obtained for other lithotypes throughout the study.

The observed pattern for both mudstones at site D, however, is similar because neither appears to be affected by fragment size or structural control. Even though the green mudstone initially undergoes chip slaking in the field, it slakes with time to its constituent particle size. Figure 5 shows examples of the pattern of slaking for these mudstones in the field and the laboratory.

PROPOSED CLASSIFICATION SYSTEM

The proposed classification system is based on a series of field and laboratory decision filters. Each portion of the system provides the necessary ingredients to identify potentially problematic geologic materials. In addition, the final mixture of slakable and nonslakable units in spoil piles is considered, based on information obtained during the field program. It should be noted, however, that because this system is based on limited information obtained from only seven sites (four of which were studied in detail), the proposed scheme is tentative and additional input is warranted.

The proposed system is presented in Figure 6 and includes those techniques that were found to reliably predict the slaking potential and behavior of geologic materials. It is divided into field and laboratory phases and uses relatively simple inspection and routine test procedures that are often a part of surface mine permitting programs. The classification system is divided into four categories: preliminary field reconnaissance, exploration program, preliminary laboratory program, and durability testing program.

The preliminary field reconnaissance takes advantage of the useful information that can be gathered by examining bedrock outcrops in the vicinity of the proposed site. Of particular importance are the type and quantity of fine-grained sediments because these factors bear a direct relation to the nature of problems that may be encountered following spoiling operations.

Drilling programs represent the second tier of the proposed classification system. At this level, the behavior of fresh rock samples can be compared with the behavior in bedrock exposures. In addition, simple testing with dilute HCl can be used to ascertain the need for proceeding with a laboratory testing program. The specific decisions in this phase of the program are illustrated in Figure 6.

The laboratory testing program is designed as a two-phase filtering system. The principal purpose is to separate durable and nondurable geologic materials based on two tests by using crushed, powdered rock samples (1:1 pH and CEC). A relation exists between CEC and 1:1 pH that can be extended to include rock durability based on a five-cycle DI. Figure 7 demonstrates that the DI increases with increasing 1:1 pH and CEC, which is related to the influence of carbonates and various clay mineral phases. To incorporate this behavior into the proposed classification scheme, a DI value of 50 percent (representing the delineation between moderate and high degradation levels) was selected as a threshold. This value, in conjunction with the chemical performance of the suite of materials tested, suggested that maximum limits of 7.8 (1:1 pH) and 15 meq/100 g (CEC) be used, as indicated in Figure 7, as criteria to assess the need for durability testing. If, for a particular sample, the preliminary test results fall below these boundaries (crosshatched zone in Figure 7), no further testing is necessary nor are special design measures beyond normal practical requirements expected. If either of these bounds is exceeded, then further testing is required.

The final stage in the classification system uses

Figure 6. Proposed classification system.

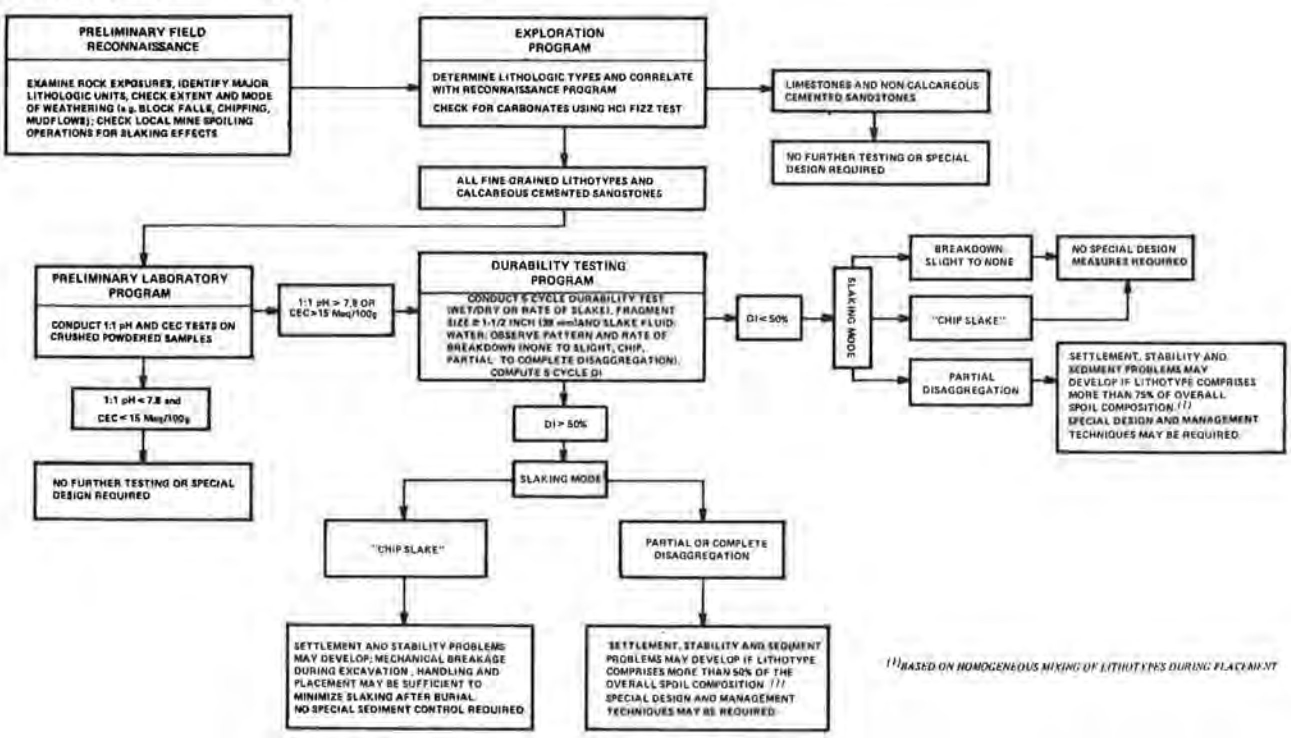
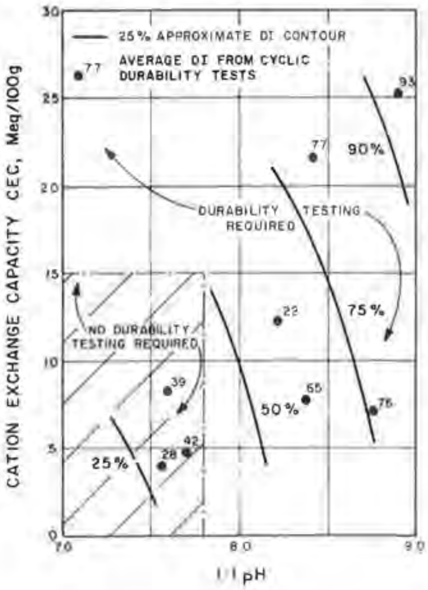


Figure 7. Proposed laboratory classification filter.



durability testing. This may be readily and simply accomplished by subjecting the remaining samples to either of the cyclic durability test procedures described previously. Based on results from the laboratory program, it is recommended that these tests be conducted by using coarse fragments 1.5 in or larger and water as the slaking medium. Intensity and mode of breakdown then can be identified and compared with the results of the field reconnaissance, and appropriate design considerations can be implemented to minimize potential environmental impacts of spoiling these materials.

The assessment of spoil management control measures for the above categories was developed from the field program. At each site, the reclamation procedures that are practiced follow relatively routine patterns that seem to provide sufficient amounts of breakdown and densification to minimize slaking effects. These result in relatively minor problems (mostly surficial) despite the fact that some lithotypes developed DI values that were higher than expected (e.g., gray siltstone at site B) or were present in sampled spoils in relatively high amounts (e.g., 60-70 percent mudstone at site D). This suggests that the effort expended during excavation, handling, and placement and the intermixing of durable and nondurable sediments can control, to a large degree, the ultimate behavior of geologic materials. Therefore, present spoiling and reclamation practices may overcome the majority of problems that could be associated with low-durability materials.

SUMMARY

Based on evidence provided from a detailed field and laboratory testing program, laboratory durability testing appears to provide a rational basis for predicting field behavior. Several types of durability tests and various procedures were evaluated along with a variety of supplemental tests to provide index, mineralogical, and chemical data. Of these, cyclic wet-dry durability testing using coarse site fragments soaked in water along with other simple chemical tests (i.e., soil pH and CEC) appears to offer a reasonable basis for a predictive model. Finally, a classification system is outlined that uses observed field spoil response as a basis for development. Although the proposed model is preliminary due to its limited data base, the classification system nonetheless represents a reasonable first-step approach to the assessment of mine spoil durability.

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REFERENCES

1. D.E. Andrews, J.L. Withiam, E.F. Perry, and H.L. Crouse. Environmental Effects of Slaking of Surface Mine Spoils: Eastern and Central United States. Bureau of Mines, U.S. Department of the Interior, Denver, CO, Final Rept., 1980, 247 pp.
2. 1979 Annual Book of ASTM Standards: Part 19--Natural Building Stones; Soil and Rock; Peats, Mosses and Humus. ASTM, Philadelphia, 1979.
3. E.T. Brown, ed. Rock Characterization, Testing, and Monitoring: ISRM Suggested Methods. Pergamon Press, Oxford, England, 1981, pp. 92-94.
4. M.J. Bailey. Degradation and Other Parameters Related to the Use of Shale in Compacted Embankments. Indiana State Highway Commission, Indianapolis, and Engineering Experiment Station, Purdue Univ., West Lafayette, IN, Joint Highway Research Project JHRP-76-23, 1976, 230 pp.

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Prediction of Soil Properties from Simple Indices

Y-K. T. LO AND C. W. LOVELL

The Indiana geotechnical data bank has been established to collect geotechnical information from the subsurface investigation reports of various geotechnical projects previously conducted in the State. The data bank is user oriented, simple to use, and flexible to accommodate further change as the requirements of users become more clearly defined. Statistical operations on the data, varying from generation of sample distribution parameters of soil characteristics to correlation efforts among variables, are done to aid in evaluating information from new sites or in approximating geotechnical information for preliminary investigations. Correlations between simple soil indices and soil characteristics and properties that are costly to measure directly are discussed. Significant relations were developed for compressibility parameters under sustained loading and for compaction parameters under repeated transient loadings. Little success was achieved in predicting strength parameters from simple indices. The study establishes relations between soil characteristics, which may be valuable in preliminary studies. The information does not replace fuller site investigation, sampling, and testing but does produce a framework against which various test results can be judged for their consistency and reliability.

The need for pedologic and engineering soils information for use in planning, site selection, design, construction, and maintenance of transportation facilities is recognized by civil engineers. Data are necessarily limited in quantity and quality due to economic and time constraints. Although large amounts of detailed soil data are often available from work performed on adjacent or nearby projects, these data are usually not readily accessible for use or their existence is unknown.

Extensive laboratory and field test data have been accumulated by the Indiana State Highway Commission for use in characterizing the engineering properties of Indiana soils. The information is retained in the form of subsurface investigation reports, prepared by or for consulting design firms and government agencies for use in routine soil investigations. In its present voluminous form, most of the information is not very useful. The need existed to make this information more accessible for the engineer interested in detailed information on a site and the engineer interested in soil characteristics over a larger region.

In July 1977, research was initiated to develop and test a computerized information storage and retrieval system for soils in Indiana. The earlier parts of this research effort, which involved developing codes and storage techniques, have been reported elsewhere (1,2).

Phase 2 of the research is the subject of this paper. It involved the storage of 6934 additional data sets, for a total of 9442 data sets as of December 1979. These data sets were from roadway soil boring reports and from those reports on boring for bridge and culvert sites that contained laboratory test data. Additional purposes of phase 2 were to (a) show how to manage the data bank and (b) evaluate the information stored and develop correlations and quantitative values for planning and preliminary design by using statistical methods.

Both conventional and nonparametric statistical methods were used. However, the nonparametric statistical methods appear to fit and explain the varieties of soil characteristics in a superior way. The data were grouped by using physiographic regions, engineering soil classifications, soil associations, or a combination of these. One-way and two-way classification and factorial experiment layouts were used to examine the distributions of the data. Regression analysis was used to investigate the functional relations between design parameters and index properties. The total results are presented in a report by Lo (3). Because the soils data were extremely variable in their characteristics, choosing suitable groupings for study was the most difficult task in the investigation.

PREVIOUS INVESTIGATIONS

Concept of a Geotechnical Data Bank

In 1965, the South Dakota Department of Highways, in cooperation with the U.S. Soil Conservation Service, began a program to collect the accumulated geotechnical data from previous projects (4). The data were then stored in a computerized system and analyzed by using statistical methods for the purposes of planning, location, and preliminary design.

Spradling (5) developed a computerized data storage and retrieval system for the State of Kentucky. The Kentucky Department of Transportation devised an extensive coding system for data that were collected but not suitable for direct computer storage. Due to the completeness of this coding system and its applicability to soil information in general, some of its details were adopted for the Indiana data bank (1).

Recently, the U.S. Department of Transportation (DOT) published a state-of-the-art report that documented basic information on automatic data processing techniques used by eight states (Colorado, Georgia, Illinois, Louisiana, Minnesota, New York, Pennsylvania, and West Virginia) in managing test data for highway materials (6). Three basic data processing techniques--batch information systems, on-line interactive information systems, and on-line interactive laboratory information systems--were in use.

The Prairie Farm Rehabilitation Administration (PFRA) of Canada collected data on the geotechnical properties of glacial till deposits, glacial lake deposits, and alluvial deposits from many of its previous projects in western Canada (7,8). A number of empirical relations between routine classification tests and consolidation and strength characteristics were developed, as discussed below. The correlations were generated primarily to aid in evaluating information from new sites or to approximate geotechnical information for preliminary investigations.

Similar geotechnical data storage and retrieval systems were developed in the following countries: Sweden, Finland, and Denmark (9); France (10); Rhodesia (11); Algeria (12); and South Africa (13).

Empirical Relations in Practice of Geotechnical Engineering

Compressibility Parameters

The laboratory oedometer test is normally used to determine compressibility parameters such as compression index (C_c), recompression index (C_r), and preconsolidation pressure (p_c). The compression index (C_c) has often been correlated with either percentage liquid limit (w_L), percentage natural moisture content (w_n), initial void ratio (e_o), or a combination of these (3). The compression ratio (C_r') is defined as $C_c/(1 + e_o)$. It, too, is correlated with w_L, w_n, e_o, or a combination of these. Rutledge (14) and Fadum (15) showed that, as the natural moisture content increases, the compression ratio increases linearly for normally consolidated clays. Lo (3) gives a summary of available regression equations, together with their geographic region of applicability, for the prediction of C_r'.

By definition, p_c is the greatest effective pressure the soil has carried in the past. Preconsolidation may be caused by a variety of factors (16). The Canadian PFRA (7,8) has described a relation between liquidity index (LI) and log p_c, where as LI increases log p_c decreases linearly for the soils in western Canada. Bjerrum (17) developed a relation between the preconsolidation pressure/overburden pressure ratio and plastic index for late-glacial and postglacial clays.

Compaction Parameters and California Bearing Ratio

Compaction is defined as the densification of a soil by means of mechanical manipulation at constant water content and is measured quantitatively in terms of dry density of the soil. Proctor (18) demonstrated that there is a definite relation between density and moisture content. The characteristic peak in the Proctor curve is known as the maximum dry density (ρ_{dmax}) and its corresponding moisture content as optimum moisture content (OMC).

As the Atterberg liquid limit, plastic limit, or plastic index increases, OMC increases and ρ_{dmax} decreases. Furthermore, as OMC increases, ρ_{dmax} decreases. These relations have been investigated and verified by Woods and Litehiser (19), the U.S. Navy (20), Narayama Murty (21), and PFRA (7,8) for a variety of soils. Lo (3) gives a summary of regression equations, together with their geographic regions of applicability, for the prediction of OMC and ρ_{dmax}.

The California bearing ratio (CBR) test is used to provide a low-deformation measure of strength of compacted subgrade soil and is used with empirical curves to design asphalt pavement structures. In the literature, CBR values have been predicted by means of index properties, strength characteristics, and soil classification units. A relation between the CBR and the group index (GI) was suggested by the Asphalt Institute (22) and later by Gawith and Perrin (23). Both CBR values were measured at 90 percent modified American Association of State Highway and Transportation Officials (AASHTO) maximum dry density. As the GI increases, the CBR decreases. Kassiff, Livneh, and Wiseman (24) found that, for Israel soils, the CBR values increase with decrease in the difference between plastic limit (w_p) and shrinkage limit (SL) and with increase in surcharge.

Robinson and Lewis (25, p. 72) showed that a relation existed between the CBR value and failure load P (in pounds) of a 3-in-square plate pushed into the ground. A rational approach known as the suction method was proposed by Black (26) to estimate CBR values from plasticity and consistency indices. Recently, Black and Lister (27) found that CBR = $c_u/23$, where c_u is the undrained shear strength in kilopascals.

The U.S. Navy (20) correlated the values of typical characteristics such as ρ_{dmax}, OMC, and CBR of compacted materials against the Unified Soil Classification system. The American Hoist and Derrick Company (28) used the AASHTO classification system, along with qualitative descriptions of soil characteristics, to approximate CBR values. Having made a comparison of groups in the AASHTO and Unified Soil Classification systems, Liu (29) proposed approximate relative relations of various groups of both systems to CBR values.

Strength Parameters

The British Road Research Laboratory (30) described a curvilinear relation between unconfined compressive strength (q_u) and w_n for a heavy clay in situ. Peters and Lamb (7) presented an empirical relation between LI and q_u for the soils in western Canada. They showed that, as LI decreases, the unconfined compressive strength increases. Peck, Hanson, and Thornburn (31) suggested a relation between the qualitative terms describing consistency, along with field identification of clays, and the quantitative values of q_u. The National Research Council of Canada (32) also suggested a similar relation for the rough estimate of the undrained shear strength (half of the unconfined compressive strength) for clay soils.

Unconfined compressive strength (q_u) has been correlated with the standard penetration test (SPT)--i.e., the number of blows (N-values) for 1-ft penetration. The U.S. Navy (20) recommended simple relations for clayey silts, CL clays, or varved clays and silts. Terzaghi and Peck (33) also suggested relations among the consistency of clay described in qualitative terms, N-values, and q_u. Sanglerat (34) recommended additional relations.

Summary

A review of the large number of empirical relations documented in the geotechnical engineering literature leads to two principal conclusions:

1. The values of soil parameters are expressed as means.
2. The functional relations among soil characteristics are established with data from a certain region or pooled data from several regions. The regional effects on relations are not commonly investigated or compared.

The random process of soil formation explains the great variation often encountered for a given soil parameter. Therefore, it seems more reasonable to define and use the median, rather than mean, for soil parameter values. The regional effects on functional relation among soil characteristics can be investigated by using the techniques of qualitative variables as regressors associated with analysis of variance. The details of these procedures are discussed and illustrated in the following section.

DEVELOPMENT OF GEOTECHNICAL DATA BANK FOR INDIANA

Goldberg (1,2) initiated the development of the Indiana geotechnical data bank. A total of 2508 data sets were collected and subjected to statistical analyses in an initial research phase. An additional 6934 data sets were added to the bank in the subsequent year and subjected to more detailed statistical analyses. As of January 1980, the Indiana geotechnical data bank contained 9442 data sets. The distribution of data sets throughout the state is presented by Lo (3).

Source and Structure of Data

Both geotechnical and pedological soils information was collected. The geotechnical information was taken from the subsurface investigation reports of various geotechnical projects previously conducted in Indiana, and the pedological soils information was from recent U.S. Department of Agriculture soil survey manuals (35) and general soils maps (36).

A data input form was developed to record the information. Details of the listing and their corresponding coding systems are described by Goldberg (1). Computer programs were written in the Statistical Package for the Social Sciences (SPSS) language available in the Purdue University Computing Center for CDC 6500 and 6600 systems. The SPSS language, like any other recent statistical language, is a "conversational" statistical analysis software. It also has features of data manipulation, data transformation, file definition, and file creation (37). This study has relied heavily on the SPSS language. However, any computer language is merely a means of access to an optimum use of the computer hardware. Therefore, the logical sequences of a program are more important than the program itself.

Methods of Analysis

Two types of prediction equations were used in this work: (a) median models and (b) regression models. To define numerically the variability of selected soil characteristics, the frequency distributions of these characteristics are examined and described. One way to describe the sample distribution is to use the conventional constant mean model (38), which is based on the assumption of normality of the population distribution. This model is characterized by the mean and the standard deviation of the sample distribution. It is shown (39) that in most cases this model is also effective for moderately abnormal distributions of a large sample. In the case of small samples, however, this model may not give accurate approximations. The nonparametric or distribution-free methods are the preferred techniques of inference for nonnormal population (40). These methods make a minimum of assumptions regarding the sample distribution and are generally appropriate for any form of the distribution. They are of high efficiency in comparison with classical techniques, under the assumption of normality, and are often of higher efficiency in other situations (39-41). In this study, the sample distributions were sometimes normal but frequently skewed or bimodal. The median model was used to characterize the sample distribution.

Distributional data are covered in detail by Lo (3) and will not be further considered here.

Regression Models

Regression analysis provides a conceptually simple method for investigating functional relations among variables. In general, the first stage of the analysis is to select the variables to be included in the regression model. This is done based on theory or former examples or by other procedures.

The most thorough approach, known as the all-possible-regression method, is to develop the regression of y (dependent variable) on every subset of the kx variables (independent variables). The major drawback of this method is the amount of computation. Another approach for selecting variables, and the one used in this study, is the stepwise regression method (42,43). It is recommended that the stepwise procedures be applied only to noncollinear data and that the order in which the variables enter or leave the equation not be interpreted as reflecting the relative importance of the variables (42).

In entering the variables to formulate a regression model, a question arises concerning the form of each variable--i.e., whether the variable should enter the model as an original variable x or as some transformed variable such as x^2, log x, or a combination or both. If, from an examination of scatter plots of y versus x, the relation between y and x appears to be nonlinear, appropriate transformations of the data are introduced to produce linearity (42). In this study, all variables, their possible transformations, and their combinations were included in the stepwise procedure for selecting variables as long as they were not collinear.

The variables were selected to minimize the mean square due to error of the prediction. Because a large value of R^2 [square of (multiple) correlation coefficient] or a significant t-statistic does not ensure that the data were well fitted (44), a careful residual analysis was also made. The procedure to reduce the number of independent variables was to compare the full model and the reduced model by using the F-statistic (42).

It was believed that the soil was more homogeneous in a small geologic or pedologic unit. Therefore, the regression models were established on these units. It was often found that for a given dependent variable the regression models generated in this way used different sets of independent variables for various locations. It seems unwise to conclude that these differences are caused by soil differences alone.

The effects of soil location and genesis--namely, physiographic region and parent material--were investigated by means of the statistical technique of using qualitative variables as regressors (42). In order to do so, the qualitative variables were represented by dummy variables that take on only two values, usually zero and one. These two values designated whether the observation belonged in one of two possible categories. Accordingly, the number of these variables required was one less than the number of categories in a grouping unit. Goldberg (1) shows that for Indiana the physiographic regions are coded from 1 to 12 and the parent materials from 1 to 13. The dummy-variable indicators were set up as follows:

$$x_i \begin{cases} = 1 \text{ if soil sample is taken from physiographic region coded i} \\ = 0 \text{ otherwise} \end{cases} \quad (1)$$

where i = 1,2,3, ..., 11; and

$$z_j \begin{cases} = 1 \text{ if soil sample is derived from parent material coded j} \\ = 0 \text{ otherwise} \end{cases} \quad (2)$$

where j = 1,2,3, ..., 12.

For the soil sample taken from the physiographic region coded 13, let $x_1 = x_2 = x_3 = \ldots = x_{12} = 0$. For the soil sample derived from the parent material coded 12, let $z_1 = z_2 = z_3 = \ldots = z_{11} = 0$.

Assume that the following relation exists:

$$C_c = a_0 + a_1 w_n + a_2 e_o + a_3 w_L \quad (3)$$

To investigate how these two soil grouping units affect this relation singly or in combination, the F-statistics were used in making comparisons of the following models:

Model	Equation
1	$C_c = a_0 + a_1 w_n + a_2 e_o + a_3 w_L$
2	$C_c = a_0' + a_1' w_n + a_2' e_o + a_3' w_L + a_4' x_1 + a_5' x_2 + \ldots + a_{15}' x_{12}$
3	$C_c = a_0'' + a_1'' w_n + a_2'' e_o + a_3'' w_L + a_4'' z_1 + a_5'' z_2 + \ldots + a_{14}'' z_{11}$
4	$C_c = a_0''' + a_1''' w_n + a_2''' e_o + a_3''' w_L + a_4''' x_1 + a_5''' x_2 + \ldots + a_{15}''' x_{12} + a_{16}''' z_1 + a_{17}''' z_2 + \ldots + a_{26}''' z_{11}$

Application of Regression Models

The regression models were used to correlate soil design parameters, such as those of compaction, consolidation, and strength, with soil index properties. The following examples illustrate how to apply the procedures described in the previous sections to arrive at the regression model for ρ_{dmax} and for C_c by using index properties.

It was found that ρ_{dmax} correlated well with w_L and w_p. There should also be an examination of whether the interaction between w_L and w_p--i.e., $(w_L \cdot w_p)$--has any significant effect on ρ_{dmax}. Therefore, the following independent variables enter the stepwise regression program for consideration: w_L, w_L^2, w_p, w_p^2, $(w_L \cdot w_p)$, percentage of sand, and percentage of silt.

The F-statistic tests were used to reduce the number of independent variables and produce the final model in the following form:

$$\hat{\rho}_{d\,max} = -0.554\,w_L - 0.0900\,\text{silt} - 0.727\,w_p + 0.008\,49\,w_L w_p + 142.888 \qquad (4)$$

where

$|R| = 0.808$,
SD = 4.994, and
n = 601.

For a given predicted ρ_{dmax} $(\hat{\rho}_{dmax})$, about 68 percent of sample observations (measured ρ_{dmax}) fall in the range of $\hat{\rho}_{dmax}$ + 4.994 (pcf) and $\hat{\rho}_{dmax}$ − 4.994 (pcf).

To investigate the effects of physiographic regions and parent materials on Equation 4, the dummy-variable indicators were set up as described previously. The F-statistic tests were then used to examine the value of physiographic regions and parent materials in the model. Neither added significant information (3). Therefore, Equation 4 was the final model for ρ_{dmax}.

For the soil variable of C_c, it was found that effects of both physiographic regions and parent materials did add significant information statistically to the regression model of C_c on index properties. The regression model was found to be

$$\begin{aligned}\hat{C}_c = &-0.151 + 0.003\,26\,w_n + 0.191\,e_o + 0.003\,25\,w_L \\ &+ 0.0162\,x_1 - 0.0110\,x_2 + 0.0208\,x_3 + 0.0296\,x_4 \\ &+ 0.0120\,x_5 - 0.0110\,x_6 + 0.0365\,x_7 + 0.0351\,x_8 \\ &+ 0.0646\,x_9 + 0.0649\,x_{10} - 0.0594\,x_{11} - 0.0245\,z_1 \\ &- 0.0313\,z_2 - 0.009\,87\,z_3 - 0.0917\,z_4 - 0.121\,z_6 \\ &- 0.0292\,z_7 - 0.0667\,z_8 + 0.008\,41\,z_9 - 0.0418\,z_{10} \\ &- 0.008\,84\,z_{11} \end{aligned} \qquad (5)$$

where

$|R| = 0.952$,
SD = 0.0670, and
n = 302.

Figures 1 and 2 show the scatter plots with regression lines and 95 percent population confidence intervals for the measured ρ_{dmax} (ρ_{dmax}) and predicted ρ_{dmax} $(\hat{\rho}_{dmax})$ determined by using Equation 4 and the measured C_c (C_c) and predicted C_c $(\hat{C}_c)$ determined by using Equation 5. In the presentation of the data, the solid line represents the best fit line whereas the dashed lines define the boundaries of 95 percent population confidence intervals.

DISCUSSION OF RESULTS

In this study, both median and regression models were developed for statistical forecasting. Predictions are extrapolations into the future of features shown by relevant data in the past. Therefore, a considerable population of values for the dependent variable is required. Another basic requirement for prediction is the existence of a stable data structure. The trend of the data and the statistical variation about the trend must be stable. This can be detected by the confidence intervals and interquartile range for a median model or the standard deviation of estimate and multiple correlation coefficient (|R|) for a regression model. A large difference between the confidence intervals, a large value of interquartile range for a median model, or a large standard deviation of estimate for a regression model indicates that the data structure is not stable. Either more data or a change in grouping unit is needed for better prediction.

Regression Models and Correlations

Regression models were used to correlate soil design parameters such as compaction, consolidation, and strength with index properties. In Table 1, a number of successful regression equations are represented together with their correlation coefficients (R) or multiple correlation coefficients (|R|), standard deviations (or errors) of estimate (SD), and the number of cases (n). The standard deviation of estimate is important, since it represents the variation of estimate $(\hat{y})$; i.e., 68 percent of sample observations (y) fall in the range of $\hat{y}$ − SD and $\hat{y}$ + SD. Specific results are presented below.

Compaction Parameters and CBR

The correlations of ρ_{dmac} and OMC versus plasticity characteristics, as given in Table 1, indicate that, as w_L or w_p increases, OMC increases but ρ_{dmax} decreases. In addition, as OMC increases, ρ_{dmax} decreases. A relation between CBR values at 100 and 95 percent maximum dry densities is also developed and presented in Table 1.

Consolidation Parameters

With either w_L, w_n, or e_o, C_c increases. The quantity of C_r' is a linear function of e_o, as given in Table 1. C_r is usually taken as a fraction of C_c. In this study, it is found that $C_r = A + B\,C_c$, where A is −0.003 27. The standard deviation of A is 0.001 99, B is 0.139, and the standard deviation of B is 0.007 26. Therefore, with 95 percent confidence the C_r will lie in the range of, approximately, $(1/6.5)C_c$ and $(1/8)C_c$ for Indiana soils. p_c was correlated with LI, but this correlation was not a strong one for Indiana soils.

Strength Parameters

The relations of strength and strength parameters with simple index values were examined with little general success (3). However, w_L was found to be a function of w_n, SPT, and location factors--i.e., physiographic regions (x's) and parent material (z's), as given in Table 1.

CONCLUSIONS

A computerized data storage and retrieval system has been developed for the State of Indiana. Both conventional and nonparameteric statistical methods have been used in the analysis of these data. Regression analysis (Table 1) showed certain good to adequate functional relations between design parameters and index properties, as follows:

1. As the liquid limit or plastic limit of a soil increases, the optimum moisture content increases but maximum dry density decreases. This confirms the findings of earlier authors.

2. A unique relation exists between optimum moisture content and maximum dry density, which confirms the findings of earlier authors.

3. The CBR value is a function of plasticity characteristics, and a correlation exists between the CBR value at 100 percent maximum dry density and the CBR value at 95 percent maximum dry density, which confirms the findings of earlier authors.

4. Compression index (C_c) is a function of natural moisture content, initial void ratio, and liquid limit and is significantly influenced by geological factors, which confirms the findings of earlier authors. With 95 percent confidence, the recompression index (C_r) lies in the range of, approximately, $(1/6.5)C_c$ and $(1/8)C_c$ for Indiana soils.

5. The preconsolidation pressure (p_c) is a function of natural water content, initial void ratio, and natural dry density and is significantly influenced by geological factors. But the scatter in the data is large.

6. No definite correlation of strength angle and cohesion intercept versus plasticity characteristics was found for Indiana soils.

7. It is emphasized that the data bank is not proposed as a substitute for fuller site investigation, sampling, and testing but as a framework against which various test results can be judged for their consistency and reliability. It can also be enormously helpful in preliminary assessment of a site or route.

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Figure 1. Measured maximum dry density versus predicted maximum dry density for Indiana soils.

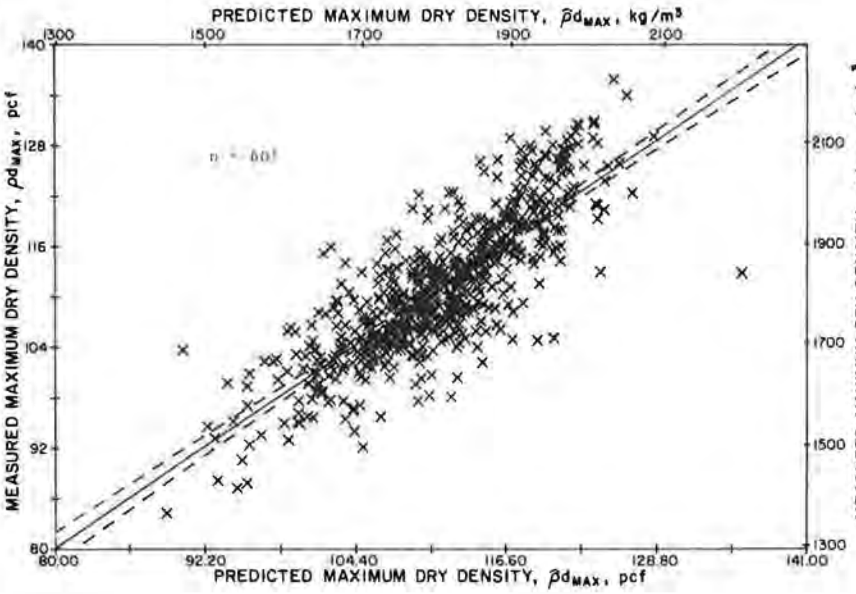

Figure 2. Measured compression index versus predicted compression index for Indiana soils.

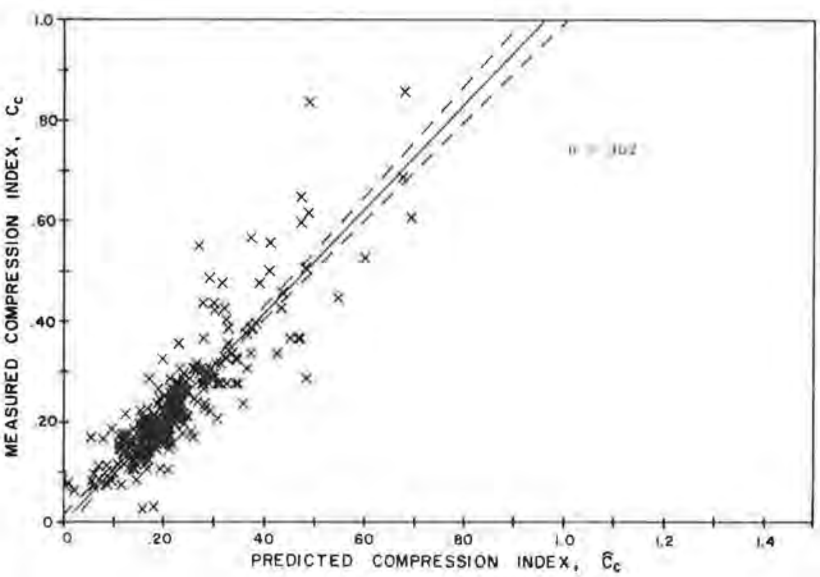

Table 1. Summary of regression equations and correlations.

Parameter	Equation	Statistical Measure
Compaction		
$\rho_{d\,max}$	$\hat{\rho}_{d\,max} = 135.843 - 1.279\ w_p$	$R = -0.692$ $SD = 6.434$ $n = 601$
	$\hat{\rho}_{d\,max} = 128.338 - 0.463\ w_L$	$R = 0.744$ $SD = 5.651$ $n = 601$
	$\hat{\rho}_{d\,max} = 142.888 - 0.554\ w_L - 0.727\ w_p + 0.008\,49\ w_L w_p - 0.0900\ \text{silt}$	$\|R\| = 0.808$ $SD = 4.994$ $n = 601$
OMC	$\hat{OMC} = 4.464 + 0.619\ w_p$	$R = 0.698$ $SD = 2.905$ $n = 596$
	$\hat{OMC} = 7.626 + 0.237\ w_L$	$R = 0.794$ $SD = 2.464$ $n = 596$
	$\hat{OMC} = 7.457 - 0.0369\ \text{sand} + 0.0174\ \text{silt} + 0.171\ w_L + 0.155\ w_p - 0.413\ x_1 - 0.401\ x_2 - 1.740\ x_3 + 0.968\ x_4 - 0.409\ x_5 + 0.503\ x_6 - 0.186\ x_7 - 0.560\ x_8 + 0.386\ x_9 - 0.463\ x_{10} - 0.752\ x_{11}$	$\|R\| = 0.843$ $SD = 2.210$ $n = 596$
$\rho_{d\,max}$ versus OMC	$\hat{\rho}_{d\,max} = 150.667 - 3.016\ OMC + 0.0333\ (OMC)^2$	$\|R\| = 0.906$ $SD = 3.691$ $n = 701$
CBR at 100 percent $\rho_{d\,max}$ (CBR S01)	$\log \hat{CBR}\ S01 = 1.204 + 0.145 \log w_L - 0.137 \log PI - 0.149 (\log PI)(\log w_L) - 0.0778\ z_1 - 0.109\ z_2 + 0.112\ z_3 - 0.0505\ z_4 - 0.0858\ z_5 - 0.122\ z_6 - 0.0575\ z_7 - 0.0633\ z_8 - 0.0587\ z_9 - 0.105\ z_{10} - 0.688\ z_{11}$	$\|R\| = 0.620$ $SD = 0.166$ $n = 493$
CBR S01 versus CBR at 95 percent $\rho_{d\,max}$ (CBR S02)	$\hat{CBR}\ S02 = 1.339 + 0.433\ CBR\ S01$	$R = 0.851$ $SD = 2.010$ $n = 553$
	$\hat{CBR}\ S02 = 0.051 + 0.667\ CBR\ S01 - 0.007\,60\ (CBR\ S01)^2$	$\|R\| = 0.864$ $SD = 1.515$ $n = 553$
Consolidation		
C_c	$\hat{C}_c = 0.007\,97\ (w_L - 8.16)$	$R = 0.829$ $SD = 0.116$ $n = 312$
	$\hat{C}_c = 0.0126\ w_n - 0.162$	$R = 0.925$ $SD = 0.112$ $n = 332$
	$\hat{C}_c = 0.496\ e_o - 0.195$	$R = 0.873$ $SD = 0.143$ $n = 335$
	$\hat{C}_c = -0.151 + 0.003\,26\ w_n + 0.191\ e_o + 0.003\,25\ w_L + 0.0162\ x_1 - 0.0110\ x_2 + 0.0208\ x_3 + 0.0296\ x_4 + 0.0120\ x_5 - 0.0110\ x_6 + 0.0365\ x_7 + 0.0351\ x_8 + 0.0646\ x_9 + 0.0649\ x_{10} + 0.0594\ x_{11} - 0.0245\ z_1 - 0.0313\ z_2 - 0.009\,87\ z_3 - 0.0917\ z_4 - 0.121\ z_6 - 0.0292\ z_7 - 0.0667\ z_8 + 0.008\,41\ z_9 - 0.0418\ z_{10} - 0.008\,84\ z_{11}$	$\|R\| = 0.952$ $SD = 0.0670$ $n = 302$
C_r'	$\hat{C}_r' = 0.0125 + 0.152\ e_o$	$R = 0.704$ $SD = 0.0448$ $n = 333$
	$\hat{C}_r' = 0.0249 + 0.003\ w_n$	$R = 0.701$ $SD = 0.0361$ $n = 325$
	$\hat{C}_r' = 0.0294 + 0.002\,38\ w_L$	$R = 0.665$ $SD = 0.0373$ $n = 309$
C_c versus C_r'	$\hat{C}_c = 0.0844 + 9.121\ (C_r')^2$	$\|R\| = 0.948$ $SD = 0.0928$ $n = 339$
C_r versus C_c	$\hat{C}_r = -0.003\,27 + 0.139\ C_c$	$R = 0.743$ $SD = 0.0173$ $n = 298$
Strength		
LL versus SPT	$\hat{w}_L = 47.946 + 0.495\ w_n - 9.931 \log SPT + 0.488\ w_n \log SPT - 19.323\ x_1 - 25.121\ x_2 - 30.455\ x_3 - 17.775\ x_4 - 28.875\ x_5 - 24.529\ x_6 - 26.068\ x_7 - 26.272\ x_8 - 24.100\ x_9 - 16.493\ x_{10} - 37.182\ x_{11} + 9.104\ z_1 - 2.930\ z_2 - 3.809\ z_3 - 0.378\ z_4 - 12.178\ z_5 - 5.404\ z_6 - 7.021\ z_7 + 23.069\ z_8 - 0.249\ z_9 + 0.962\ z_{10} + 13.878\ z_{11}$ (in %)	$\|R\| = 0.859$ $SD = 19.503$ $n = 533$

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REFERENCES

1. G.D. Goldberg. Development of the Computerized Geotechnical Data Bank for the State of Indiana. Purdue Univ., West Lafayette, IN, M.S.C.E. thesis and Joint Highway Research Project Rept. 76-6, May 1978, 163 pp.
2. G.D. Goldberg, C.W. Lovell, and R.D. Miles. Use

the Geotechnical Data Bank! TRB, Transportation Research Record 702, 1978, pp. 140-146.
3. Y.-K.T. Lo. Geotechnical Data Bank for Indiana. Purdue Univ., West Lafayette, IN, Ph.D. thesis and Joint Highway Research Project Rept. 80-7, Aug. 1980.
4. R.A. Crawford, J.B. Thomas, and M. Stout, Jr. Computerized Soil Test Data for Highway Design. Physical Research Section, South Dakota Department of Highways, Pierre, 1972, 24 pp.
5. D. Spradling. A Soils Data System for Kentucky. Division of Research, Bureau of Highways, Kentucky Department of Transportation, Frankfort, Res. Rept. 441, 1976, 58 pp.
6. Data Transformation Corporation. Materials and Test Data Information Systems in Highway Construction. FHWA, Rept. FHWA-TS-78-221, 1978, 156 pp.
7. N. Peters and K.N. Lamb. Experiences with Alluvial Foundations for Earth Dams in the Prairie Provinces. Canadian Geotechnical Journal, Vol. 16, No. 2, 1979, pp. 255-271.
8. P.J. Rivard and T.E. Goodwin. Geotechnical Characteristics of Compacted Clays for Earth Embankments in the Prairie Provinces. Canadian Geotechnical Journal, Vol. 15, No. 3, 1978, pp. 391-401.
9. S.E. Lundin, O. Stephansson, and P. Zetterlund. Geoteknisk Databank. National Swedish Building Research Institute, Stockholm, Rept. R70, 1973, 158 pp.
10. Lille Regional Laboratory. Establishment of a Card Index for Borings and Tests. Ministry of Public Works, Paris, France, Jan. 1971.
11. A. Holden. Engineering Soil Mapping from Airphotos. Ministry of Roads and Traffic, Salisbury, Rhodesia, July 1969.
12. J.P. Redon. Computerized Inventory of Soil Profiles Throughout Algeria. National Laboratory of Public Works and Buildings, Algiers, Algeria, Sept. 1972.
13. Acquisition and Use of Geotechnical Information. NCHRP, Synthesis of Highway Practice 33, 1976, 40 pp.
14. P.C. Rutledge. Compression Characteristics of Clays and Application to Settlement Analysis. Harvard Univ., Cambridge, MA, thesis, 1939.
15. R.E. Fadum. Observations and Analysis of Building Settlements in Boston. Harvard Univ., Cambridge, MA, thesis, 1941.
16. G.A. Leonards. Engineering Properties of Soils. In Foundation Engineering (G.A. Leonards, ed.), McGraw-Hill, New York, 1962, pp. 145-156.
17. L. Bjerrum. Embankments on Soft Ground. Proc., ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Lafayette, IN, Vol. 11, 1972, pp. 1-54.
18. R.R. Proctor. Fundamental Principles of Soil Compaction. Engineering News-Record, Vol. 3, Aug. 31, 1933, pp. 245-248.
19. K.B. Woods and R.R. Litehiser. Soil Mechanics Applied to Highway Engineering in Ohio. Engineering Experiment Station, Ohio State Univ., Columbus, Bull. 99, 1938.
20. Design Manual: Soil Mechanics, Foundations, and Earth Structures. Naval Facilities Engineering Command, U.S. Department of the Navy, Alexandria, VA, NAVFAC DM-7, March 1971, pp. 7-3-8, 7-9-2, and 7-10-1 to 7-10-23.
21. P.L. Narayama Murty. Relationship Between the Plasticity Characteristics and Compaction of Soils. Journal of Indian National Society of Soil Mechanics and Foundation Engineering, Vol. 4, No. 4, 1965, pp. 455-463.
22. Thickness Design: Asphalt Pavement Structures for Highways and Streets. Asphalt Institute, College Park, MD, MS-1, Dec. 1965, 95 pp.
23. A.H. Gawith and C.C. Perrin. Development in the Design and Construction of Bituminous Surfaced Pavements in the State of Victoria, Australia. Presented at International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1962, pp. 897-910.
24. G. Kassiff, M. Livneh, and G. Wiseman. Pavements on Expansive Clays. Jerusalem Academic Press, Jerusalem, Israel, 1969, pp. 103-117.
25. P.J.M. Robinson and T. Lewis. A Rapid Method of Determining In Situ CBR Values. Geotechnique, Vol. 8, No. 2, 1958.
26. W.P.M. Black. A Method of Estimating CBR of Cohesive Soils from Plasticity Data. Geotechnique, Vol. 12, No. 4, 1962, pp. 271-282.
27. W.P.M. Black and N.W. Lister. The Strength of Clay Fill Subgrades: Its Predictions in Relation to Road Performance. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1979, 30 pp.
28. Handbook of Soil Compactionology. American Hoist and Derrick Co., St. Paul, MN, 1977, 53 pp.
29. T.K. Liu. A Review of Engineering Soil Classification Systems. HRB, Highway Research Record 156, 1967, pp. 1-22.
30. Soil Mechanics for Road Engineers. British Road Research Laboratory, London, England, 1952.
31. R.B. Peck, W.E. Hanson, and T.H. Thornburn, eds. Foundation Engineering, 2nd ed. Wiley, New York, 1974, pp. 20, 114, 307-330.
32. Canadian Manual On Foundation Engineering. National Research Council of Canada, Ottawa, Ontario, draft, 24 pp.
33. K. Terzaghi and R.B. Peck. Soil Mechanics in Engineering Practice, 2nd ed. Wiley, New York, 1967, pp. 73, 112, 193-216.
34. G. Sanglerat. The Penetrometer and Soil Exploration. Elsevier Publishing Co., Amsterdam, Netherlands, 1972, 464 pp.
35. Soil Survey Manuals. Soil Conservation Service, U.S. Department of Agriculture, 1958 to present.
36. General Soils Maps and Interpretation Tables for the Counties of Indiana. Agriculture Experiment Station/Cooperative Extension Service, Purdue Univ., West Lafayette, IN; Soil Conservation Service, U.S. Department of Agriculture, Nov. 1971.
37. N.H. Nie and others. Statistical Package for the Social Sciences, 2nd ed. McGraw-Hill, New York, 1975, 675 pp.
38. W. Gilchrist. Statistical Forecasting. Wiley, New York, 1976, 300 pp.
39. H.J. Larson. Introduction to the Theory of Statistics. Wiley, New York, 1973, 242 pp.
40. G.W. Snedecor and W.G. Cochran. Statistical Methods, 6th ed. Iowa State Univ. Press, Ames, 1967, 593 pp.
41. M. Hollander and D.A. Wolfe. Nonparametric Statistical Methods. Wiley, New York, 1973, 503 pp.
42. S. Chatterjee and B. Price. Regression Analysis by Example. Wiley, New York, 1977, 228 pp.
43. N.R. Draper and H. Smith. Applied Regression Analysis. Wiley, New York, 1966, 407 pp.
44. F.J. Anscombe. Graphs in Statistical Analysis. American Statistician, Vol. 27, 1973, pp. 17-21.

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Field Evaluation of Rapid Data Collection System for Rock Slope Stability Analysis

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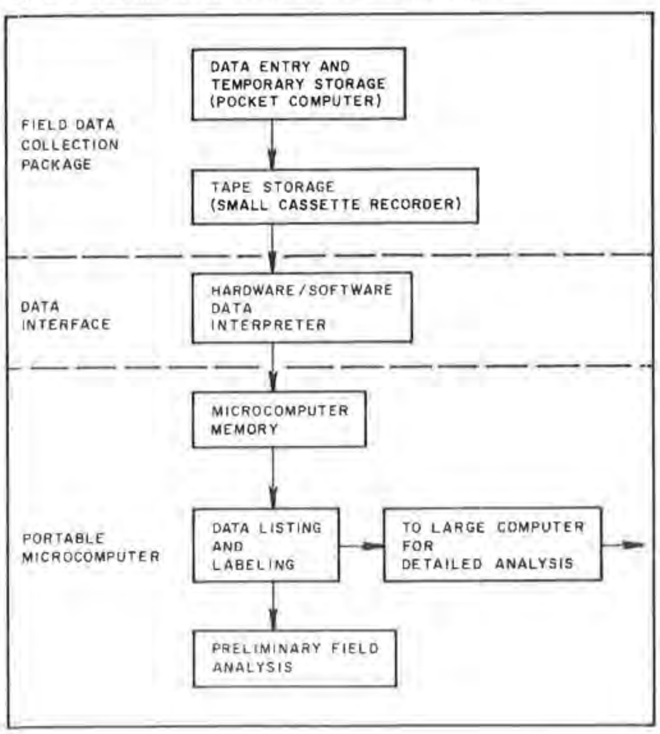
A system for rapid collection and evaluation of geologic-structure data for rock slope stability analysis developed by the engineering geology group at Purdue University is described. The data collection system allows for rapid, direct collection of field data and provides a preliminary computer analysis of the slope while on site. It consists of (a) a lightweight data collection package carried in the field that stores on magnetic tape orientations and characteristics of rock discontinuities; (b) a specially designed computer and software interface that provides data exchange from collection package to microcomputer; and (c) a fairly inexpensive, portable microcomputer maintained in a field vehicle or motel room. Field testing of the system began in May 1981 in the Valley and Ridge Province of Virginia with the cooperation of the Virginia Department of Highways and Transportation. Three highway road cuts were examined in detail. As a result of information obtained during the testing process, a number of changes to the standard data collection procedure are recommended that tend to streamline collection of structural rock slope information. The efficiency of the rapid data collection system is compared with that of standard methods. The advantages of the new system are enumerated.

The complete stability analysis of a given rock slope depends on a detailed survey of the orientations and characteristics of discontinuities within the rock mass. A discontinuity is defined as a structural weakness plane or surface along which movement can take place. Types of rock discontinuities are joints, faults, shear zones, bedding surfaces, and foliations. The significant characteristics of discontinuities are (a) geometry, (b) continuity, (c) spacing, (d) surface irregularities, (e) physical properties of adjacent rock, (f) infilling material, and (g) groundwater (1). Field surveys conducted to collect these data are extremely time consuming, and there are generally significant time lapses between the mapping and the detailed analysis of the data.

Figure 1. Flow chart of data collection and evaluation system.



A computer-based system for rapid collection and evaluation of geologic-structure data for rock slope stability analysis has been developed by the engineering geology group at Purdue University. It was field tested during the 1981 summer season with the cooperation of the Virginia Department of Highways and Transportation. The prototype system has been described in considerable detail by Watts and West (2).

The rapid data collection and evaluation system consists of three components (see Figure 1):

1. A lightweight data collection package carried in the field that stores on magnetic tape orientations and characteristics of rock discontinuities (Figure 2);
2. A specially designed computer hardware and software data interface that provides for data ex-

Figure 2. Computerized data collection field package which interfaces to microcomputer system.

Figure 3. Portable microcomputer system used for data analysis.

change from the collection package to the microcomputer; and

3. A fairly inexpensive, portable microcomputer maintained in a field vehicle or motel room (see Figure 3).

The purpose of the system is to speed up the data collection process and to place the data into computer memory for analysis as quickly and as simply as possible. A number of different procedures and philosophies exist for the collection of geologic-structure data. Among these are the procedures outlined in two Federal Highway Administration (FHWA) short-course publications (3,4). The rapid data collection and evaluation system can be incorporated into any procedure dictated by personal preference and by site characteristics.

PRELIMINARY INVESTIGATION

The Virginia Department of Highways and Transportation selected three possible sites in Virginia for the field testing. This paper deals primarily with the first two sites; the third is mentioned briefly later. The two initial sites are located less than 0.75 mile from each other on US-460 near the town of Cedar Bluff. Prior to detailed mapping, a preliminary site investigation was conducted to give a sense of the regional geology and a feeling for the general structure of the site.

The sites are located in the folded and faulted sedimentary rock of the Valley and Ridge Province in southwestern Virginia. Figure 4 illustrates the general structure of that portion of the valley and ridge, and Figure 5 (modified from a field trip guide of the area by C. Ashbrook) shows a generalized geologic cross section near the sites.

Both of the cut slopes are in the Honaker dolomite group. The Maryville limestone member appears to make up the southernmost of the two slopes, and the Rogersville shale member is responsible for the

Figure 4. Major faults and folds of area surrounding the test site.

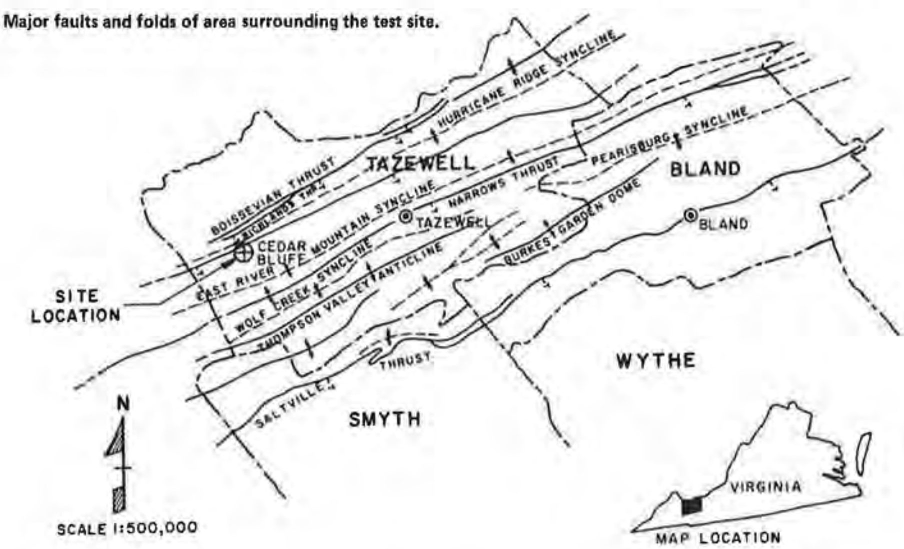


Figure 5. Generalized geologic cross section of Cedar Bluff, Ward Cove, and Thompson Valley area of Virginia.

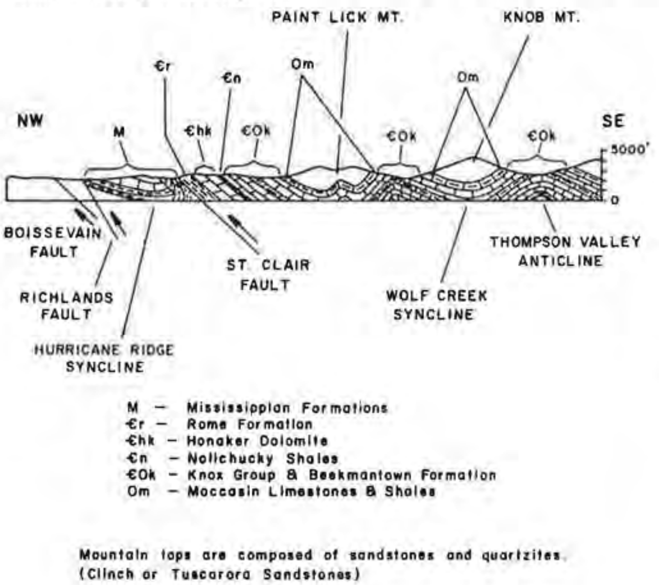


Figure 6. Contorted bedding and associated faulting near the St. Clair Fault in the roadcut.

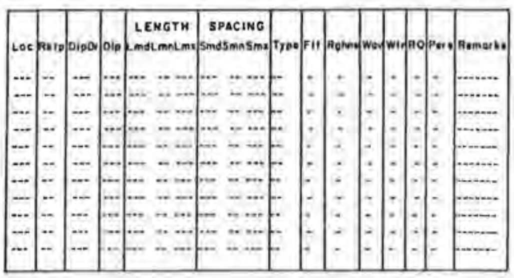
valley found between the two slopes. The remaining slope consists primarily of an equivalent to the Rutledge limestone member and a small slice of the Rome formation. The north end of the slope is cut by the St. Clair thrust illustrated in Figures 4 and 5. This fault brings these Cambrian formations into contact with overturned Devonian and Mississippian carbonaceous shales of the Hurricane Ridge syncline.

The northwest limb of this syncline is nearly flat and comprises the beginning of the Appalachian Plateau (5). The geologic structure becomes increasingly complex near the St. Clair fault, which results in intense folding and small-scale faulting (see Figure 6). Recent earthquake activity in Giles County, Virginia, may be the result of movement along the St. Clair thrust in that region (5).

GEOLOGIC-STRUCTURE DATA COLLECTION

Several methods of geologic-structure data collection are described in Appendix 6 of the FHWA publication, Rock Slopes (4). The structural data for this study were collected both with and without the computerized data collection package and by using two different procedures--window mapping and line mapping.

Figure 7. Typical window mapping form.



LOC = LOCATION
RKTP = ROCKTYPE
DIPDR = DIP DIRECTION
DIP = DIP
LMD = LENGTH MODE
LMN = LENGTH MINIMUM
LMX = LENGTH MAXIMUM
SMD = SPACING MODE
SMN = SPACING MINIMUM
SMX = SPACING MAXIMUM
TYPE = DISCONTINUITY TYPE
FLT = FILLING TYPE
RGHNS = ROUGHNESS
WAV = WAVINESS
WTR = WATER
RQ = ROCK QUALITY
PERS = PERSISTANCE

Window mapping consists of selecting a number of "windows" in the slope and recording geologic-structure data for all joint sets and all individual discontinuities that lie within that window. Figure 7 shows a typical collection form for such data. Unlike measurements in line mapping, distance measurements in window mapping are not generally made between individual joints, so estimates of minimum spacing, maximum spacing, and spacing mode are required for each joint set. Similar estimates are needed for the joint lengths of each set. The size and spacing of the windows depend on individual slope characteristics. Recent projects used 10-ft windows every 50 ft on both walls in tunnels and 30-ft windows every 150 ft for slopes (4). Areas between windows should be examined to ensure similarity to the areas being mapped. The advantage of window mapping is that it is faster than line mapping.

The detailed line method of data collection is described in the two FHWA publications cited earlier (3,4). Line mapping involves placement of a measuring tape along the slope face and recording data for every discontinuity that crosses the tape. These data include measurements of position along the tape so that spacing can be determined by computer analysis. The main advantage of line mapping is the control that it imposes on the collection of data for which statistical analysis may be desired. A major disadvantage of line mapping is that it becomes quite tedious when large areas are being mapped. It should be kept in mind that, although some of the data might not be useful in a quantitative way, they will nevertheless be useful to the slope investigator in determining the most likely failure surfaces. Figure 8 (4) shows an example of a typical data-recording form for line mapping.

Data collection with the computerized field package is quite similar to existing data collection procedures. The most obvious difference is that the data are collected by pressing buttons on a pocket computer instead of by writing them in a notebook. The pocket computer keeps track of what has been recorded and prompts the slope investigator on what is to be recorded next. Provisions exist for reviewing the collected data as well as for correcting them. One other difference is that, although letters may be used as data, the system is more efficient when only numbers are used. This means that, instead of using letter codes for such items as hardness and rock type, number codes should be used instead.

Figure 8. Structural mapping coding form.

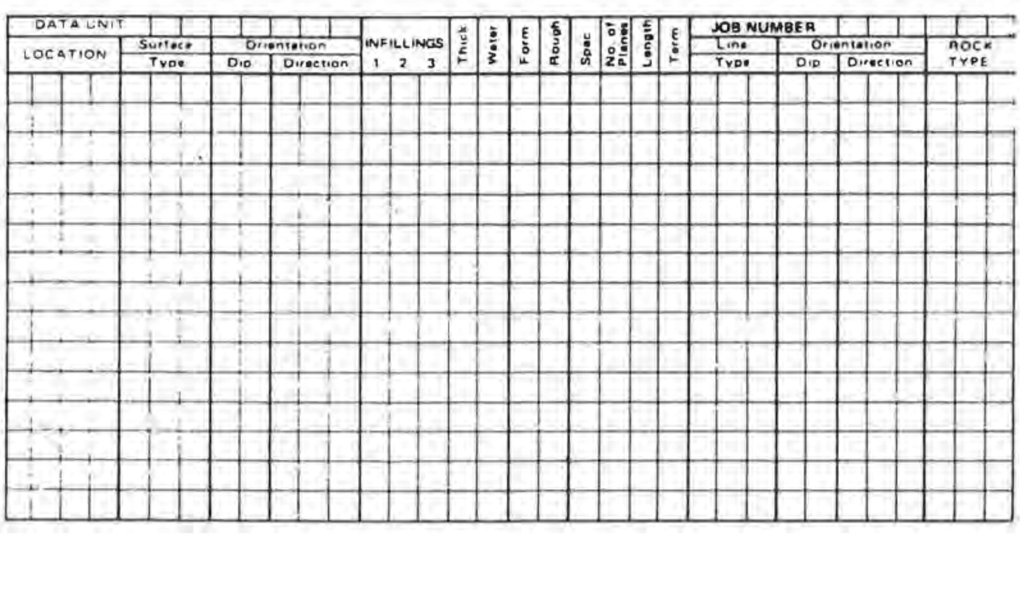


Figure 9 illustrates some of the data that were collected in this study. It closely follows the format found in Part G of Rock Slope Engineering (3). The same data format can be used for both window mapping and line mapping. Data can easily be omitted if they are not relevant to the mapping method being used. Finally, since some of the data are highly repetitive in nature, it is desirable not to reenter those data that are unchanged. If no entry is made for any prompt by the pocket computer and ENTER is pressed, the computer will simply assume that there has been no change in that item since the last entry. This feature helps to speed up the data collection process and to relieve some of the tedium of line mapping.

The actual amount of time saved in data collection by using the computerized method depends on the type of mapping undertaken and the specific procedures being followed. In this study, the computerized package appeared to reduce the data collection time by somewhere between 20 and 50 percent. However, the real time savings are not in data collection but in data listing and preliminary data analysis.

DATA LISTING AND PRELIMINARY ANALYSIS

The field data collected on cassette tape at the Cedar Bluff sites were loaded into the microcomputer and analyzed via the computer interfacing program. Each detailed line contained 100 joint readings. The data from the first detailed line were read into the computer, listed, and plotted on rectangular dip plots in about 30 min. A later, improved computer interfacing program reduced that time to about 20 min. Figure 9 illustrates a typical data listing. The rectangular dip plot for each line was automatically stored in computer memory for later use.

Next, the computer produced an equal area net plot of the poles of all of the discontinuities measured (see Figure 10). This is generally done to enable the slope investigator to pick out groupings or clusters of points that represent specific joint sets. The orientations of the clusters with respect to the slope face indicate which types of failures are likely to be present. Circular, planar, wedge, and toppling failures are possible, and various analytic techniques exist for each. Additional details regarding these failure modes and stereonet

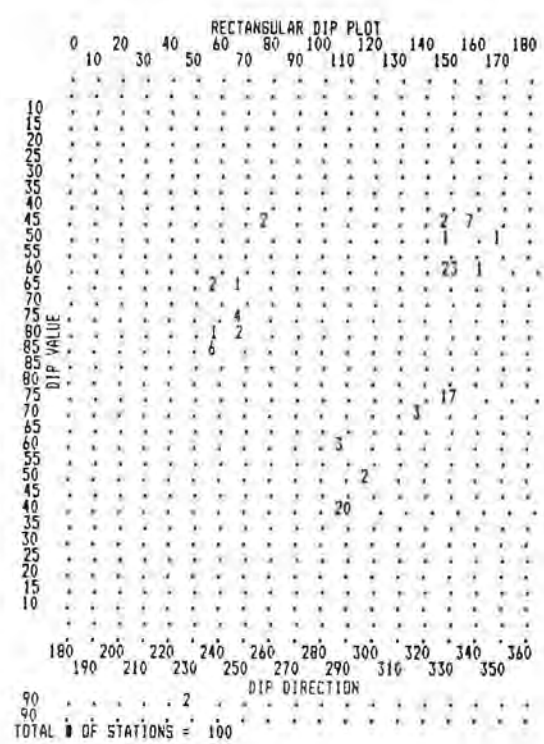
Figure 9. Data listing of last 20 discontinuities of typical detailed line from Cedar Bluff site and rectangular dip plot for entire line.

JNT#	TRAV	DIST	RKTP	HDS	STR	#JNS	JNSP	DIPDR	DIP	JNTLN	CONT	FLT	FLTH	FLHDS	WTR	RGHNS	WAVILA	WAVLN
1281	356	21.1	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1282	356	24.5	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1283	356	25.5	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1284	356	26.0	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1285	356	26.3	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1286	356	27.9	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1287	356	31.7	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1288	356	33.1	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1289	356	35.0	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1290	356	36.6	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0

JNT#	TRAV	DIST	RKTP	HDS	STR	#JNS	JNSP	DIPDR	DIP	JNTLN	CONT	FLT	FLTH	FLHDS	WTR	RGHNS	WAVILA	WAVLN
1291	356	38.5	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1292	356	41.0	2	04	04	00	00	146	62	100.0	2	003	1	00	2	2	000	000.0
1293	356	05.6	2	04	02	00	00	065	78	002.0	2	003	1	00	1	3	000	002.0
1294	356	09.7	2	04	02	00	00	072	80	002.0	2	003	1	00	1	3	000	002.0
1295	356	12.8	2	04	02	00	00	068	80	002.0	2	003	1	00	1	3	000	002.0
1296	356	19.5	2	04	02	00	00	071	75	002.0	2	003	1	00	1	3	000	002.0
1297	356	23.5	2	04	02	00	00	075	66	002.0	2	003	1	00	1	3	000	002.0
1298	356	29.0	2	04	02	00	00	074	75	002.0	2	003	1	00	1	3	000	002.0
1299	356	35.8	2	04	02	00	00	074	75	002.0	2	003	1	00	1	3	000	002.0
1300	356	43.5	2	04	02	00	00	075	74	002.0	2	003	1	00	1	3	000	002.0

KEY

JNT# = Joint number
TRAV = Traverse trend
DIST = Distance along traverse
RKTP = Rocktype
HDS = Hardness
STR = Structure type
#JNS = # of joints (within a given zone)
JNSP = Joint spacing (within a given zone)
DIPDR= Dip direction
DIP = Dip
JNTLN= Joint length
CONT = Continuity (or persistence)
FLT = Filling type
FLTH = Filling Thickness
FLHDS= Filling hardness
WTR = Water
RGHNS= Roughness
WAVILA=Waviness (inter-limb angle)
WAVLN= Wave length



representations are given by Hoek and Bray (6, p. 57). In this field testing situation, clusters were picked by eye. We have since developed several microcomputer programs that produce contoured output to assist in clustering (see Figure 11). Note that on the initial pole plot (Figure 10) the numbers indicate the number of poles falling at that point. If more than nine poles lie at the same point, letters are used. For example, A is equal to 10, B is equal to 11, and so on.

It was also determined that an equal area net plot of dip vectors can be extremely valuable in the slope analysis (Figure 12). A dip vector can be thought of as the stereonet representation of a line drawn along the direction of maximum dip on an inclined plane. On a stereographic projection, it plots as a single point at the position of maximum dip for the plane. In other words, it lies at the midpoint of the great circle that represents the plane. Just as there is only one pole for a given inclined plane, there is also only one dip vector. Two valuable characteristics of dip vectors were noted. First, the clustering of joints into joint sets by eye is often more easily accomplished by using both the dip vector plots and the pole plots than by using pole plots alone. Clusters that are not readily apparent on the pole plots, due to scattering of the poles near the edge of the net, can become apparent in dip vector plots and vice versa. Second, the dip vector plot can be especially useful for the identification of potential plane failure surfaces (see Figure 13). Dip vectors that fall within the critical zone, defined by the slope face and the friction angle, represent a possible failure. In other words, if the discontinuities daylight on the slope face and are steeper than the friction angle, they warrant additional study. This is referred to as Markland's Test and is described by Hoek and Bray (6, pp. 56-59). The dip vectors in Figure 13 have been clustered and numbered for comparison with Figure 14. Three of the clusters fall within the critical zone.

The next step in the preliminary slope analysis was the plotting of planes, representing discontinuities, on the equal area net for a wedge analysis (see Figure 14). Each great circle represents a cluster determined earlier. In this type of plot, the numbers represent a specific plane. For example, a line drawn through all number 2's with a flexible straight edge is the equal area net plot of

Figure 10. Pole plot of discontinuities from Cedar Bluff site (data points clustered by eye).

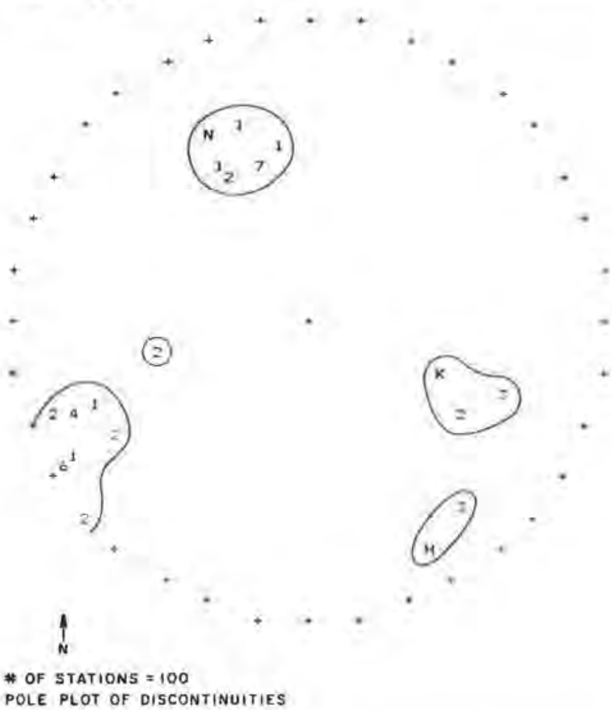


Figure 11. Data from Cedar Bluff computer contoured to assist in cluster recognition.

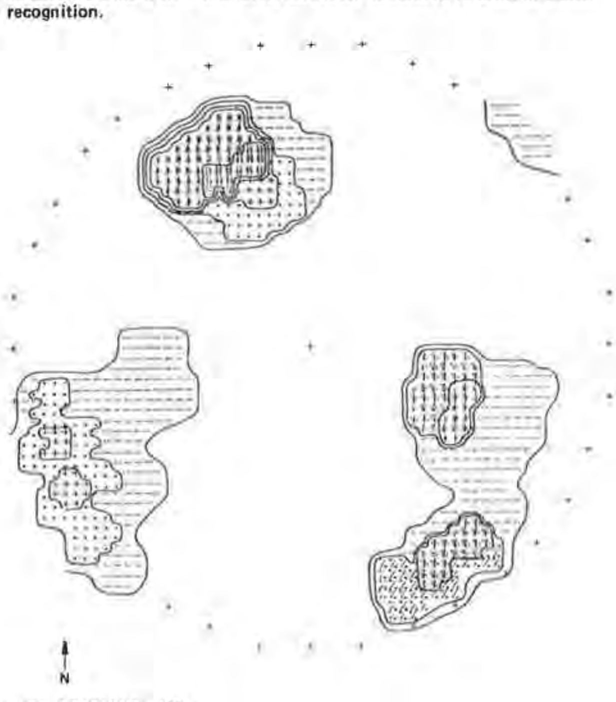
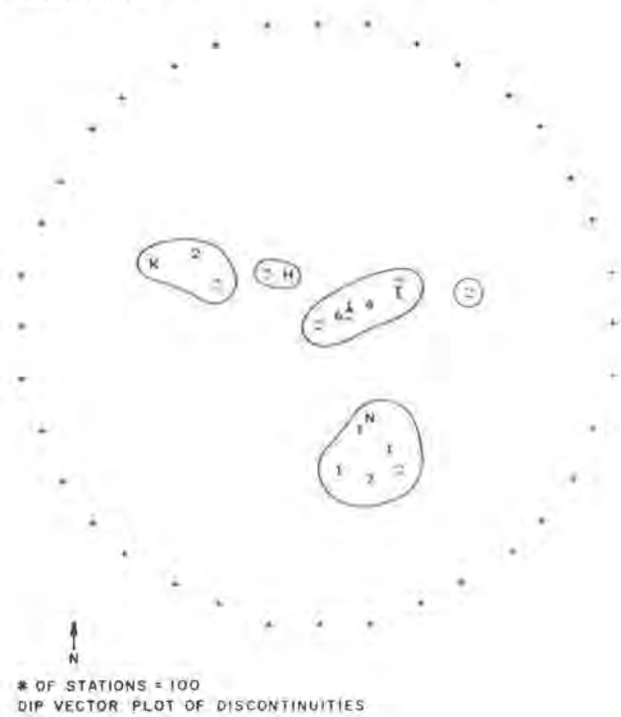
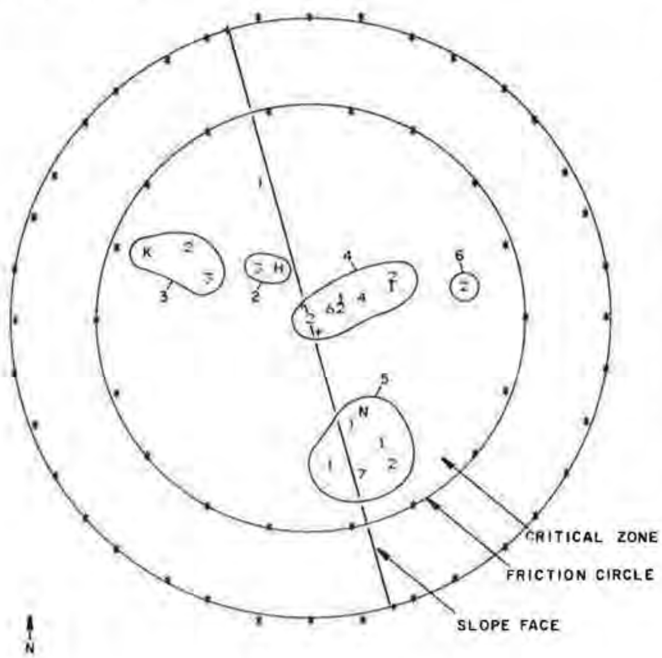


Figure 12. Dip vector plot of discontinuities from Cedar Bluff (data points clustered by eye).



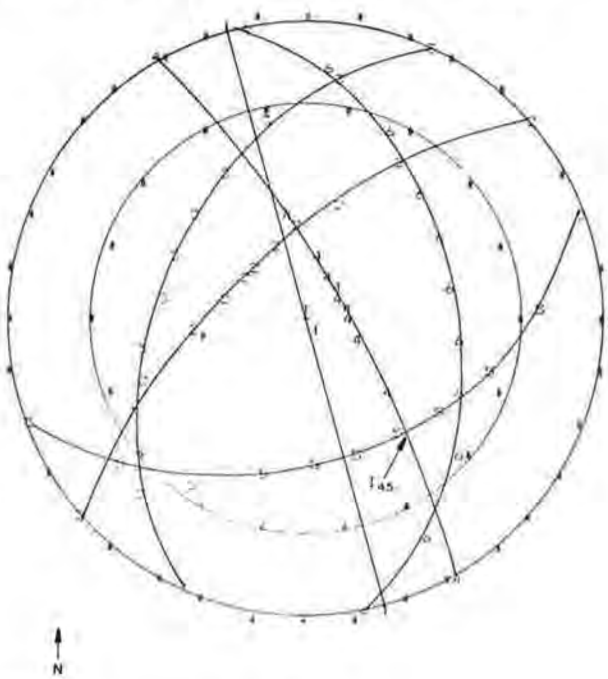
discontinuity number 2. The dip and dip directions of the various planes are listed in the lower left

Figure 13. Dip vector clusters from Figure 12 with friction circle and slope face also plotted.



PHI = 30 SLOPE DIP DIR = 345, DIP = 90
DIP VECTOR PLOT PLUS
PHI CIRCLE AND CUT SLOPE

Figure 14. Computer plot of great circles representing discontinuities clustered in Figure 12 (great circle 1 represents vertical slope face).



PHI = 30
1 DIP DIRECTION = 75 DIP = 90
2 DIP DIRECTION = 320 DIP = 70
3 DIP DIRECTION = 295 DIP = 50
4 DIP DIRECTION = 60 DIP = 80
5 DIP DIRECTION = 160 DIP = 50
6 DIP DIRECTION = 78 DIP = 48

corner of the printout. Plane 1 is usually reserved to represent the slope face. This plot is useful in a number of ways. Potential plane, wedge, and toppling failures can be identified on this plot if they exist in the data. The primary application of this plot involves the identification of the lines of intersection of discontinuities. If a point representing a line of intersection on the stereonet falls within the critical zone, defined once again by the slope face and the friction angle, then that point represents a potential wedge failure and warrants additional study.

At the time the study was conducted, the analysis plot just described was the last of the plots developed. Additional types of preliminary analysis are planned for the system. Among them are programs to locate and plot the points that represent wedge intersections directly and thus eliminate the required hand drawing to locate the intersections in Figure 14. Recently available programs include programs to perform cumulative sums analyses (7) and the data contouring programs previously mentioned. Even before this, however, truly significant time savings were realized by using the initial plotting routines. In this study, 600 individual joints were plotted on a number of different stereonets as both poles and dip vectors, were clustered, and then were checked for kinematically possible failure surfaces, all in a few hours. To accomplish this by hand would require several days, and to accomplish it back at the office on a large computer involves transportation time and time to have the data punched on computer cards or entered at a terminal.

SITE EVALUATION

The preliminary evaluations of the slopes at Cedar Bluff were based on both the equal area net plots of the detailed mapping and on visual observations of significant single features in the slopes. The equal area net plots give a good representation of the distribution and orientation of joints throughout most of the two rock slopes. The structure in the proximity of the St. Clair fault, however, is extremely complex. The bedding planes are contorted into tight folds, and a number of smaller faults have caused displacements of varying distances. These associated faults are not always planar and needed to be mapped individually.

The road cut that actually contains the St. Clair fault serves as a good example. Figures 9-14 represent the detailed lines from this slope. Note that three of the dip vector clusters in Figure 13 lie within the critical zone. Plane 6 is insignificant due to a lack of continuity. Figure 14 shows the plot of discontinuities in the slope. Visual examination of the plot indicates that there is a potential wedge failure at the intersection of planes 4 and 5. Once a potential failure is located, it can be assigned a factor of safety based on any of numerous techniques (4, Appendix 6; 6, Chapter 8). Due to the complexity of the structure in the north end of the road cut, the stereonet plot fails to show one other potential failure associated with the folding and faulting. This illustrates the importance of mapping significant individual features visually whenever possible (Figure 6). There also happen to be a few areas where blast damage has created features best noted by direct observation.

SYSTEM EVALUATION AND RECOMMENDATIONS

Both window mapping and line mapping techniques were used in this study. The purpose was not to determine which technique is best but to determine whether the data collection system functioned better with one rather than the other. No reliable criteria have been determined for deciding a preference between window mapping and line mapping. It remains a matter of site conditions and personal preference. A third site for testing the system was mentioned at the beginning of this paper. The site is located near Clifton Forge, Virginia, in the Iron Gate anticline. A large road cut is planned through the anticline, but currently rock exposures are poor along most of the planned construction. At a site such as this, detailed line mapping is not possible and window mapping appears to be the best alternative.

It was determined during the study that a combination of the window mapping and line mapping techniques is sometimes useful. At the Cedar Bluff sites, much of the two slopes was mapped by setting up 50-ft-long windows, 30-40 ft apart, but mapping within them as if doing detailed lines. This proved to be a satisfactory compromise between the two techniques for this particular location. As expected, the computer-based data collection system worked equally well for either technique, using the same data collection format for both. Some of the data were simply not collected depending on which procedure was being used.

The rapid data collection and evaluation system has been shown to provide a number of time-saving services. It decreases data collection time by about 20-50 percent and provides data plots for preliminary analyses in hours instead of days. It is also capable of transferring the collected data to a larger computer system for additional analysis by use of sophisticated FORTRAN programs.

A few changes could be made to the system to improve its ease of operation. The pocket computer data-collection program could be rewritten to allow a short remark to be recorded after each joint if desired. By using new knowledge, the prototype hardware interface can be rebuilt to provide a less complex form, which makes it simpler to use. In addition, a newer model of microcomputer could be used for the preliminary analyses and stereonet plotting. Microcomputers that are smaller, will connect to any television set, and are less complicated to disassemble and transport are now available in the same price range. Finally, a number of programs are currently being written for the microcomputer system that will handle involved calculations of factors of safety for potential failures once they have been identified. The proposed changes will make this computer-based data collection system simpler to use while making it an even more powerful analytic tool.

REFERENCES

1. T.R. West. Short Course on Selected Geotechnical Design Principles for Practicing Engineering Geologists: Lecture 4--Rock Properties, Rock Mass Properties, and Stability of Rock Slopes. Meeting of Assn. of Engineering Geologists, Chicago, 1979, pp. 4-1 to 4-89.
2. C.F. Watts and T.R. West. A System for Rapid Collection and Evaluation of Geologic-Structure Data for Rock Slope Stability Analysis. Presented at 32nd Annual Highway Geology Symposium, Gatlinburg, TN, 1981.
3. D.R. Piteau. Rock Slope Engineering: Part G--Description of Detail Line Engineering Geology Mapping Method. FHWA, FHWA-TS-79-208, Field Manual, 1980.
4. Golder Associates. Rock Slopes: Design, Excavation, Stabilization. FHWA, 1981, Appendix 6.
5. W.D. Lowry. Nature of Thrusting Along the Allegheny Front near Pearisburg and of Overthrusting in the Blacksburg-Radford Area of Virginia. Department of Geological Sciences, Virginia Polytechnic Institute and State Univ., Blacksburg, Guidebook 8, 1979, pp. 1 and 17.
6. E. Hoek and J. Bray. Rock Slope Engineering, 2nd ed. Institute of Mining and Metallurgy, London, England, 1977, Chapters 3 and 8.
7. D.R. Piteau and L. Russell. Cumulative Sums Technique: A New Approach to Analyzing Joints in Rock. In Stability of Rock Slopes, 13th Symposium on Rock Mechanics, ASCE, Univ. of Illinois, Urbana, Aug. 1971.

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The Malibu Landslide

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A case history of one of the most highly publicized landslides in the history of the California Department of Transportation is presented. In the spring of 1979, the recognition of an incipient and potentially dangerous massive slide above steep sandstone bluffs adjoining the Pacific Coast Highway (CA-1) between Malibu and Santa Monica necessitated closure of this vital transportation link. The installation of an extensive and unique warning system plus a rock fall catchment area permitted partial reopening one month later while the slide investigation proceeded. The correction ultimately selected involved a partial unloading and slope reinforcement. The four-way warning system proved to be extremely useful in permitting the partial reopening of the highway while providing for the general safety of the traveling public and adjacent home owners. The use of rock dowels for slope reinforcement reduced the required excavation to achieve stability by more than 1 million yd^3 in extremely difficult terrain.

What has become known in Southern California as "the Malibu landslide" is located adjacent to the Pacific Coast Highway (CA-1) 1.5 miles west of Topanga Canyon in the central part of the Santa Monica Mountains between Santa Monica and Malibu. This area is characterized by steep natural slopes and rocky escarpments. Landsliding, a common occurrence along this part of the Pacific Coast Highway, is a natural process of bluff recession due to past oversteepening by wave erosion. The bluffs are composed primarily of thick-bedded sandstone with interbeds of siltstone and conglomerate. These sedimentary rocks have been intruded by volcanic rocks that are now highly altered. They have been folded and faulted to create a complex geological structure. Some of the joint patterns of the rock are oriented adversely and thus contribute to the instability of the slope.

Figure 1 is an oblique aerial photograph of the

Figure 1. Oblique aerial view of Malibu slide.

Figure 3. Fissure at head of slide.

Figure 2. Rock slide on Pacific Coast Highway, April 13, 1980.

bluff that shows the active slide area, the Pacific Coast Highway, and homes on its seaward side.

In January and February 1979, California Department of Transportation (Caltrans) maintenance personnel observed a significant increase in the number of rock falls along this stretch of the highway. One extremely large rock (>100 tons) appeared to be precariously balanced on the bluff face and was judged to be a menace, not only to the heavy traffic on the highway (average daily traffic = 30 000 vehicles) but also to the homes on its seaward side. Removal of the rock was accomplished on February 16, 1979, by emergency contract. On April 13, 1979, a rock fall occurred that was of sufficient magnitude to warrant closure of the Pacific Coast Highway (see Figure 2).

A subsequent field review of the area by Caltrans geologists and engineers immediately above the bluff where the closure had been made revealed an extensive system of cracks and fissures extending as far as 350 ft up slope from the bluff face. The generally geometric pattern of the cracks and the 1.5-ft-wide, 3-ft-deep scarp at the head of the slide, shown in Figure 3, indicated the existence of an unstable mass of up to 60 000 yd^3 of soil and rock above the bluff face. The bluff face, except for one area later identified as "the chute", appeared to be relatively stable. An earthquake of Richter magnitude 5, which occurred January 1, 1979, and had an epicenter about 8 miles southwest of the slide, may have been a factor in the initial cracking, although this could not be established definitely.

MONITORING

The first problem posed by the slide was how to open the Pacific Coast Highway, at least partially, while ensuring the safety of those traveling the road and the residents of the homes on the seaward side. The plan that ultimately evolved was the development of a barrier suitable for the catchment of relatively small earth and rock falls and a monitoring system that would provide advance warning of a large mass movement. At any indication of mass movement, the road would be closed and the houses evacuated. The feasibility of such an approach was based on past experience by Caltrans geotechnical personnel, which showed that certain types of incipient slides that are properly instrumented and monitored provide hours to days of warning of sudden mass movement. It was reasoned that development of an adequate catchment zone and monitoring system would permit partial reopening of the highway while exploration leading to the development of a permanent correction could be undertaken.

Three "early warning systems" were installed in the badly fissured area above the bluff between mid-April and early May. A plan view of the monitoring system is shown in Figure 4. Steel pins were driven into the slope at various locations, and measurements were taken between them on a daily basis so that the rate of movement across various segments of the hillside could be determined. Beginning April 18, four acoustic emission [subaudible rock noise (SARN)] stations were established for the purpose of obtaining a relative measure of slide activity. These microacoustic devices consist of highly sensitive geophones capable of sensing rock noise well below human acoustic range. Individual clicks or events per minute at a given site are counted by trained personnel (see Figure 5). Having established a background level of activity for a particular site, an increase in events or counts per minute provides warning of mass movement. Eventually, six acoustic emission stations were established and monitored on a near-continuous basis. On the evening of April 25, three of the six stations then in operation indicated a sudden acceleration in count rate, which prompted closure of the pedestrian walkway along the highway. A plot of counts per minute versus time for one such station (PATH) is shown in Figure 6. Three days later, noise levels dropped to the original background level so that reopening of the walkway was permitted.

Figure 4. Malibu slide instrumentation.

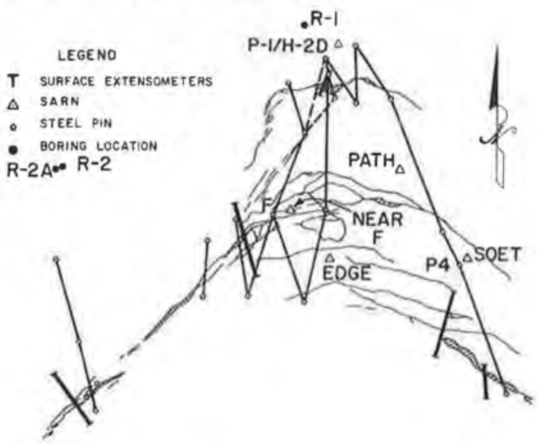
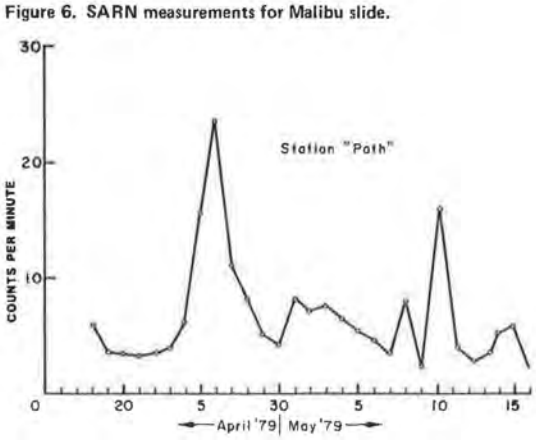


Figure 5. Monitoring SARN.

Figure 6. SARN measurements for Malibu slide.



The two monitoring systems described required the presence of surveyors or geophysicists on a continuing basis. It was deemed extremely hazardous for personnel to work on the unstable and very steep bluff at night for monitoring purposes. With this in mind, a third warning system was installed that would automatically trigger an alarm at roadway level in the event of significant slope movement. These devices, designated as "surface extensometers", were installed across selected zones of high activity based on previous survey measure- ments. They consisted of wire several feet long connected to two steel pins 3-4 ft above the ground surface (see Figure 7). A constant tension was maintained by a spring mounted in a metal housing attached to the system. Movements exceeding 0.15 ft closed an electrical contact, which in turn would trigger a battery-driven alarm system at roadway level. The alarm itself consisted of a flashing yellow light and an air horn (see Figure 8).

A fourth safeguard was the presence, on a 24-h basis, of a maintenance worker who continuously observed the slope, provided warning of rock falls, and could immediately initiate highway closure if so warranted by activity on the unstable slope. The

Figure 7. Surface extensometer installation.

Figure 8. Surface extensometer alarm system.

maintenance worker on duty was instructed to inform geological personnel assigned to the project immediately in the event of an alarm in order that monitoring of the acoustic emission stations could be initiated. In the event that two extensometer alarms were triggered, he was instructed to close the roadway immediately and alert the County for possible evacuation purposes.

During the second week in May, survey points were established on the face of the bluff below the slide mass to verify that the bluff was not moving.

Concurrent with installation of the monitoring systems, a soil-and-rock berm approximately 8-10 ft high was constructed near the toe of the slope. This berm consisted primarily of slide debris that covered the northbound outer lane (see Figure 9). A 10-ft-high chain link fence was installed on top of the berm. The purpose of the berm-fence system was catchment of small earth and rock falls, which would safely permit the opening of the remaining three lanes. Partial opening of the highway occurred on May 6, 1980, after a period of relatively little slide activity. Shortly after the highway was reopened, however, rock and soil falls increased in frequency and magnitude and quickly filled the catchment area. Slide activity precluded the safe removal of debris that collected behind the berm. Consequently, the Pacific Coast Highway was again closed to vehicle traffic on May 8. Construction of an improved catchment system was immediately begun. This consisted of H-beam uprights with heavy timber lagging. Individual H-piles were set in 10-ft-deep predrilled holes and backfilled with lean concrete. The H-pile wall was 12 ft in height from highway profile grade. Secured to the wall was an additional 12 ft of chain link fence for rock catchment (see Figure 10). This system substantially increased the storage capacity for rock and soil fall. On completion of the H-pile timber wall on May 20, three lanes of the highway were again opened. The new system functioned effectively throughout the remainder of the preconstruction phase of the slide.

Figure 9. Soil berm catchment system on Pacific Coast Highway.

Figure 10. H-pile catchment wall and fence on Pacific Coast Highway.

ELEMENTS OF FAILURE

Information obtained from the exploration and monitoring indicated that geological conditions for the slope on which the material was failing differed from conditions exposed in the bluff area. This relation is consistent with geologic mapping by Yerkes and others of the U.S. Geological Survey in 1973. A generalized geologic cross section of the slide is shown in Figure 11.

The beds exposed in the bluff below the failing material have been mapped as Vaqueros Formation and are composed mostly of thick-bedded sandstone with interbeds of siltstone and conglomerate. These beds have been intruded by irregular masses of diabase. Analysis of discontinuities in the rock indicates a stable condition except for the aforementioned "chute area".

The formation within which the failure occurred has been mapped as Topanga sandstone with inter-

Figure 11. Generalized geologic cross section.

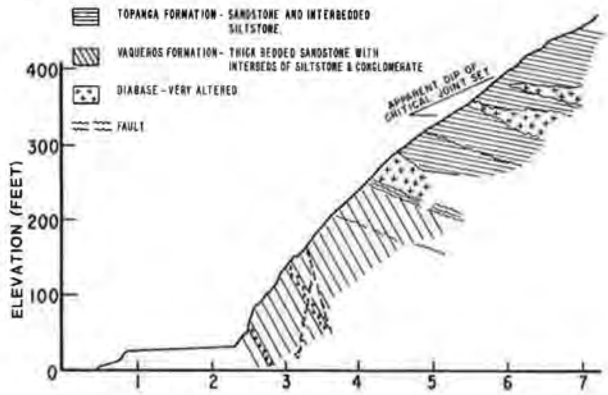
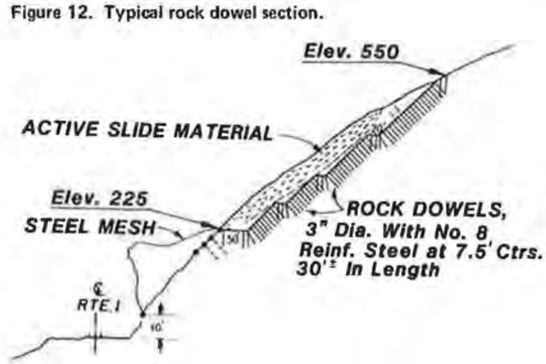


Figure 12. Typical rock dowel section.



bedded siltstone. This formation has also been intruded by diabase that is now intensely altered. Some of the joint patterns exposed in the upper slope are oriented adversely and contribute to the instability. Joint-controlled failure is also suggested by the geometric pattern of the cracks that developed.

Initial cracking of the hillside may have occurred during the January earthquake, but that cannot be proved. The rains of January through March certainly reduced the stability of the hillside.

The pattern of cracking and the types of failure that were occurring suggested that the depth of failed material was shallow. Drilling and seismic refraction indicated a thickness of about 30 ft of disturbed material. The active slide material is shown in section in Figure 12.

LABORATORY TESTING PROGRAM

To better establish the strength characteristics of the materials involved and to provide input for design of a correction, selected samples of weathered sandstone and volcanic rocks common to the site were subjected to a series of multistage triaxial compression and direct shear tests. The purpose of these tests was to determine fracture strength at varying confining pressures and the residual shear strength (strength available after excessive strain and fracture have occurred). Wetting of the weathered sandstone resulted in complete degradation to sand in a relatively short period of time. This demonstrated the need for effective surface drainage for final correction of the slide.

Triaxial test specimens of the sandstone were prepared by diamond coring 2-in-diameter samples from selected boulders. Triaxial tests (unconsolidated undrained) were conducted under an initial confining pressure of 5 psi to determine rupture and thus bond strength of the cementing agent. Samples were sheared in a dry state at a strain rate of approximately 1 percent/min. Shearing continued after rupture until a residual strength value was obtained. Subsequently, confining pressure was increased and shearing continued. This procedure permitted the development of an effective stress Mohr envelope for each specimen. Results of the triaxial test series are summarized in Table 1.

Results of triaxial testing on specimens of the sandstone fragments indicate that the cohesive or binding agents provide a highly competent material prior to fracturing if water is not present. On exposure to water, very slight finger pressure was necessary to crumble the various sample fragments that remained intact after the tests whereas extreme finger pressure was required to break or crumble fragments before the test. The tests revealed high resistance to sliding friction while the specimens were in a dry state. Shear planes that developed after the initial rupture during triaxial testing were irregular or rough compared with the cut or smooth surface for the direct shear test, and thus exhibited ϕ angles in excess of 50°.

Direct shear samples were prepared from the cores by hand cutting two 0.5-in-long samples and assembling the two halves together in a direct shear box for eventual shearing on the cut faces. Tests were conducted on specimens in a dry state under normal loads that varied from 0.125 to 2 tons/ft². Shearing continued until a residual value was reached.

The direct shear tests of specimens in a dry state, which are summarized in Table 2, revealed peak ϕ angles of approximately 40°. The residual ϕ angles for the direct shear tests varied between 24° and 29°. The latter results are not considered representative of normal conditions, however, due to the smoothness of the shearing surface.

Strength characteristics of the specimens in a weathered but dense dry state are presumed to be typical of subrounded to subangular sand and can be expected to have ϕ angles between 40° and 45° and, in a loose state, between 30° and 35°. No cohesion was evident. Based on the results of the above tests, representative angles of internal friction (residual) of from 30° to 33° were assumed to repre-

Table 1. Results of triaxial compression tests on sandstone specimens.

Specimen Group	Test	Compressive Strength (psi): Rupture @ σ_3, 5 psi	Residual @ σ_3: 5 psi	10 psi	15 psi	20 psi	30 psi	40 psi	80 psi	Residual Shear-Strength Parameters: ϕ (°)	C (psf)
A-1	0	1040	140	180	–	80	–	–	–	–	–
B-1	1	2920	92	128	–	200	–	324	–	51[a]	–
C-1	2	2700	180	240	–	–	–	440	606	54[a]	–
C-1	3	2600	206	226	290	366[b]	–	–	–	62[a]	–
Weathered volcanics			37	45	–	56	72	–	–	34	2100

[a]Multistage triaxial compression test; high ϕ angle due to cementation.
[b]Measures 324 with the addition of water.

Table 2. Summary of direct shear tests on sandstone specimens.

Specimen Group	Sliding Shear Stress (ton/ft²): Peak	Residual	Normal Stress on Horizontal Plane (ton/ft²)	ϕ Angle (°): Peak	Residual	Cohesion
A	1.45	1.02	2.0			
	0.73	0.53	1.0			
	0.55	0.30	0.5	39	29	0
	0.24	0.21	0.25			
	0.29	0.10	0.13			
B	1.72	1.23	2.0			
	1.65	0.92	2.0			
	0.93	0.40	1.0	42	24	0
	0.52	0.20	0.5			
	0.27	0.11	0.25			
	0.25	0.05	0.13			

Figure 13. Dowel placement.

sent the controlling shear strength of the design for permanent correction.

CORRECTION DESIGN

Based on the inclination of the apparent critical joint set (Figure 11) and the residual shear strength of the dominant material, it was estimated that the flattening necessary to achieve stability without reinforcement would involve removal of from 1.5 million to 3 million yd^3 of material. Because of the extreme expense of the magnitude of unloading under very difficult conditions, plus obvious aesthetic considerations and lack of nearby disposal sites, the basic concept of a combined unloading reinforcement scheme evolved early in the investigation. It was postulated that an unloading of the active slide material, coupled with the use of rock dowels to "knit" or pin remaining material together, could reduce necessary excavation to approximately 150 000 yd^3, or approximately 10 percent of that necessary without slope reinforcement. Improved stability with dowels was to be achieved in four ways, three of which cannot be quantified but would certainly have a significant effect. These are

1. Development of a shear key;
2. Posttensioning--As the dowels were installed immediately following excavation, it was assumed that adjustment would result in some degree of posttensioning and would, in effect, increase normal load on the assumed potential slide surface and thus increase shear strength;
3. Forcing a deeper failure plane--It was assumed that, after excavation and installation of the dowels, the potential failure surface would be forced through at least partially intact rock blocks with the resulting increase in shear strength; and
4. Improved local stability--It was assumed that the rock dowels would pin or knit individual blocks into position, thus eliminating progressive movement from local failure.

The final design, shown in section in Figure 12, involved unloading to a series of benches 20 ft in width separated by 1:1 slopes. The maximum depth of excavation into the slide mass would be approximately 40 ft, which, based on previous exploration, was judged sufficient to remove the failed material. Rock dowels were to be installed in the configuration shown in Figure 12 on 7.5-ft centers to a depth of 30 ft. Each would consist of a 3-in-diameter hole in which a No. 8 reinforcing bar would be inserted, after which the hole would be backfilled with neat cement grout. In addition, placement of steel mesh over the active portion of the bluff face, also pinned with rock dowels, was planned.

The design involved sufficient rock dowels to provide a minimum safety factor of approximately 1.2 against sliding. The dowels were designed to act as shear pins under the assumption of low active strain, which was believed to be justified as a result of improved local stability; i.e., rock doweling would pin individual blocks into position, thus eliminating progressive movement from local failure.

After careful consideration of a number of alternative corrective schemes, including a shift of the Pacific Coast Highway seaward, the decision was made in June to proceed with the design of the unload-reinforcement scheme. Of primary concern at that time was expeditious commencement of the unloading and reinforcement to ensure removal of the most dangerous portion of the slide mass prior to the onset of the rainy season, which in the Malibu area normally begins in the month of November. In slightly more than a month, Caltrans District 7 project development completed the bidding package, and the project was advertised on July 27, 1979. Bids were opened on August 17. The Novo-Rados Construction Company was low bidder at \$3 013 650 for the bid items. The key items, roadway excavation and rock dowels, were bid at \$8.78/yd^3 and \$10/lineal ft, respectively. The contract was awarded on August 29 and approved on September 12, 1979.

Because the results of laboratory testing indicated that material at the site was extremely susceptible to degradation on application of water, it was considered essential that the four benches developed during the unloading process be given special treatment to prevent degradation resulting from surface runoff.

Based on an evaluation of several possible treatments, the decision was made to use an asphalt membrane to provide a moisture barrier. This material, which has been used for lining reservoirs, pits, and ponds, has been successfully applied to both fabric and compacted native soils. Based on the results of test plots on the native material near the site, the decision was made to apply the membrane without fabric reinforcement in a spray application. Because of the extremely rough surface texture of the benches, an application rate of 0.1-0.2 gal/yd^2 was necessary to provide complete coverage.

On approval of the contract, the existing H-pile catchment system was strengthened and, at key locations, increased in height. Excavation of the slide mass and rock doweling of the underlying material began in early October. Excavated material was pushed down the face of the slope into the catchment area on a work schedule of 12 h/day, 6 days/week. Hauling from the catchment area was performed 10 h/night, 5 days/week. Disposal of most of the debris was accomplished by construction of a buttress fill along another portion of the bluff face adjacent to the Pacific Coast Highway about 5 miles east. Slide removal proceeded at an average rate of 2000 yd^3/day, and maximum production was 5000 yd^3/day. Dowel installation (see Figure 13) was accomplished by using two air-actuated pressure drills on a continuous basis. The construction operation was significantly benefited by the rainfall pattern in the immediate area. In contrast to normal conditions, rainfall was relatively light during the months of November and December, which permitted a period of almost three months of uninterrupted unloading and slope reinforcement. In January and

Figure 14. Completed project.

February, rainfall increased significantly but without adverse effect, probably due to the fact that the most unstable position of the slide mass had been removed by this time.

The project was completed and accepted on July 17, 1980. An oblique aerial view of the completed project is shown in Figure 14.

CONCLUSIONS

The following conclusions can be drawn concerning the effectiveness of techniques used to monitor and correct the Malibu landslide:

1. The slope monitoring system, consisting of survey points, SARN, and surface extensometers placed to provide advance warning of incipient mass movement, proved to be extremely effective. It permitted a partial reopening of the Pacific Coast Highway during the investigation and design phases of the slide correction.
2. The correction method that was ultimately used (slope reinforcement and partial unloading) reduced the excavation quantities required to achieve permanent stability by an estimated million cubic yards or more.

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