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DESIGN AND CONSTRUCTION OF COMPACTED SHALE EMBANKMENTS.

Vol. 1. Survey of Problem Areas and Current Practices

J. H. Shamburger, D. M. Patrick, and R. J. Lutten

August 1975

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16. Abstract The purpose of the three-phase study is to develop design and construction methodologies that will enable shales causing settlements and slope failure in highway embankments in the past to be identified and used successfully in future construction.

During the first year's work (Phase I), state and federal agencies were contacted. Information obtained on the extent and types of problems, possible causes, and problem formations is discussed, and current highway practices are summarized. Physical and chemical weathering of shale placed as rock fill is a primary cause of problems. Nine states do not permit shale to be placed as rock fill, and seven states allow placement as rock fill with special provisions. Corps of Engineers and Bureau of Reclamation experiences indicate that heavy compaction equipment and relatively thin lifts produce well compacted embankments having no problems. Data from 16 projects indicate that saturated compacted shale materials have low shear strengths. A review of shale composition, factors contributing to degradation, and laboratory testing emphasizes the importance of mineralogy and slake-durability characteristics. The natural variability of shales collected from formations in five geologic age groups is described.

This is the first of two volumes. Volume 2, covering Phase II, is published as FHWA-RD-75-62, subtitle: Evaluation and Remedial Treatment of Compacted Shale Embankments.

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PREFACE

This report describes Phase I of a three-phase investigation funded by the Department of Transportation, Federal Highway Administration (FHWA), under Intra-Government Purchase Order No. 4-1-0196. Contract manager was Mr. D. G. Fohs, Materials Division, FHWA.

The work was conducted during the period July 1974-June 1975 by the Soils and Pavements Laboratory (S&PL), U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss. Mr. W. E. Strohm, Jr., was project coordinator. Messrs. Strohm and J. H. Shamburger and Drs. D. M. Patrick and R. J. Lutton prepared the report. The investigation was accomplished under the direct supervision of Mr. D. C. Banks, Chief, Engineering Geology and Rock Mechanics Division, S&PL, and under the general supervision of Mr. J. P. Sale, Chief, S&PL.

Mineralogical examinations and their interpretation for a study of shale variability were provided by Mr. G. S. Wong, Engineering Sciences Division (ESD), Concrete Laboratory, WES. Mr. A. D. Buck, ESD, provided mineralogical examinations and their interpretation for a study of shale samples from an embankment in Clifton Forge, Va.

Personnel of the State highway organizations of California, Colorado, Indiana, Oklahoma, Ohio, Oregon, Kentucky, Missouri, Montana, North Carolina, North Dakota, New York, Pennsylvania, Tennessee, Utah, Virginia, and West Virginia provided valuable information for the study and assistance in obtaining shale samples.

Director of the WES during the conduct of this study and the preparation of the report was COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
Angstroms	0.0000001 (10^{-7})	millimetres
inches	2.54	centimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
square inches	0.00064516	square metres
square feet	0.09290304	square metres
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
grams	0.001	kilograms
pounds (mass)	0.4535924	kilograms
tons (2000 pounds)	907.1847	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (mass) per cubic yard	0.59327631	kilograms per cubic metre
pounds (force)	4.448222	newtons
pounds (force) per square inch	6894.757	pascals
pounds (force) per square foot	4.882428	kilograms per square metre
miles per hour	1.609344	kilometres per hour
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

I. INTRODUCTION

Background

Construction of the modern interstate highway system has required large, high embankments using economically available materials from adjacent cuts or nearby borrow sources. Because of the widespread distribution of shale,* it has been necessary to use this material for embankments in many locales. This practice has led to numerous problems caused by excessive settlements and slope failures of large embankments. While these problems have occurred in a number of States (Chapman and Wood, 1975), the more severe problems have occurred in the East Central States where the climate is humid. Failure repair is often very expensive, amounting to nearly $2 million on one project (DiMillio and Haugen, 1974).

The underlying cause of the problems appears to be detrimental changes in the properties of certain shales with time after construction of the embankment. Some shales have the appearance and properties of sound rock upon excavation but deteriorate during or after placement into weak clay or silt of low shear strength. When shale chunks are placed as rock fill, their deterioration can cause large settlements, blockage of seepage, and eventual slope failure. Other shales, often interbedded with limestone, tend to degrade when excavated. However, when compacted as soil fill, the larger shale and limestone pieces can prevent adequate compaction. These shales can deteriorate further and soften to form a weak clay or silt of low shear strength. The problems associated with shale are complicated by variations in the stratigraphy and lithological characteristics of weak sedimentary rocks, the climate

* The term "shale," whenever used in this report, includes all weak sedimentary rocks such as claystones, siltstones, mudstones, etc. The terms "argillaceous rock" and "mudrock" are also used as a general term for these types of rock. However, the term "shale" is used almost exclusively in highway literature and is therefore used as a general name throughout this report for simplicity except in discussing classifications used by others (Part IV).

and groundwater conditions, and the weather and construction conditions.

All of the States concerned have recognized the problems in using shale in embankments. A number of States have delineated problem shale formations and have developed special design features and specification provisions for use with compacted shale embankments. Several States have also adopted the use of field test strips in selecting compaction requirements for shales. In many States, shale is required to be compacted as soil fill in the absence of proven criteria for classifying shale durability. A recent study at Purdue University (Deo, 1972) for the Indiana State Highway Commission has led to the development of preliminary classification criteria based on laboratory tests. These criteria are currently being used by the State of Indiana.

Although progress is being made in limited areas, the state of the art is still not well defined; there remain a number of unsolved problems. The basic causes responsible for the deterioration of shales in embankments in different locales have not been defined. Suitable criteria and tests, related to the basic causes, are needed to distinguish durable shales from those that will deteriorate with time in the embankment. Information is needed on the distribution and characteristics of problem shales and their probable natural variability and range of properties.

Comprehensive guidance is required on the methodology for field investigation, design, and construction of compacted shale embankments. The required information includes (a) determination of strength and compressibility properties, both for end-of-construction and long-term conditions; (b) field compaction specifications and compaction control techniques; (c) pretreatment techniques and compaction equipment requirements; (d) methods of design and analysis; and (e) selection of appropriate design features.

Of more immediate concern is the need for suitable methods of evaluating existing embankments which are exhibiting signs of distress. Some embankments are settling excessively. In the past, continuing settlement has led to large slope failures in some embankments, while in others the settlement has stopped and no further distress has occurred.

The likelihood of failure, the types of remedial measures needed, and when and where they should be employed are paramount problems. The presence of large pieces of hard rock virtually precludes obtaining suitable undisturbed samples. Improved methods are needed to evaluate in situ density, permeability, shear strength, and compressibility. Guidance is needed on appropriate instrumentation for monitoring deformations, groundwater conditions, and pore water pressures and for determining changes with time.

The repair of embankment failures usually involves removal of all or a part of the slide material, installation of drainage measures, and reconstruction using flatter slopes and/or berms or retaining structures. Criteria and guidance on applicability of these remedial measures and other less expensive methods of stabilization are needed.

The needs outlined above led to the initiation of this study which is being conducted by the U. S. Army Engineer Waterways Experiment Station (WES) over a 4-yr period. The first year was devoted to the immediate needs, which are dealt with in this volume and in a second companion volume.

Basic Objectives

The basic objectives of the research effort are:

a. Identification of factors responsible for the deterioration of compacted shales.

b. Development of techniques to evaluate the stability of existing compacted shale embankments.

c. Development of remedial treatments for existing distressed compacted shale embankments.

d. Development of design criteria and construction control techniques for compacted shale embankments.

Objective a was accomplished under Phase I and concurrently objectives b and c were accomplished under Phase II during the first year (FY 75) to meet the critical needs established by the Federal Highway Administration (FHWA). Objective d will be accomplished under Phase III and covers many of the tasks required to successfully complete the research needs identified by the FHWA. Initiation of Phase III is contingent on the results of Phases I and II and the approval of the FHWA.

Scope of Study

The scope of this study is summarized in Table 1. The study excludes the following:

a. Settlement and stability problems originating in the foundation materials beneath embankments.

b. Stability problems originating in cut slopes.

c. Deterioration of compacted shale arising from frost action.

Approach

Phase I. Pertinent information in the literature was reviewed on classification criteria; location, areal extent, geology, and stratigraphy of various shale formations exposed in the United States; compaction and performance of shale mixtures in embankments; strength and compressibility data; and intrinsic and extrinsic factors associated with problem shales.

Concurrently, State and Federal agencies were contacted and visited to develop current information on the above-listed items and to identify sources of problems encountered in compaction of shale embankments and the current practices for design investigation, construction, maintenance, and remedial treatment of compacted shale embankments. Available information was sought on classification and material properties, physical and chemical tests, sampling and testing procedures for in situ shales and compacted shale mixtures, and construction control procedures and tests.

Information from the literature and field visits was used to establish a basis for identifying physical and/or chemical parameters affecting the deterioration of shales. Parameters considered to be important were mineral associations, particle degradation, clay content and sensitivity, pore water composition, and chemical alterations (e.g.,

Table 1. Scope of research study and schedule.

PHASE I - IDENTIFICATION OF FACTORS RESPONSIBLE FOR THE DETERIORATION OF COMPACTED SHALES (FY 1975, Vol. 1)

Task A

1. Review existing literature and reports pertinent to compaction and performance of compacted shale mixtures in embankments.

2. Contact Federal and State agencies concerned with highway construction in areas where problems have been encountered for the purpose of identifying the sources of the problem and discussing, as well as describing, the state-of-the-art design, construction, maintenance, and remedial treatment of compacted shale embankments.

Task B

1. Identify the geologic, stratigraphic units and the specific geographic localities of shales that have caused problems in compacted embankments.

2. Accomplish preliminary identification and validation of the intrinsic and extrinsic factors and combinations of these factors causing the problems.

Task C

Perform field and laboratory study to determine the probable natural variability of intrinsic properties of stratigraphic units of shales of different geologic ages that have caused problems to embankments.

PHASE II - STABILITY EVALUATION AND REMEDIAL TREATMENT OF EXISTING EMBANKMENTS (FY 1975, Vol. 2)

Task A

Review and evaluate available experience and recommend appropriate methods for evaluating the stability of existing compacted shale embankments.

Task B

Review and evaluate available experience and recommend appropriate methods for remedial treatments of existing embankments.

PHASE III - DEVELOPMENT OF DESIGN CRITERIA AND CONSTRUCTION CONTROL TECHNIQUES (FY 1976-1978)

Task A

Develop, evaluate, and recommend the appropriate shale sampling program for obtaining embankment design data and preparation of compaction specifications.

Task B

Develop new index tests or improve existing tests and evaluate them as techniques for obtaining compaction specifications for shale mixtures.

PHASE III (continued)

Task C

Develop, evaluate, and recommend an appropriate laboratory specimen preparation methodology, a laboratory compaction methodology, and a laboratory testing technique for determination of shear strength and compressibility properties of compacted shale mixtures for the end-of-construction condition of the compacted shale mixture. These are to be used when:

1. The grain-size distribution of the laboratory specimen appproximates that of the compacted shale mixture in the embankment.

2. The particles in the laboratory specimen are substantially smaller than particles in the compacted shale mixture in the embankment.

Task D

1. Develop a methodology for evaluation of the end-of-construction shear strength and compressibility of the shale mixtures compacted in test strips.

2. Conduct field tests to evaluate methodology developed in D-1 above.

Task E

Develop, evaluate, and recommend tests to quantitatively evaluate the long-term strength and compressibility properties for compacted shale mixtures for specimens prepared and compacted either in the laboratory or in a field test strip.

Task F

Develop, evaluate, and recommend a methodology to use for extrapolating the laboratory preparation and compaction techniques to field compaction specifications for shale mixtures and also a field compaction control methodology. These are to be used when:

1. The grain-size distribution of the laboratory specimen approximates that of the compacted shale mixture in the embankment.

2. The particles in the laboratory specimen are substantially smaller than particles in the compacted shale mixture in the embankment.

Task G

Evaluate and make recommendations concerning the effectiveness of different kinds of pretreatment techniques and compaction equipment for compaction of different types of shales.

Task H

Review, evaluate, and condense results of other tasks to provide detailed guidance and recommended methodology for the following:

1. Determining the design strength and compressibility parameters from test data.

2. Performing the stability analysis.

3. Selecting design features.

change of pyrite to iron oxides and combination of sulfur into sulfates with volume change, swelling, and weakening of the shale).

Selected unweathered samples of problem and nonproblem shales collected during field trips and by courtesy of several States were examined as part of the variability study to aid in the development of meaningful parameters and to evaluate selected laboratory tests for determining deterioration susceptibility. Examinations included tests for natural water content, mineralogy (determininination of gross mineral constituents), hardness, and slaking durability.

In studying the probable natural variability of intrinsic properties, unweathered samples were obtained from a range of stratigraphic units in formations covering five geological ages located in Indiana, Ohio, Kentucky, Tennessee, and Virginia. The variability of characteristics was studied on the basis of laboratory deterioration tests, and the results were compared with embankment performance in the areas sampled.

Data on natural water content, degree of weathering, and effects of hydrologic cycles and climatic zones as well as construction practices were considered in an attempt to identify extrinsic factors contributing to compacted shale embankment problems.

Phases II and III. The approach for Phase II and Phase III is described in Appendix A. The results of Phase II are presented in Volume 2.

Scope of Report

This volume presents the results of work accomplished under Phase I. An important feature of the overall study, the collection of information from literature searches and contacts with State and Federal agencies, is briefly described first. An overview of the occurrence of shale covering the 16 States included in the study and several other adjacent States is presented on five sectional maps and is followed by a summary of shale formations considered as a problem by 12 States. The occurrence of these problem shales is shown on individual State maps. Information obtained on the types of problems with highway shale embankments and possible causes is discussed, and current highway practices are summarized.

Corps of Engineers experience with compacted shales used in earth and rock-fill dams applicable to highway embankments is described next and includes data on material properties and shear strength. A brief review of experience by the U. S. Bureau of Reclamation (USBR) is also given.

A review of shale classification and composition, factors contributing to degradation, and laboratory examination and testing are then reviewed. This review is followed by the results of the study of natural variability of shales collected from locations covering five geological age groups. The results of laboratory tests to determine the degree of susceptibility to deterioration using several different test procedures are presented, and the variability of the formations studied is discussed.

II. DATA COLLECTION

Task A of Phase I consisted principally of collecting and reviewing literature and unpublished reports pertinent to the construction and performance of compacted shale embankments and contacting Federal and State agencies concerned with highway construction that have had experience in this area. The purposes were to determine the occurrence of problems and possible causes and to identify current procedures for design investigation, construction, maintenance, and remedial treatment of compacted shale embankments.

The initial effort of identifying the types of information and data to be collected was accomplished through a joint effort of the task leaders in Phases I and II and other personnel associated with the project. This effort resulted in the checklist of desired information presented in Appendix B. The literature search was not restricted to any geographical areas; however, the contacts with Federal and State agencies were generally restricted to those physically located in the 16 selected States shown in Figure 1 or having responsibilities within these States.

Literature Search

The literature search was accomplished by examining the resources at the Technical Information Center (TIC) at WES and by obtaining professional searches from Government agencies, private organizations, and educational institutions with retrieval capabilities from data banks. Key words used for the search were selected principally from the publication, "Subject Headings for Engineering."

TIC. The subject card index was examined and about 150 references were located. Volumes for 1964-1973 of the Engineering Index were searched and 45 entries were identified for examination. The annual publication, "Applied Science and Technology Index," was also reviewed for pertinent publications. Volumes for 1964-1973 were examined and 15 articles were selected for review.

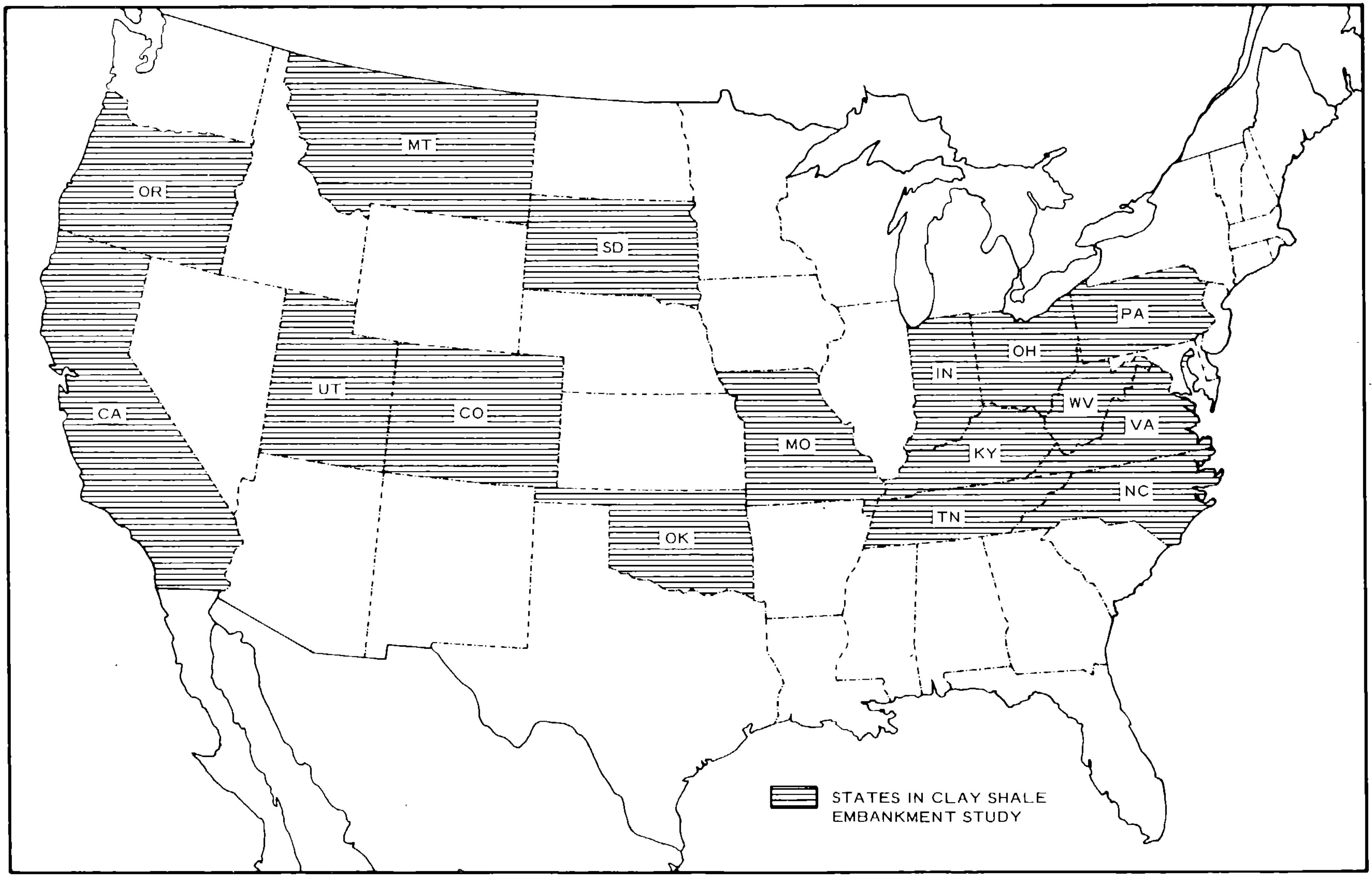


Figure 1. States selected for the study.

Professional searches. Professional searches were made through five organizations: (a) Highway Research Information Service (HRIS), (b) National Technical Information Service (NTIS), (c) American Geological Institute (ACI), (d) Defense Documentation Center (DDC), and (e) Information Science Computer Center (ISCC). The search by HRIS listed 85 reports and 15 were identified for review. A printout of 100 articles with abstracts was received from NTIS of which 20 were indexed. The AGI data search is entitled GEOREF and 396 references were reviewed from this source of which only 35 were applicable to this study. The DDC search resulted in a listing of 15 references of which only 2 were selected for review. The ISCC, located at the University of Georgia, makes searches by calendar year. A search was requested for 1 year (1972) to check the potential of this source. References identified by this trial search were not applicable to the study and no additional searches were requested.

A list of publications from the U. S. Geological Survey (USGS) and the State geological surveys of the 16 States was obtained and reviewed. Because of the data content in these publications (such as mineral resources, groundwater, detailed geological quadrangle maps, etc.), only a limited number were reviewed.

Personal Contacts

The initial contacts were made with FHWA regional offices by telephone to obtain recommendations for contacts with the State highway departments. The regional offices recommended personnel for direct contact within the State highway departments and in some cases requested that their offices be notified after scheduled visits to the States had been finalized. Personal contacts with State highway department representatives of Indiana, Ohio, Tennessee, and Kentucky were made during the Sixth Annual Southeastern Highway Soil Engineering Conference in Covington, Ky, on 16-19 September 1974. The highway departments for the other selected States (Figure 1) were visited during August-October 1974, except for North Carolina. The highway department representatives for

North Carolina stated that only a small amount of shale had been encountered during highway construction and compacted shale embankments were not a problem. At the recommendation of FHWA Region 1 Office, the New York State Department of Transportation was visited because New York had constructed numerous shale embankments with no apparent problems.

Copies of the checklist and the statement of work for the overall study were forwarded to each of the highway departments prior to the visits. Meetings with highway personnel were informal, and complete cooperation was received from all the highway departments. The number of highway department personnel attending the meetings varied from two to eight. The time spent at each highway department varied from 1/2 to 1-1/2 days depending on the complexity of the problems with compacted shale embankments and the extent of available information. Field trips were made during several of the visits. Data collected during the visits and through subsequent telephone calls and letters included standard specifications and special provisions, manuals, design standards, and reports on specific projects in addition to the information from personal discussions. A memorandum for record was prepared after each visit.

Other organizations contacted (see Table 2) were questioned as to their experience with design and construction practices using shale as embankment material. The USGS and State geological surveys are not engaged in construction, but were contacted to determine their knowledge of shale characteristics and types of information available. Purdue University was visited to obtain current information on compacted shale embankment research being conducted under a Joint Highway Research Project for the Indiana State Highway Commission.

Data Review

All referenced material located, either through searches or personal contacts, was given a cursory examination and identified as "useful" or "not useful" to the study. The categorizing of the material was accomplished by reading an abstract or by reviewing the actual material.

Table 2. Federal agencies, educational institutions, and commercial firms (exclusive of FHWA).

U. S. Army Corps of Engineers

U. S. Army Engineer Division, South Pacific
U. S. Army Engineer District, Sacramento

U. S. Army Engineer Division, North Pacific
U. S. Army Engineer District, Portland

U. S. Army Engineer Division, Ohio River
U. S. Army Engineer District, Huntington
U. S. Army Engineer District, Louisville
U. S. Army Engineer District, Pittsburgh

U. S. Army Engineer Division, Southwestern
U. S. Army Engineer District, Fort Worth
U. S. Army Engineer District, Little Rock
U. S. Army Engineer District, Tulsa

U. S. Army Engineer Division, South Atlantic
U. S. Army Engineer District, Mobile

U. S. Army Engineer District, Baltimore

U. S. Army Engineer Division, Missouri River

Department of the Interior

U. S. Geological Survey, Denver, Colo.
U. S. Geological Survey, Menlo Park, Calif.
U. S. Geological Survey, Reston, Va.

U. S. Bureau of Reclamation, Denver, Colo.

U. S. Bureau of Mines, Spokane, Wash.

Tennessee Valley Authority, Knoxville, Tenn.

State Geological Survey Offices

Oregon State Department of Geology and Mineral Industries, Portland, Oreg.
Montana Bureau of Mines and Geology, Butte, Mont.
Ohio Division of Geological Survey, Columbus, Ohio
Kentucky Geological Survey, Lexington, Ky.

Universities

Purdue University, West Lafayette, Ind.
University of California, Berkeley, Calif.
University of California, Davis, Calif.
University of Illinois, Urbana, Ill.
University of Missouri, Rolla, Mo.
University of Oklahoma, Norman, Ok.

Law Engineering Testing Company, Birmingham, Ala.

The referenced materials reviewed included manuals, specifications, design standards, technical articles, research reports, and conference papers. Although most of these were published, many unpublished papers and reports were collected. Some of the more pertinent geological reports were also obtained.

Over 850 references were reviewed in this task, and annotations were prepared on those determined useful to the study. Bibliographic cards were made for all materials received. These cards were cataloged by key word or subject and cross-indexed by State and author. These materials were available to all personnel participating in the various study phases and tasks and are listed in the Bibliography.

III. SHALE EMBANKMENTS

Shale Occurrence

An important feature of this study was to determine the general occurrence of shales in the States being studied in relation to the distribution of problem shales. The geological maps available for the States were variable in scale, level of detail, and coverage. Since the most consistent source of the geology for each State was the State geologic maps, they were selected as the principal source for compiling a shale occurrence map. A scale of 1:3,000,000 was selected as the mapping base. This scale was considered large enough to give a convenient overview of general shale occurrence, although it required rather gross generalizations.

Each State geologic map portrays formations, groups, and/or series which are described in a legend on the map. Geologic rock-stratigraphic units can vary in composition and may include multisoil and multirock types, and this factor complicated or prohibited isolation of shale units. Therefore, three mapping units were established for identifying the occurrence of shale. These units are (a) shale predominates, (b) shale subordinates, and (c) nonshale areas. The term "predominates" means that more than 50 percent of the formation consists of shale, and the term "subordinates" means that less than 50 percent of the formation consists of shale. The term "nonshale" means that shale was absent or that the areal extent of the shale was too small to delineate at the scale used for mapping. The selection of a formation for inclusion in one of the above mapping units was based on the formation description in the legend of the State geologic map. No detailed research was conducted to categorize a formation. The formation descriptions varied between State geologic maps, and in some instances a description on the State geological map was inadequate to place a formation into a unit. Where an inadequate description occurred, other data sources were consulted, primarily the "Lexicon of Geologic Names of the United States," published by the USGS (Keroher, 1966 and 1970). Each State was mapped individually; when transferred to the base map, some inconsistencies between mapping units along the State boundaries were noted. These inconsistencies were resolved using the best information available.

Five sectional maps present the general occurrence of shale within the States studied: Oregon and California (Figure 2); Montana, Wyoming, and South Dakota (Figure 3); Utah and Colorado (Figure 4); Missouri, Oklahoma, and Arkansas (Figure 5); Indiana, Ohio, Pennsylvania, Kentucky, West Virginia, Virginia, Tennessee, and North Carolina (Figure 6). Arkansas, and Wyoming were included because they were alternate States designated by the FHWA for possible inclusion in the overall study. Where details about a specific area are needed, a more comprehensive map should be consulted. For page-size presentation, the maps were reduced in scale from 1:3,000,000 to approximately 1:5,000,000 for Figure 2 and to 1:7,000,000 for Figures 3-6. These maps present a generalized picture of the presence of shale.

Problem Shales

Shales used for embankments during highway construction have presented problems in most of the 16 States selected for study. Three of these States (North Carolina, Oregon, and Pennsylvania) have not experienced problems with compacted shale embankments that are attributable to the embankments themselves. In Oregon, problems have involved shale foundations. The variable extent and complexity of the problems encountered in the other States are described in the section on types of problems (page 43). In general, the States east of the Mississippi River have had more problems with shales in embankments than those west of the Mississippi River. The rock-stratigraphic units associated with problem shales were identified by representatives of the States and ranged from one formation or group (Utah, California, and Colorado) to ten different formations (Oklahoma). These units are listed in Table 3*

* Several other formations, identified as containing problem shales by State highway personnel contacted during field trips for the variability study, are not included here but are included in Part VIII. These additional formations were not identified by the principal State highway representatives and it has not been possible to determine the severity of problems associated with these additional shales.

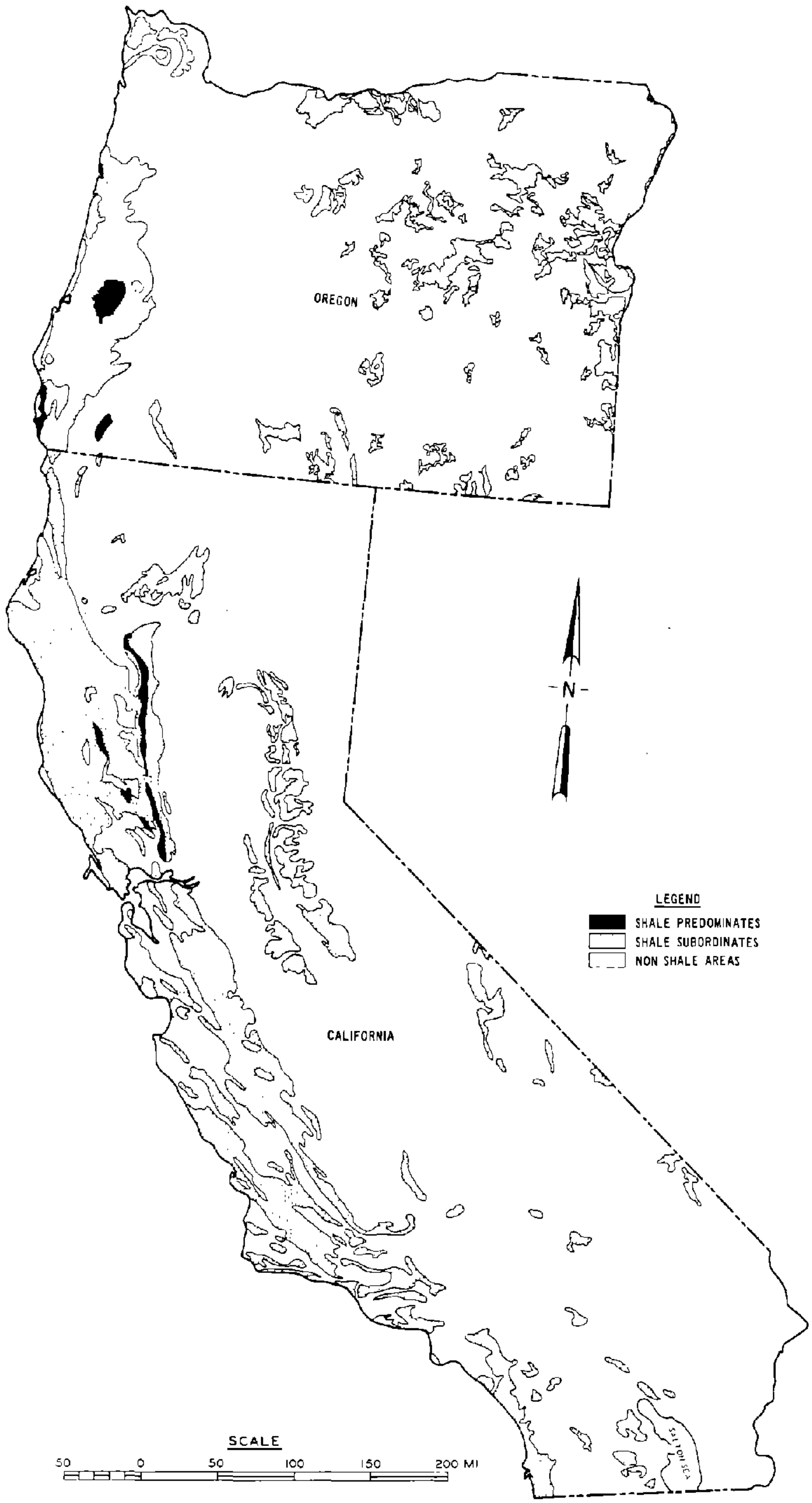


Figure 2. Shale occurrence; Oregon and California.

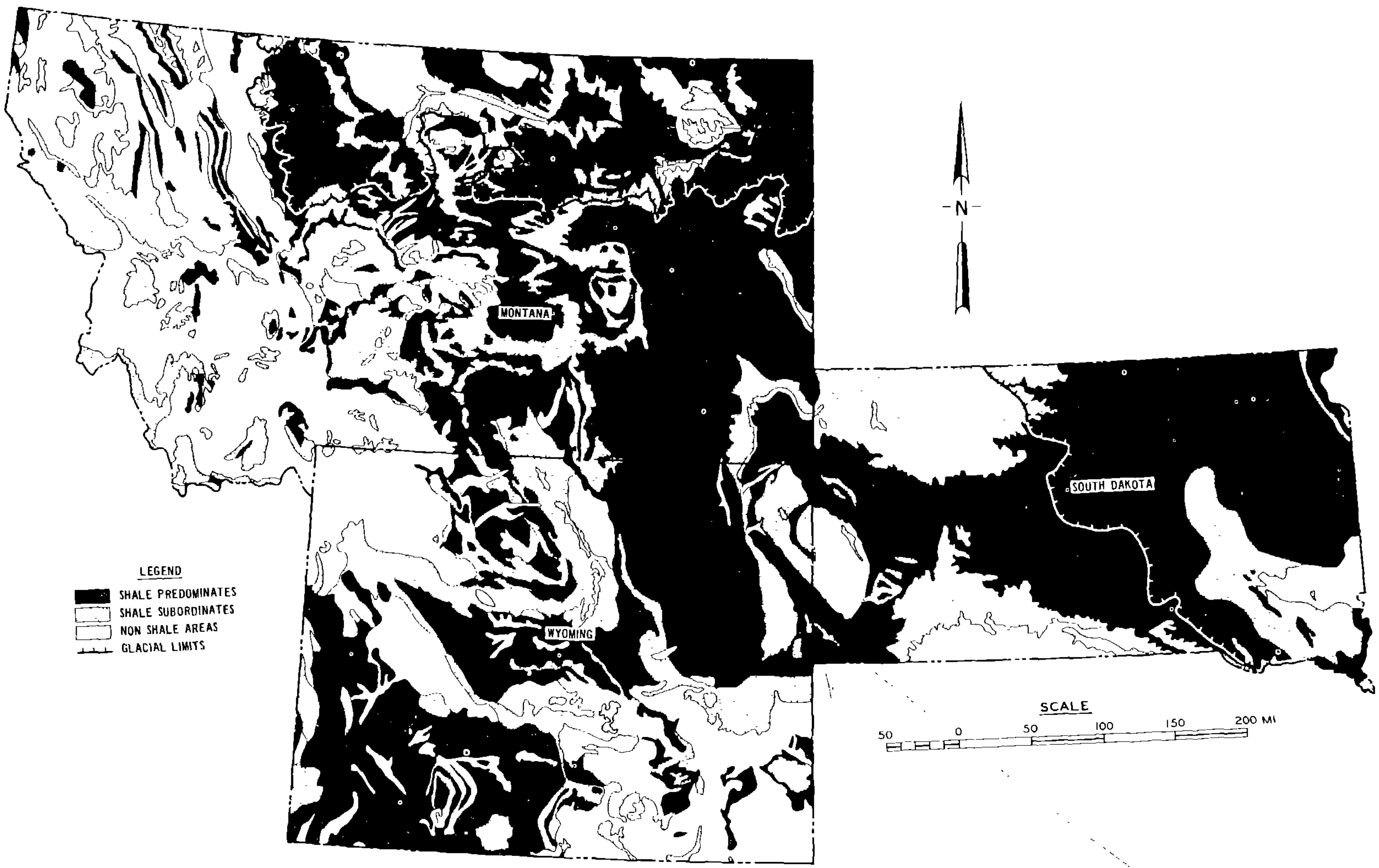


Figure 3. Shale occurrence; Montana, Wyoming, and South Dakota.

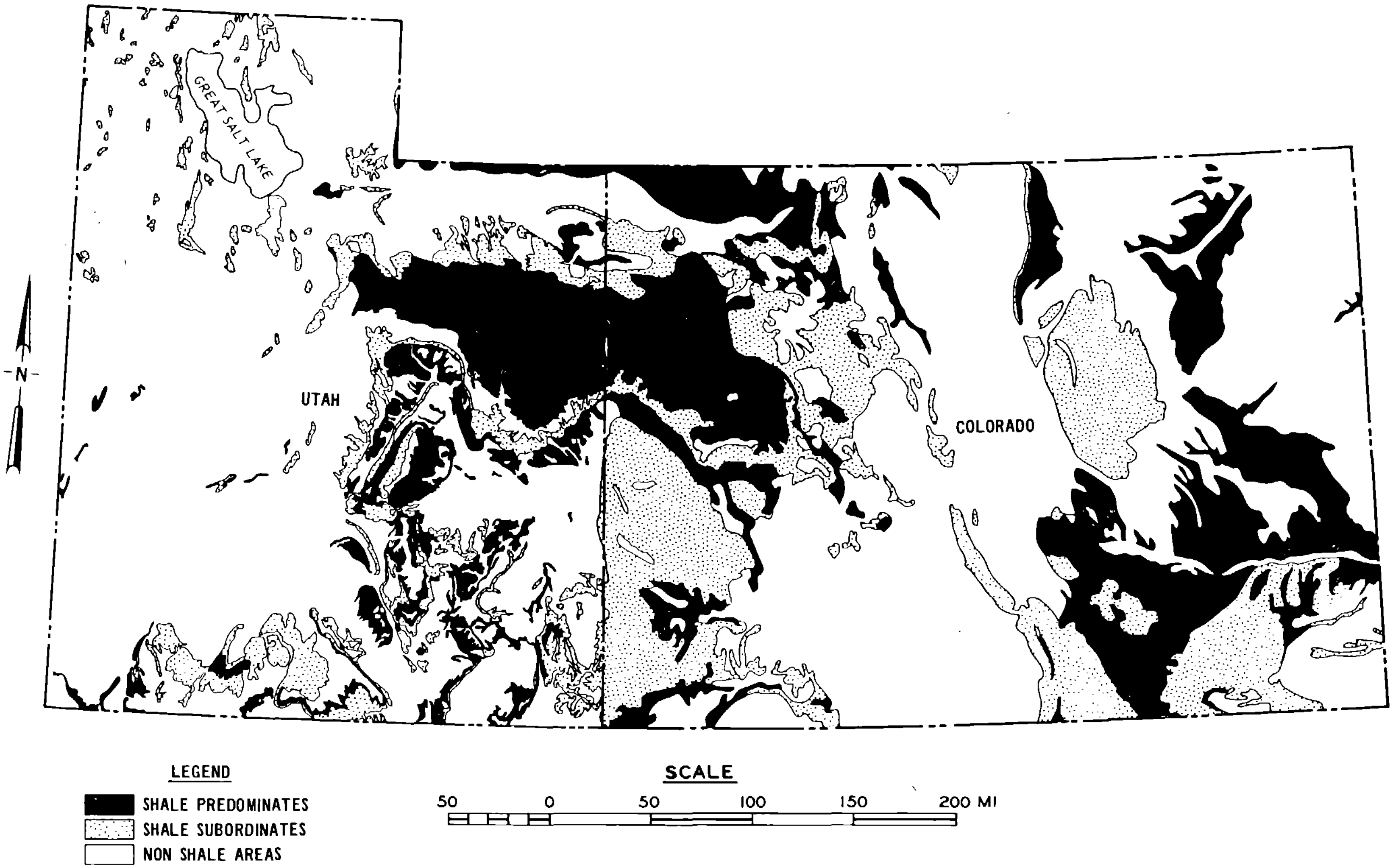


Figure 4. Shale occurrence; Utah and Colorado.

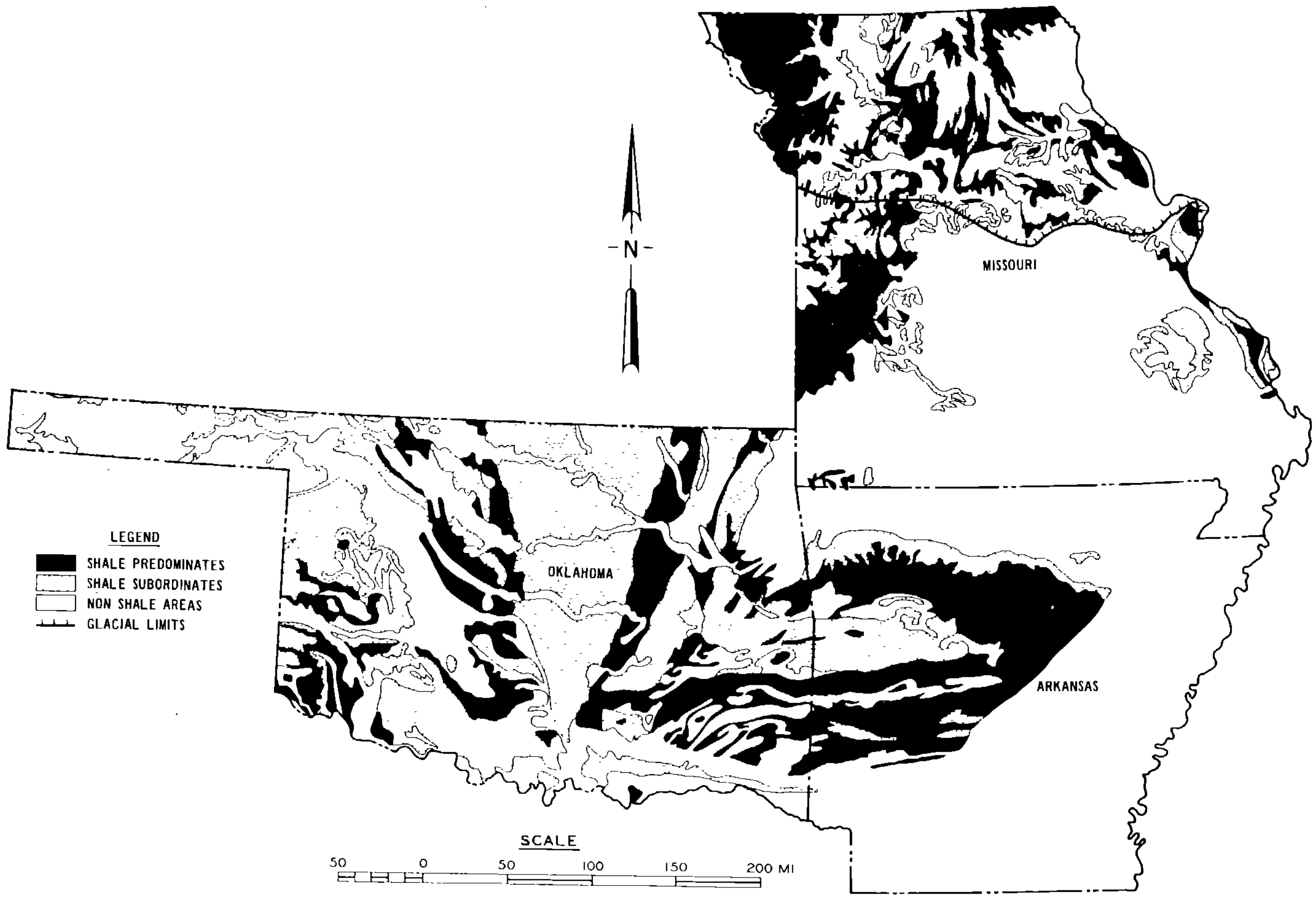


Figure 5. Shale occurrence; Missouri, Okahoma, and Arkansas.

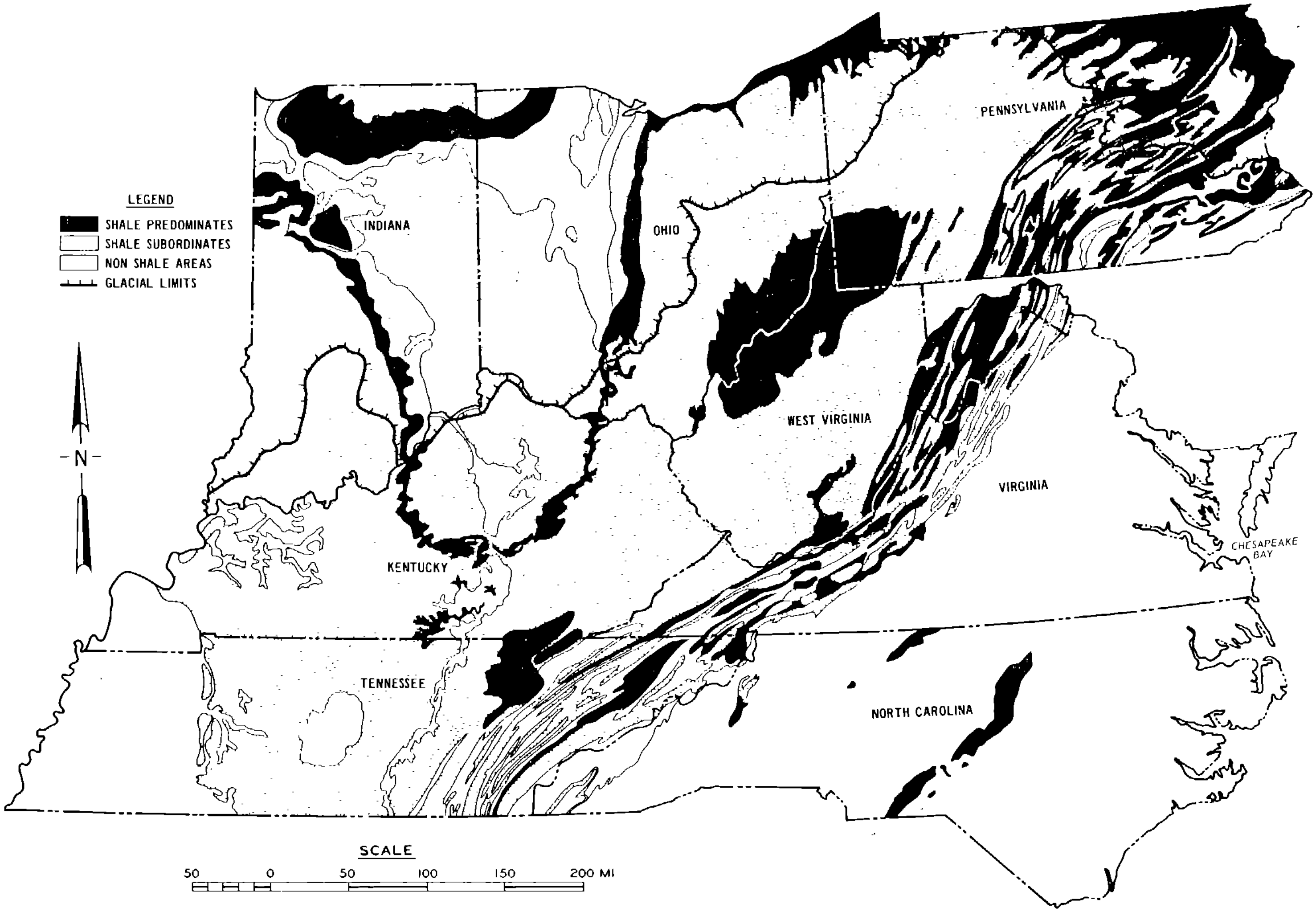


Figure 6. Shale occurrence; Indiana, Ohio, Pennsylvania, Kentucky, West Virginia, Virginia, Tennessee, and North Carolina.

Table 3. Rock-stratigraphic and geologic time units of problem shales.*

California

Rock-Stratigraphic Unit	Geologic Time Unit
Franciscan group	Jurassic and Cretaceous

Colorado

Rock-Stratigraphic Unit	Geologic Time Unit
Benton shale	Upper Cretaceous

Indiana

Rock-Stratigraphic Unit	Geologic Time Unit
Bethel formation	Upper Mississippian
Osage group	Lower Mississippian
Osgood formation	Silurian
Dillsboro formation	Upper Ordovician
Kope formation	Upper Ordovician

Kentucky

Rock-Stratigraphic Unit	Geologic Time Unit
Tradewater formation	Middle Pennsylvanian
Crab Orchard shale	Middle and Lower Silurian
Osgood formation	Middle Silurian
Bull Fork formation	Upper Ordovician
Fairview formation	Upper Ordovician
Kope formation	Upper Ordovician
Eden group	Upper Ordovician

Missouri

Rock-Stratigraphic Unit	Geologic Time Unit
**Cherokee group	Pennsylvanian
**Marmaton group	Pennsylvanian
**Pleasanton group	Pennsylvanian
Northview formation	Mississippian
Maquoketa formation	Upper Ordovician

Montana

Rock-Stratigraphic Unit	Geologic Time Unit
Fort Union formation	Tertiary
Bearpaw shale	Upper Cretaceous
Pierre shale	Upper Cretaceous
Colorado shale	Upper Cretaceous

Ohio

Rock-Stratigraphic Unit	Geologic Time Unit
Massillon member, Pottsville formation	Pennsylvanian
Vinton member, Logan formation	Mississippian
Bedford formation	Mississippian
Richmond group	Upper Ordovician
Maysville group	Upper Ordovician

Oklahoma

Rock-Stratigraphic Unit	Geologic Time Unit
Washita group	Cretaceous
Claypool formation	Permian
Vamoosa formation	Pennsylvanian
Wewoka formation	Pennsylvanian
Nowata shale	Pennsylvanian
Labette shale	Pennsylvanian
Senora formation	Pennsylvanian
Savanna formation	Pennsylvanian
McAlester formation	Pennsylvanian
Johns Valley shale	Pennsylvanian

South Dakota

Rock-Stratigraphic Unit	Geologic Time Unit
Pierre shale	Upper Cretaceous
Carlile shale	Upper Cretaceous
Belle Fourche shale	Upper Cretaceous
Mowry shale	Upper Cretaceous

Tennessee

Rock-Stratigraphic Unit	Geologic Time Unit
Pennington formation	Upper Mississippian
Chattanooga shale	Upper Devonian and Mississippian
Nolichucky shale	Upper Cambrian
Rogersville formation	Middle Cambrian
Rome formation	Lower Cambrian
Murray shale	Lower Cambrian

Utah

Rock-Stratigraphic Unit	Geologic Time Unit
Mancos shale, undivided	Upper Cretaceous

Virginia

Rock-Stratigraphic Unit	Geologic Time Unit
Millboro shale	Upper and Middle Devonian
Martinsburg formation	Upper and Middle Ordovician
Rome formation	Lower Cambrian

West Virginia

Rock-Stratigraphic Unit	Geologic Time Unit
Dunkard group	Pennsylvanian or Permian
Monongahela group	Pennsylvanian
Hinton formation	Upper Mississippian

* Not an all inclusive list.
** Troublesome shales.

and a generalized description of important geological features is discussed in the following paragraphs.

Geologic descriptions. Problem shales vary in geologic age, physical and sedimentary characteristics, chemical composition, and types of interbedded sedimentary rocks. Even these characteristics within a given formation can change within relatively short distances as indicated by the variability study in Part VII. The rock-stratigraphic units listed in Table 3 as problem shales are Cretaceous or older except for the Fort Union (Paleocene) in Montana. A geological time chart is presented in Table 4. The formations in California, Colorado, Montana, South Dakota, and Utah are all Cretaceous in age except for the Fort Union in Montana and the Franciscan in California (Jurassic and Cretaceous). Shales in the remaining States are Pennsylvanian or older except for shales in the Washita group in Oklahoma which are Cretaceous. Not only can the age of the formations be grouped geographically, but the formational characteristics can be roughly separated in a similar manner. The Cretaceous shales of the western United States are distinctively different from the older formations of the eastern United States. Bentonite or bentonitic layers occur in a high majority of shales in the west. Other distinctions of the western shales are their swelling characteristics which were not identified by the highway departments as existing in formations in the east central States.

Because of the generalized descriptions of the formations on the State geological maps, all the formational characteristics at a specific location encountered during highway construction may not be described. Formations can change vertically and laterally which may result in different shale characteristics or rock types encountered at specific locations where a highway crosses a formation.

The fine-grained rocks in problem formations are mostly identified as shale; only a relatively few are identified as claystone, siltstone, or mudstone. Shales are described as carbonaceous, calcareous, siliceous, or micaceous although many are described by other terms such as clay, sandy, fissile, and color. Other sedimentary rocks, principally sandstone and limestone, are associated with the problem formations.

Table 4. Geological time chart.

Era		Period	Approximate Total Age, years
Cenozoic	Quaternary	Recent	1,800,000
		Pleistocene	
	Tertiary	Pliocene	65,000,000
		Miocene	
		Oligocene	
		Eocene	
		Paleocene	
Mesozoic		Cretaceous	136,000,000
		Jurassic	190,000,000
		Triassic	225,000,000
Paleozoic		Permian	280,000,000
	Carboniferous	Pennsylvanian	345,000,000
		Mississippian	
		Devonian	400,000,000
		Silurian	430,000,000
		Ordovician	500,000,000
		Cambrian	570,000,000

Because of the variation in characteristics between problem formations, no specific set of shale characteristics is common to embankment problems. A limited review of selected literature was made to obtain descriptions of the problem formations; in some cases highway department representatives furnished some of the descriptions. The resulting general descriptions are presented in Table 5.

Occurrence of Problem Shales

The occurrence of problem shale formations in the 13 States was identified from the State geological maps and are presented by States in Figures 7-19. No problem shales were indicated for Pennsylvania, North Carolina, or Oregon. The Missouri Highway Department personnel identified the formations or groups in their State as "troublesome" rather than problem shales. Some of the outlined areas on the maps include a problem shale formation and a nonproblem shale formation because some of the State geologic maps did not separate these formations. In these cases, although the entire area is shown as a single formation or group (which actually includes two or more formations), embankment problems have not necessarily been encountered throughout the outlined area. However, when a highway is to be constructed within a problem formation or group, potential problems should be anticipated and considered in design investigations.

During the survey for this study, it was particularly difficult to obtain information on specific locations of troublesome shale embankments. Several highway department representatives indicated that this information is not generally known, primarily because such information is not reported to the central State office. For example, settlement could occur and be taken care of routinely by the district maintenance crews. However, any problem that could not be handled through routine maintenance was reported to the material or soils engineers.

As seen in the maps, the formations identified may cover a sizeable portion of a State or be so narrow in width that they can be shown only by a single line on the small-scale map. In a few instances a formation

Table 5. General description of problem shales.

State	Rock-Stratigraphic and Geologic Time Units	Description
CA	Franciscan group (Jur and Cret)	Arkose, sandstone, dark sandy shale, dark clay shales, conglomerates, red, gray, and black limestone, foraminiferal limestone, chert, basalts, andesites, dacites, volcanic ash, breccias and agglomerates; thickness up to 25,000 ft
CO	Benton shale (Upper Cret)	Gray to black argillaceous fissile shales with thin layer or lenses of limestone and layers of sandstone; streaks of gray to white bentonite occur; disseminated crystals of iron pyrite, a little gypsum and native sulphur are found; thickness is 410 to 460 ft
IN	Bethel formation (Upper Miss)	Dark gray shales and argillaceous sandstone; thickness ranges from 5 to 30 ft
	Dillsboro formation (Upper Ord)	Alternating layers of argillaceous limestone and calcareous shale; shale is highly fissile and bed thickness varies between 1 in. and 2 ft; thickness up to 350 ft
	Kope formation (Upper Ord)	Alternating beds of dark greenish-gray shale and light greenish-gray fossiliferous limestone; formation is about one-third limestone and two-thirds shale; average thickness of shale is 8 in. and limestone is 2 in.; composite thickness is approximately 225 ft
	Osage group (Lower Miss)	Blue argillaceous shale and massive or shaly siltstone to fine-grained sandstone with alternating shaly zones; average thickness is 125 ft
	Osgood formation (Sil)	Dense, tan, argillaceous limestone and light gray calcareous shale; thickness is 13 to 23 ft
KY	Bull Fork formation (Upper Ord)	Alternating layers of shale and limestone; shale content increases from about 20 percent near base to 80 percent near top; shale is gray to greenish-gray, calcareous, and locally fossiliferous; thickness is 200 ft in typical area
	Crab Orchard shale (Lower & Middle Sil)	Yellow-green to grayish olive, thinly bedded shale and light gray to greenish-gray dolomite; ratio of shale to dolomite is variable; thickness is from 50 to 90 ft
	Kope formation (Upper Ord)	Thin interbedded gray to dark gray shales and gray limestones; shale constitutes about 80 percent of the formation and is soft to medium hard; composite thickness is 225 ft
	Fairview formation (Upper Ord)	Interbedded gray calcareous shale and light gray limestone; shale content is about 60 to 65 percent; thickness is 100 to 150 ft
	Osgood formation (Middle Sil)	Blue or greenish-gray shale and fissile claystone with thin interbeds of dolomite near top and base; thickness up to 40 ft
	Tradewater formation (Penn)	Interlayered sandstone, shale, siltstone, and minor coal seams; thickness is 175 to 700 ft

(continued)

(Sheet 1 of 4)

Table 5. General description of problem shales. (continued)

State	Rock-Stratigraphic and Geologic Time Units	Description
MO	Cherokee group (Penn)	Composite of two subgroups--the Krebs and the Cabaniss, both of which consist of sandstone, siltstone, shale, clay, limestone, and coal beds; the Krebs subgroup has an aggregate thickness up to about 400 ft; the Cabaniss subgroup ranges in thickness from a featheredge to 280 ft
	Maquoketa formation (Ord)	Typically a thin laminated, silty, calcareous or dolomitic shale which locally contains lenses of nodular and shaly light brown or gray limestone; color of the shale ranges through various shades of dull green, dark gray, and brown; thickness ranges from 10 to 140 ft
	Marmaton group (Penn)	Includes the Fort Scott and Appanoose subgroups which consist of succession of shale, limestone, clay, and coal beds; Fort Scott thickness ranges from less than 1 ft to about 90 ft; aggregate thickness of the formations in the Appanoose subgroup ranges from 5 to about 190 ft
	Northview formation (Miss)	Brown, buff, green or blue siltstone and blue or bluish-green shale; locally fossiliferous; thickness varies from 2 to 80 ft
	Pleasanton group (Penn)	Group is divided into three unnamed formations; lower formation consists of thin-bedded, medium-grained sandstone (maximum thickness 15 ft) and clay, coal, and shale (maximum thickness 3 ft); middle formation consists of limestone (average thickness less than 1 ft) and micaceous shale (thickness about 50 ft); the upper formation consists of fine- to medium-grained sandstone (thickness up to 150 ft) and a gray shale and fine-grained micaceous sandstone (average thickness is about 25 ft)
MT	Colorado shale (Upper Cret)	Dark gray shale and siltstone with many concretions and sandy units; thickness up to 2200 ft
	Bearpaw shale (Upper Cret)	Dark gray and brownish clay shale; thick units of nonfissile bentonitic shale, calcareous and ferrigenous concretions containing thick bentonite beds; thickness is 600 to 700 ft
	Fort Union formation (Tert)	Clay shale, siltstone, and sandstone; local lenses of impure limestone and numerous lignitic beds; thickness is about 1000 ft
	Pierre shale (Upper Cret)	Dark gray clay shale with calcareous concretions and sandy members; thickness up to 2200 ft
OH	Eden group (Upper Ord)	Calcareous shale with nodular limestone; thickness is 225 to 275 ft
	Vinton member of Logan formation (Miss)	Bluish-gray sandstone and shale; thickness is 30 to 150 ft.
	Bedford formation (Miss)	Soft to firm pinkish or gray shale; thickness is 30 to 120 ft.
	Maysville group (Upper Ord)	Calcareous shale with thin bedded limestone; thickness is 180 to 270 ft
	Richmond group (Upper Ord)	Calcareous shales with thin bedded limestone; thickness is 225 to 355 ft
	Massillon member of Pottsville formation (Penn)	Varies from massive sandstone and shale to interbedded sandstone and shale; thickness is 5 to 65 ft

(continued)

(Sheet 2 of 4)

Table 5. General description of problem shales. (continued)

State	Rock-Stratigraphic and Geologic Time Units	Description
OK	Claypool formation (Perm)	Predominantly shale and sandy shale (chiefly brown with irregular gray and blue mottling) with one or more uniform sandstone members; sandstones are thin bedded, gray to buff, and friable; thickness is several hundred feet
	Johns Valley shale (Penn)	Dominantly shale with minor amounts of sandstone and "boulders;" lower portion contains dark to black shales, whereas the upper portion contains considerable amounts of lighter gray clay shale with some thin interbedded sandstone; upper and lower portions contain "exotic boulders" *of limestone ranging in size from* pebbles to boulders (up to 40 ft in diameter); thickness is 425 to 900 ft
	Labette shale (Penn)	Clay shales, sandy clays and sandstones; thickness is 100 to 250 ft
	McAlester formation (Penn)	Blue to black usually laminated shale with a wide variety of sandy material and sandstone, coal, and clay layers; shales are plastic and work easily; thickness is 800 to 2800 ft
	Nowata shale (Penn)	Clay shale to sandy shale with a few interstratified sandstones and a few limestone beds less than 1 ft thick in the lower portion; thickness is 60 to greater than 600 ft
	Savanna formation (Penn)	Predominately grayish-green fissile shales and clay shales with several sandstone members and a few thin limestones included; sandstones are generally brown, dense, fine grained, micaceous, and thin bedded to massive; sandstones vary in thickness and may reach thicknesses of 200 ft locally; thickness of formation varies from 140 to 2000 ft
	Senora formation (Penn)	Black to gray fissile shale and clay shales with thin bedded to massive buff sandstone layers and two beds (2 to 8 ft thick) of hard massive limestone; thickness is 125 to 500 ft; a lower sandstone member, 35 to 350 ft thick, may be present which consists of a series of thin-bedded gray and light brown sandstones interbedded with silty gray and maroon shales of irregular thickness
	Vamoosa formation (Penn)	Varies from gray-green, blue-green, and maroon shales, sandstones and siltstones, and thin limestones to chert conglomerate, massive coarse red and brown sandstone with red shale; thickness is 125 to 690 ft
	Washita group (Cret)	Consists dominantly of clay shales with minor amounts of limestones and sandstones; clay shales are mostly blue to black, weathering to light gray, yellow, and various other colors; limestones are fossiliferous, gray to yellow, usually with clay beds; limestone layers vary from less than 3 to 6 ft thick; sandstones are soft, yellowish-brown and occur in thin lenses; formation thickness varies from 70 to 415 ft
	Wewoka formation (Penn)	Gray to brown, thin-bedded to massive sandstone members (thickness up to 130 ft) separated by blue to gray silty shales which range from 75 to 130 ft thick; total thickness of formation varies from 400 to 780 ft
SD	Belle Fourche shale (Upper Cret)	Dark gray shale with dolomite and ironstone concretions; may be fissile to blocky; thickness is 110 to 370 ft
	Carlile shale (Upper Cret)	Medium to dark gray, plastic to fissile shale with scattered ironstone concretions; may have stringers of fine-grained sandstone; thickness is about 320 ft
	Mowry shale (Upper Cret)	Light to medium gray siliceous shale interbedded with creamy white bentonite layers (bentonite marker at top): thickness is about 275 ft
	Pierre shale (Upper Cret)	Light gray to dark gray shale with limestone and ferruginous concretions and many thin bentonite layers; thickness is 1000 to 2800 ft

(continued)

(Sheet 3 of 4)

Table 5. General description of problem shales. (concluded)

State	Rock-Stratigraphic and Geologic Time Units	Description
TN	Chattanooga shale (Upper Dev & Miss)	Fissile, black, carbonaceous shale, dark phosphatic basal sandstone member (Hardin) usually present; thickness averages 20 ft in Middle Tennessee and ranges from 0 to 500 ft in East Tennessee
	Murray shale (Lower Camb)	Dull green, brown to grayish-blue micaceous silty and sandy calcareous shale; thickness is 300 to 500 ft
	Nolichucky shale (Upper Camb)	Pastel colored (pink, greenish, olive) flaky clay shale; commonly with oolitic limestone layers, thin blocky siltstone near middle; thickness is 50 to 800 ft
	Pennington formation (Upper Miss)	Heterogenous and varicolored (red, purple, green) clay shale, normally calcareous sandstone, shaly or silty fossiliferous limestone; limestone is minor in amount, proportions of shale and sandstone vary widely; thickness varies from 200 to more than 500 ft
	Rogersville formation (Middle Camb)	Light green, fissile clay shale with occasional beds of thin red sandy shale; in places massive blue limestone bed occurs; thickness is 70 to 250 ft
	Rome formation (Lower Camb)	A heterogeneous and varigated mixture of sandstone, siltstone, shale, dolomite, and limestone; proportions vary greatly; thickness is 700 to 1200 ft
UT	Mancos shale, undivided (Upper Cret)	Three shale members (Tununk, Ghregate, and Musuk) which are separated by two sandstone members, the Ferron and Emery; dark to dull gray shale (fissility develops upon drying and weathering) with several impure bentonite beds in lower shale members; shale is mostly silt size and weakly cemented with calcite and minor amount of gypsum and clay; thickness varies from 3500 to 5000 ft (sandstone members constitute 10 to 30 percent of formation)
VA	Martinsburg formation (Middle & Upper Ord)	Bluish (unweathered) yellowish to brownish (weathered), thin-bedded calcareous mudrock; layers largely composed of very fine quartz sand and many thin layers of fossiliferous limestone with thick beds of clay shale locally; very fine-grained bluish-green sandstone and black calcareous shale also occurs; thickness is 1300 to 4000 ft
	Millboro shale (Middle & Upper Dev)	Black to light gray (weathered) fissile shale (splits into very thin flakes); locally weathers into soft clay shale; lenticular concretions occur in some places; thickness is 200 to 1000 ft
	Rome formation (Lower Camb)	Red and green shales, sandstone, dolomite, and pure limestone vary in proportions and distribution throughout the formation; up to 2000 ft thick
WV	Dunkard group (Penn or Perm)	Nonmarine cyclic sequences of sandstone, siltstone, red and gray shale, limestone, and coal; thickness is about 450 ft
	Hinton formation (Upper Miss)	Heterogenous mass of variegated shales, sandstones of varying character, and impure limestones; thickness ranges from 1000 to 1100 ft
	Monogahela group (Penn)	Consists of tan and gray shales, a few shaly sandstones, clay, coal, and some nonmarine limestones; thickness is 200 to 500 ft

(Sheet 4 of 4)

Figure 7. Occurrence of problem shale formation, California.

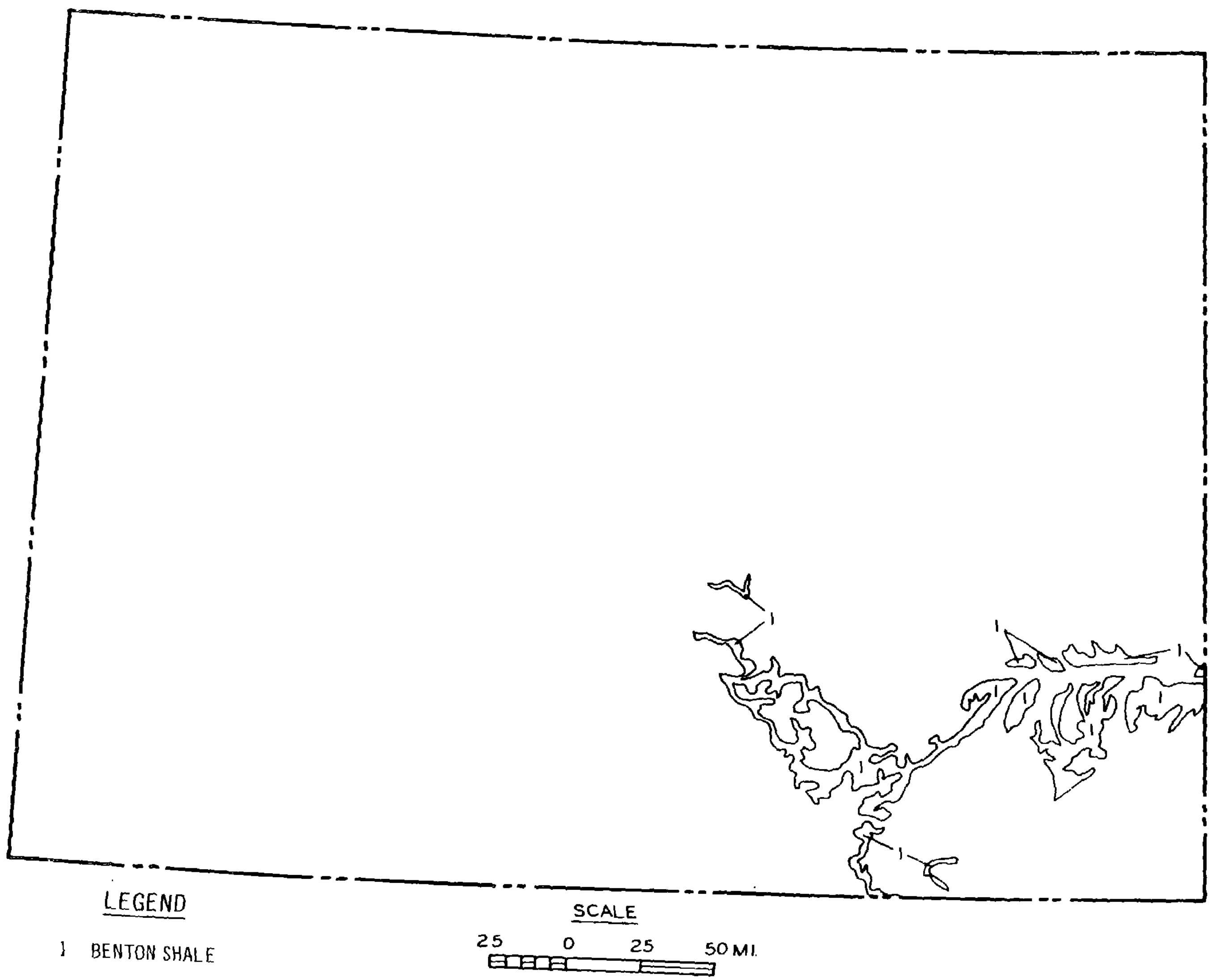
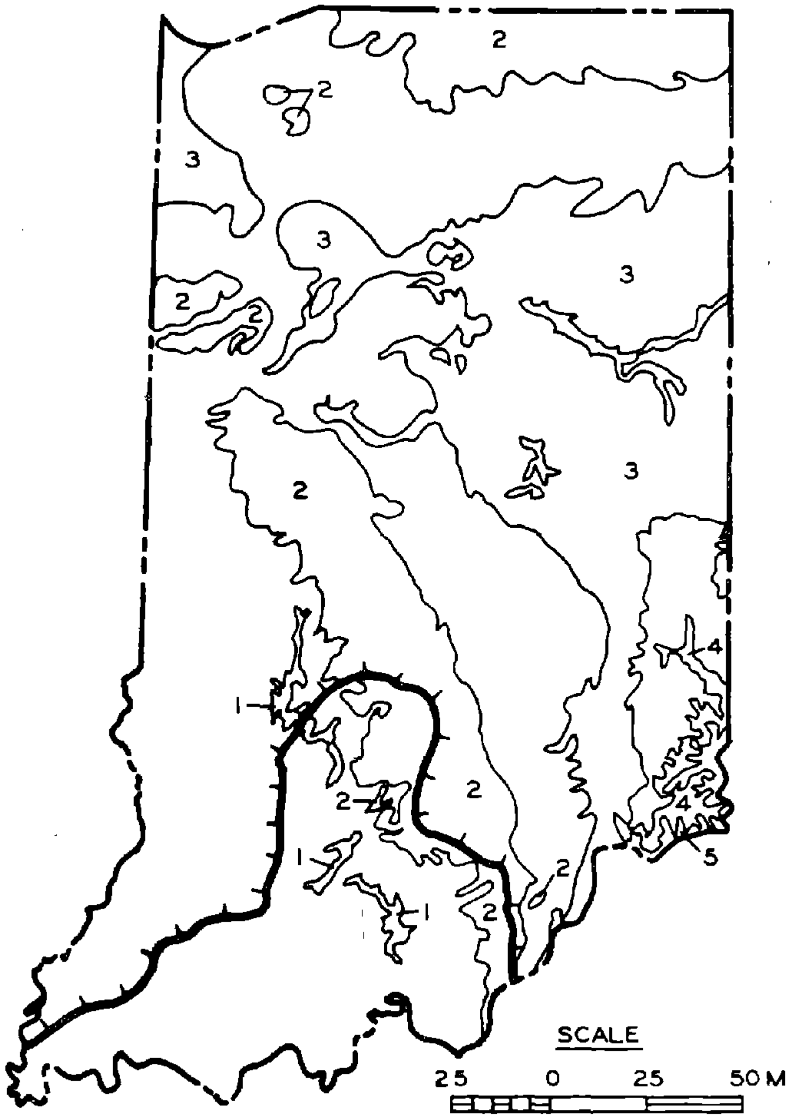


Figure 8. Occurrence of problem shale formation, Colorado.



LEGEND

1 LOWER CHESTER GROUP (INCLUDES BETHEL FORMATION)
2 OSAGIAN AND KINDERHOOK SERIES (INCLUDES OSAGE GROUP)
3 SILURIAN SYSTEM (INCLUDES OSGOOD FORMATION)
4 MAYSVILLE GROUP (INCLUDES DILLSBORO FORMATION)
5 EDEN GROUP (INCLUDES KOPE FORMATION)
GLACIAL LIMIT

Figure 9. Occurrence of problem shale formations, Indiana.

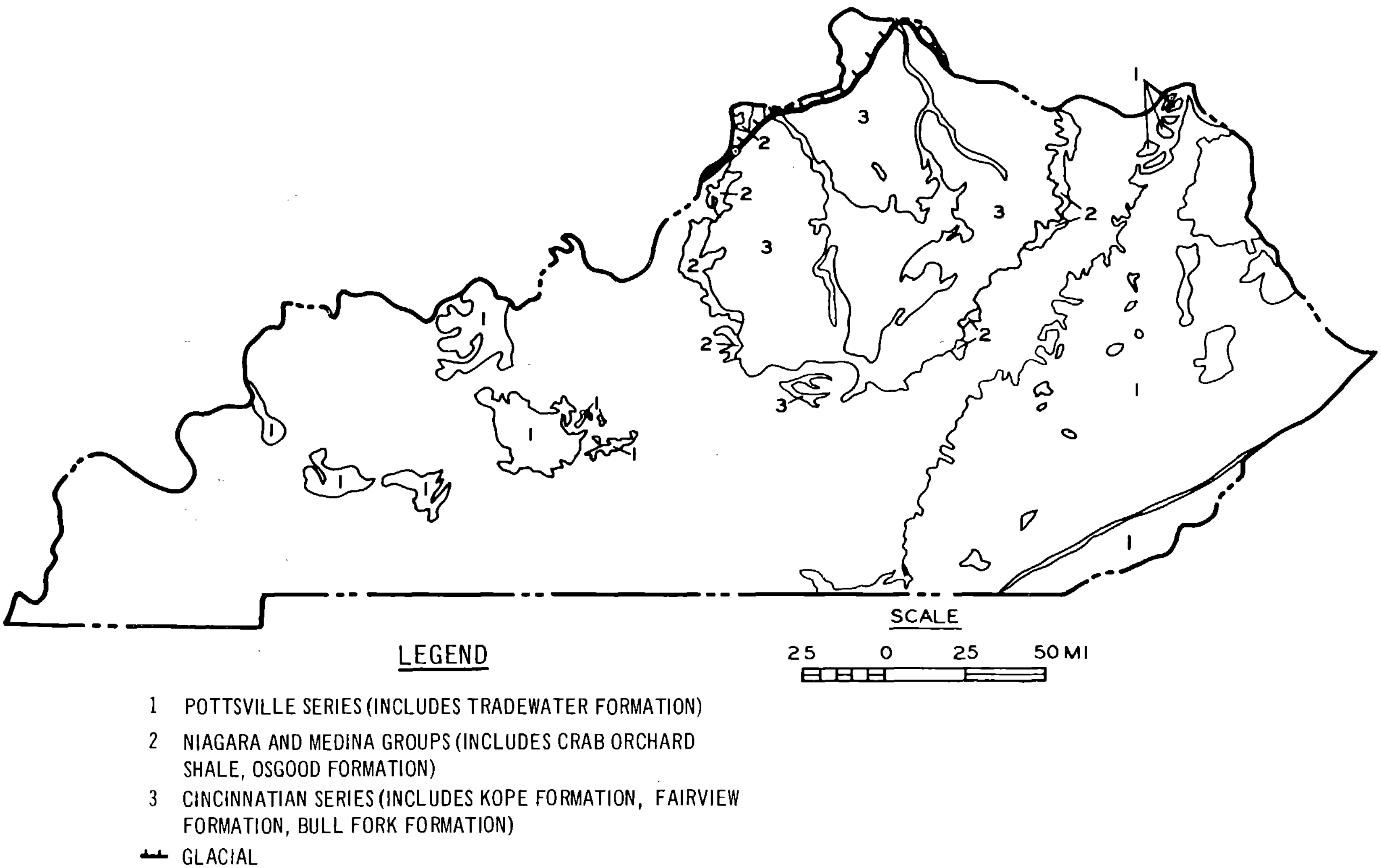


Figure 10. Occurrence of problem shale formations, Kentucky.

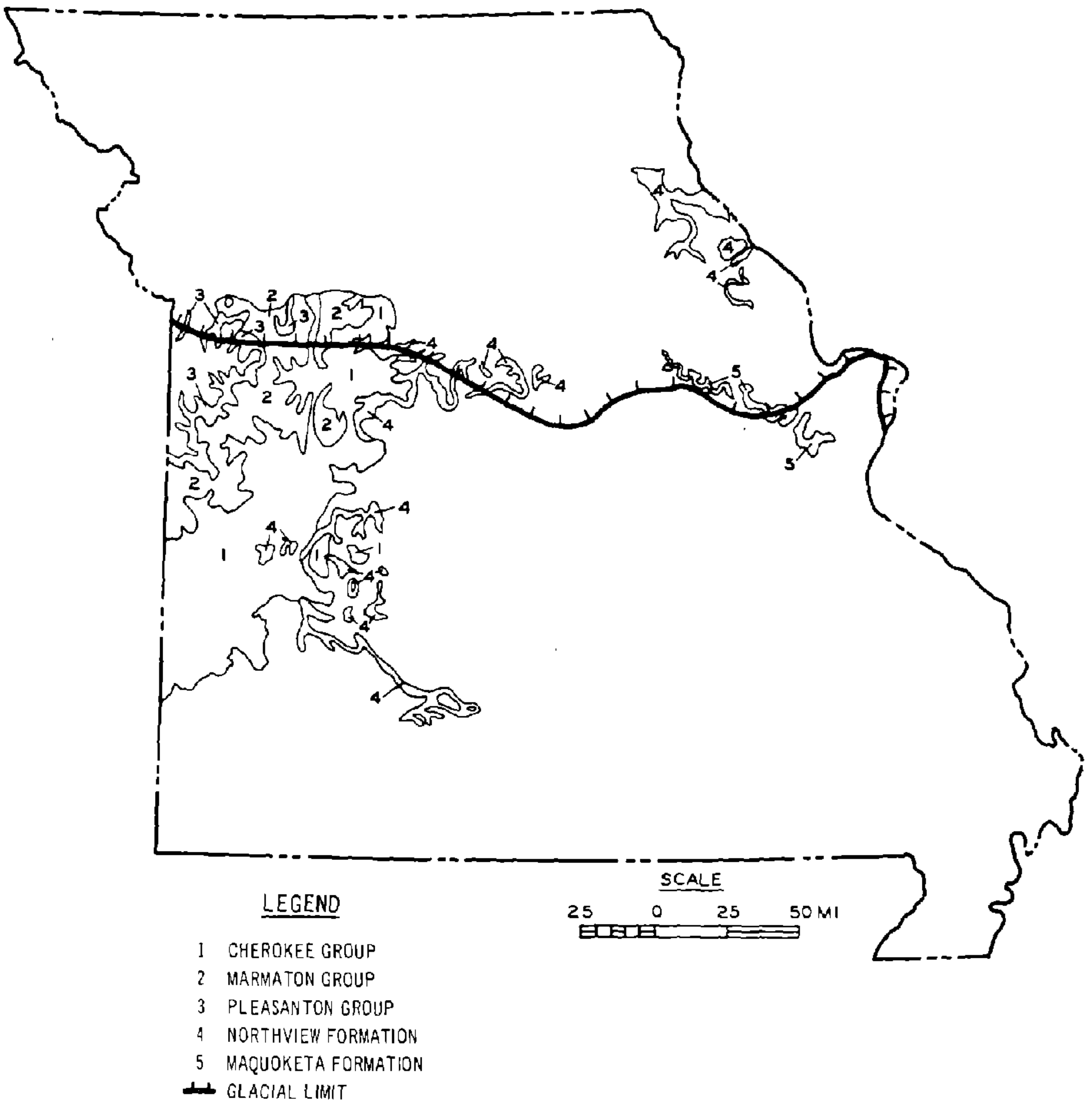


Figure 11. Occurrence of troublesome shales (1, 2, and 3) and problem shales (4 and 5), Missouri.

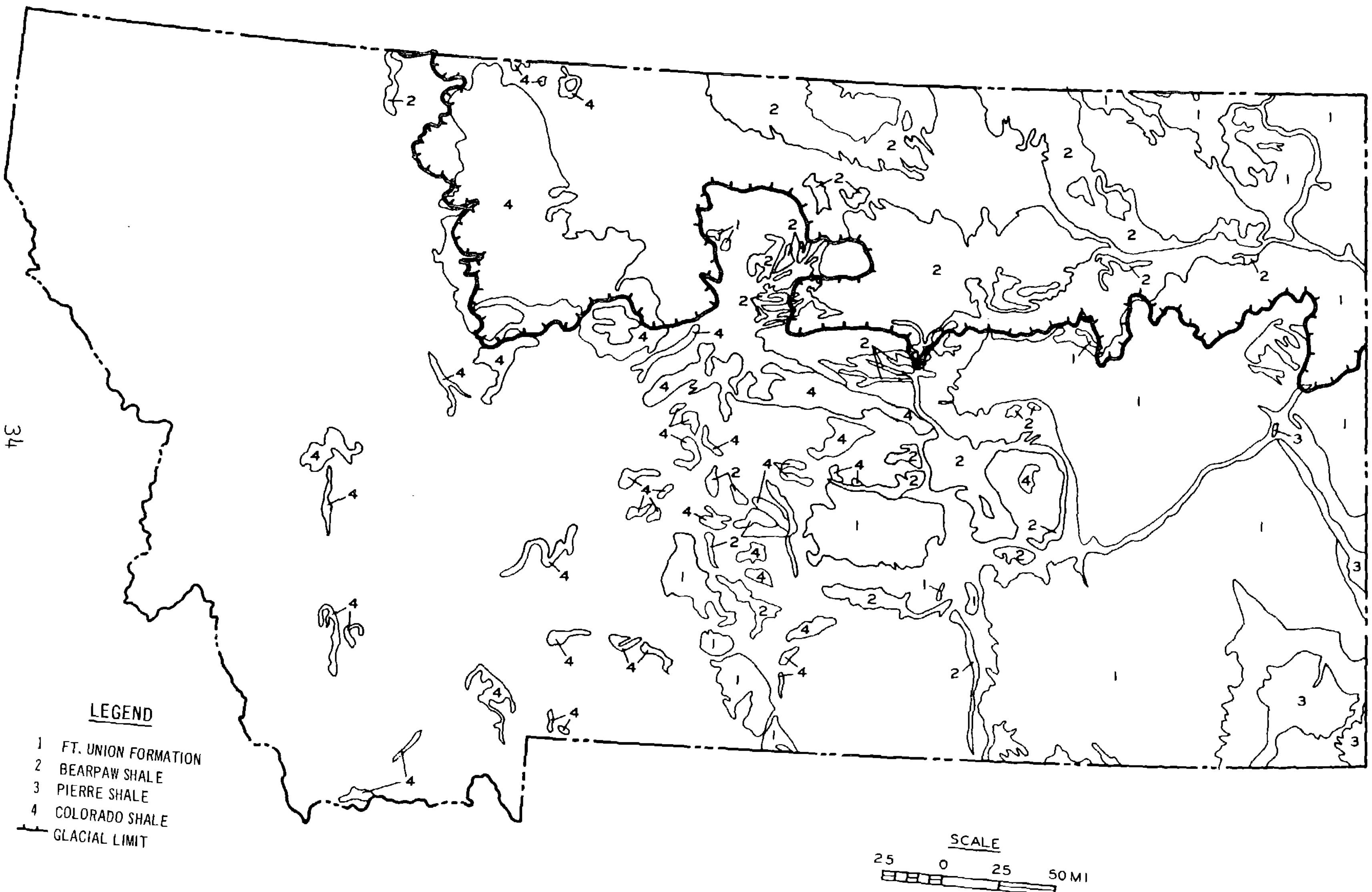


Figure 12. Occurrence of problem shales, Montana.

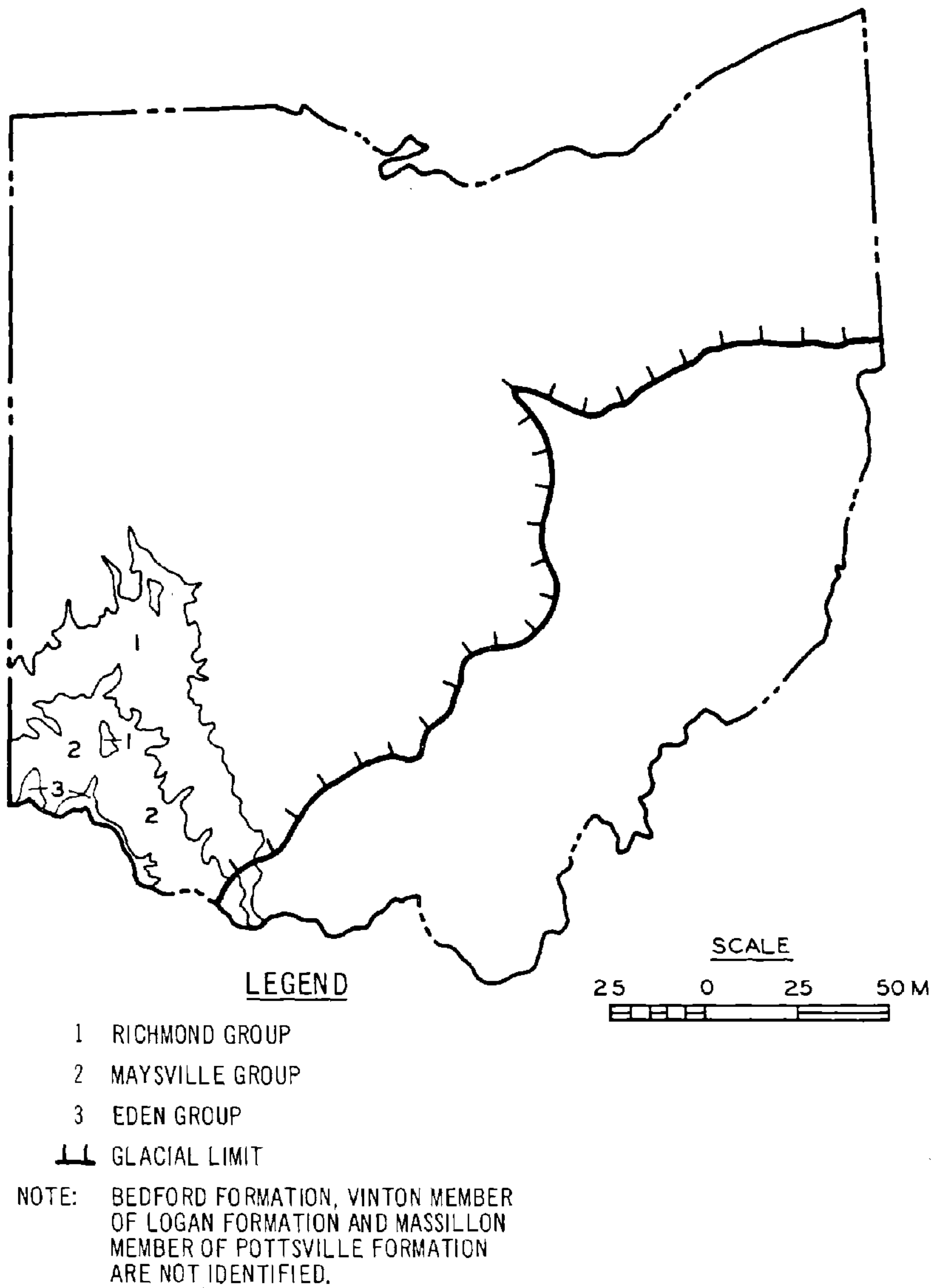


Figure 13. Occurrence of shales presenting problems in the past, Ohio.

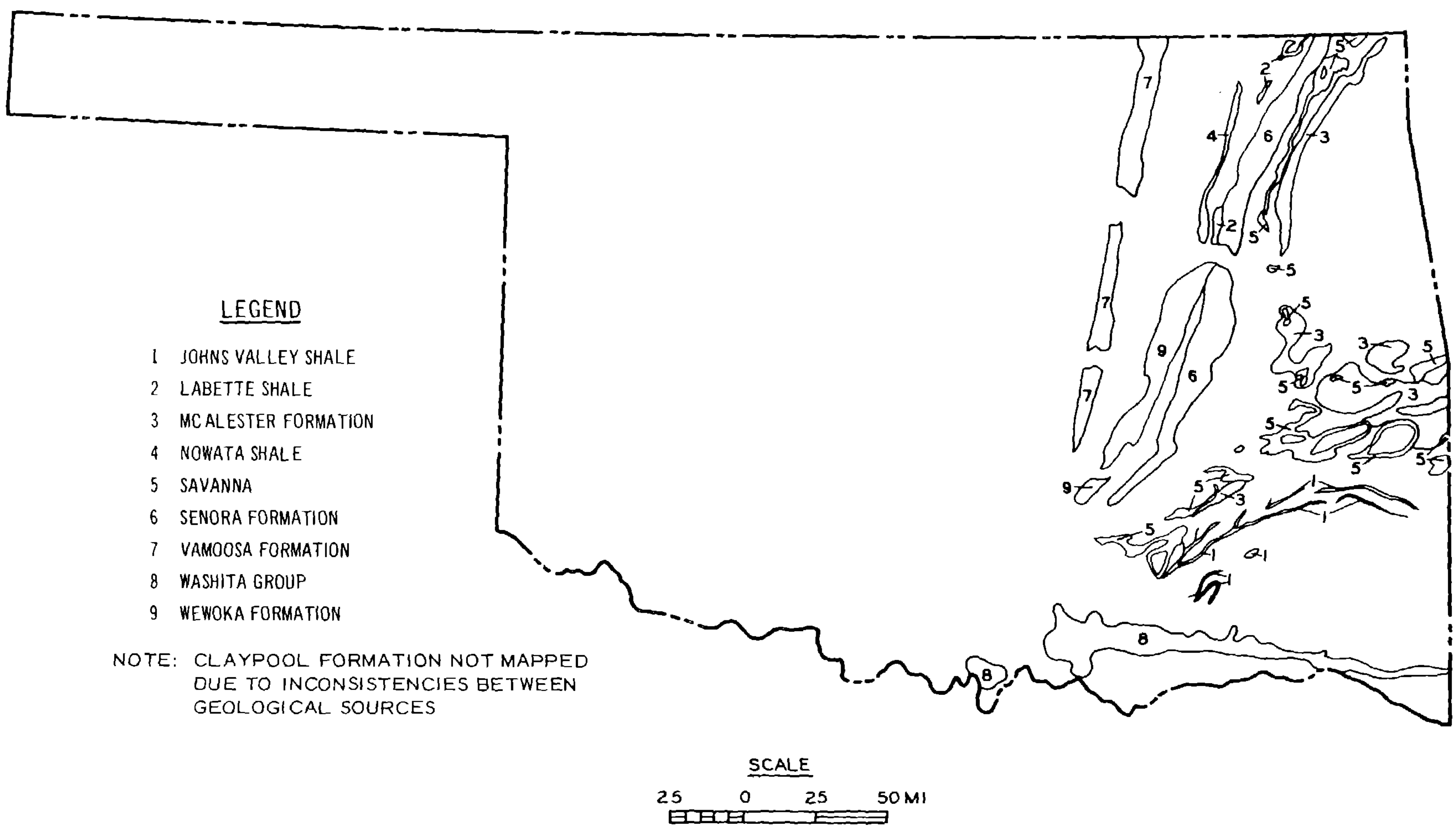


Figure 14. Occurrence of problem shales, Oklahoma.

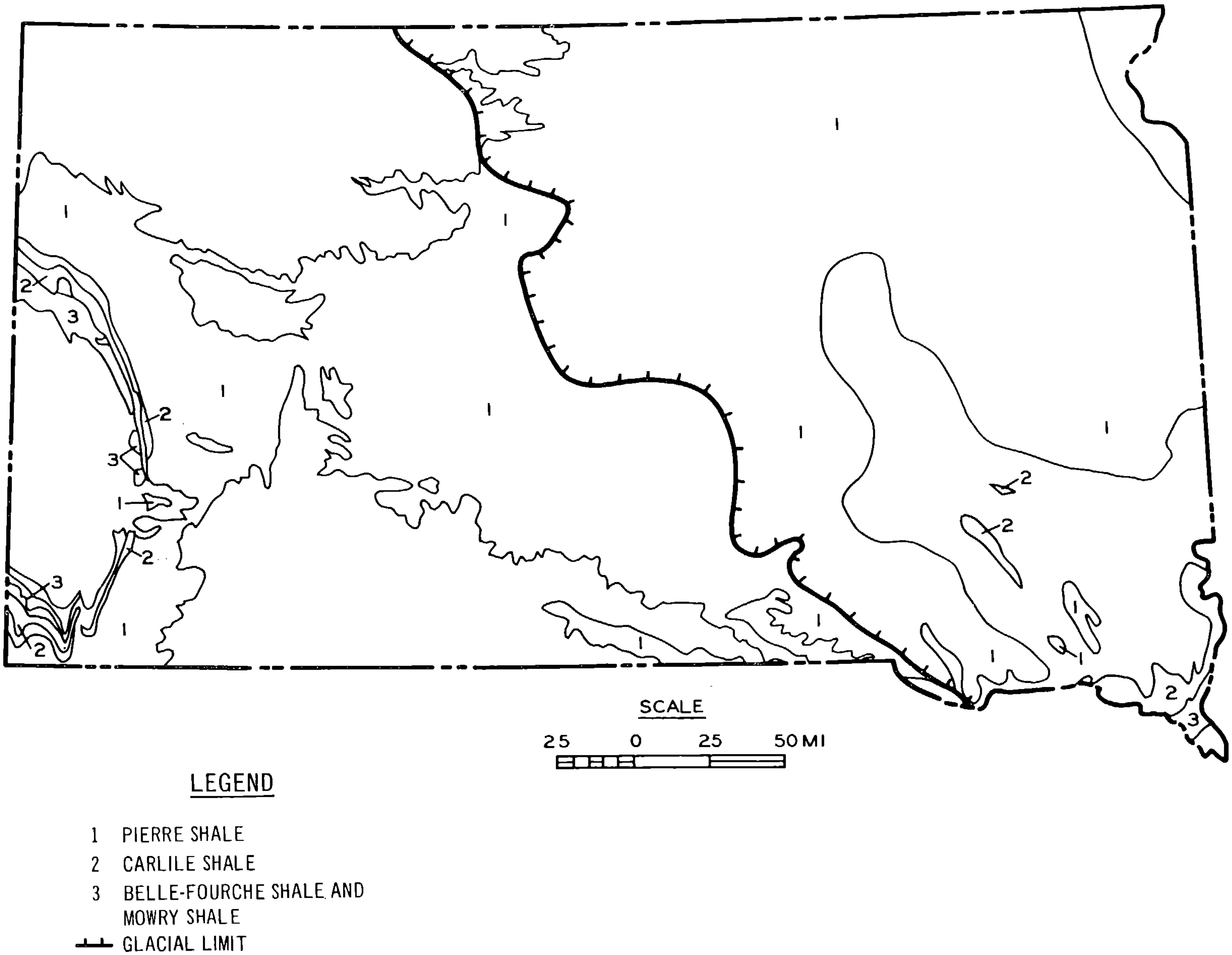
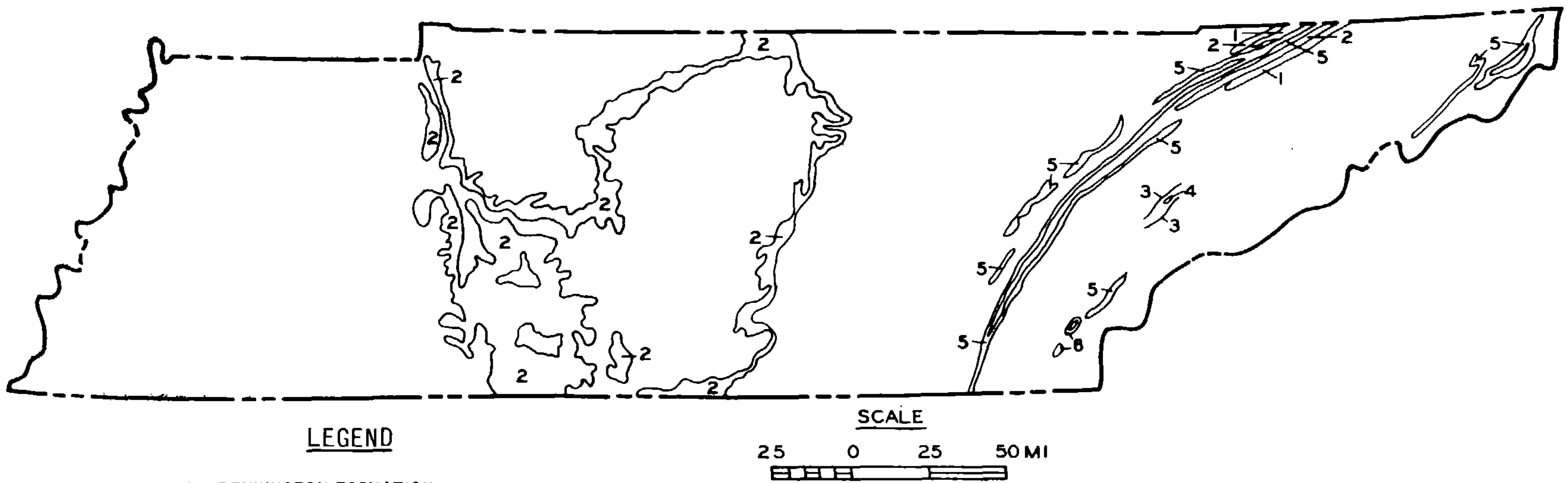


Figure 15. Occurrence of problem shales, South Dakota.



LEGEND

1 PENNINGTON FORMATION
2 CHATTANOOGA SHALE
3 NOLICHUCKY SHALE
4 ROGERSVILLE FORMATION
5 ROME FORMATION
6 MURRAY SHALE

NOTE: PENNINGTON OUTCROP AROUND CUMBERLAND PLATEAU TOO NARROW TO DELINEATE

Figure 16. Occurrence of problem shales, Tennessee.

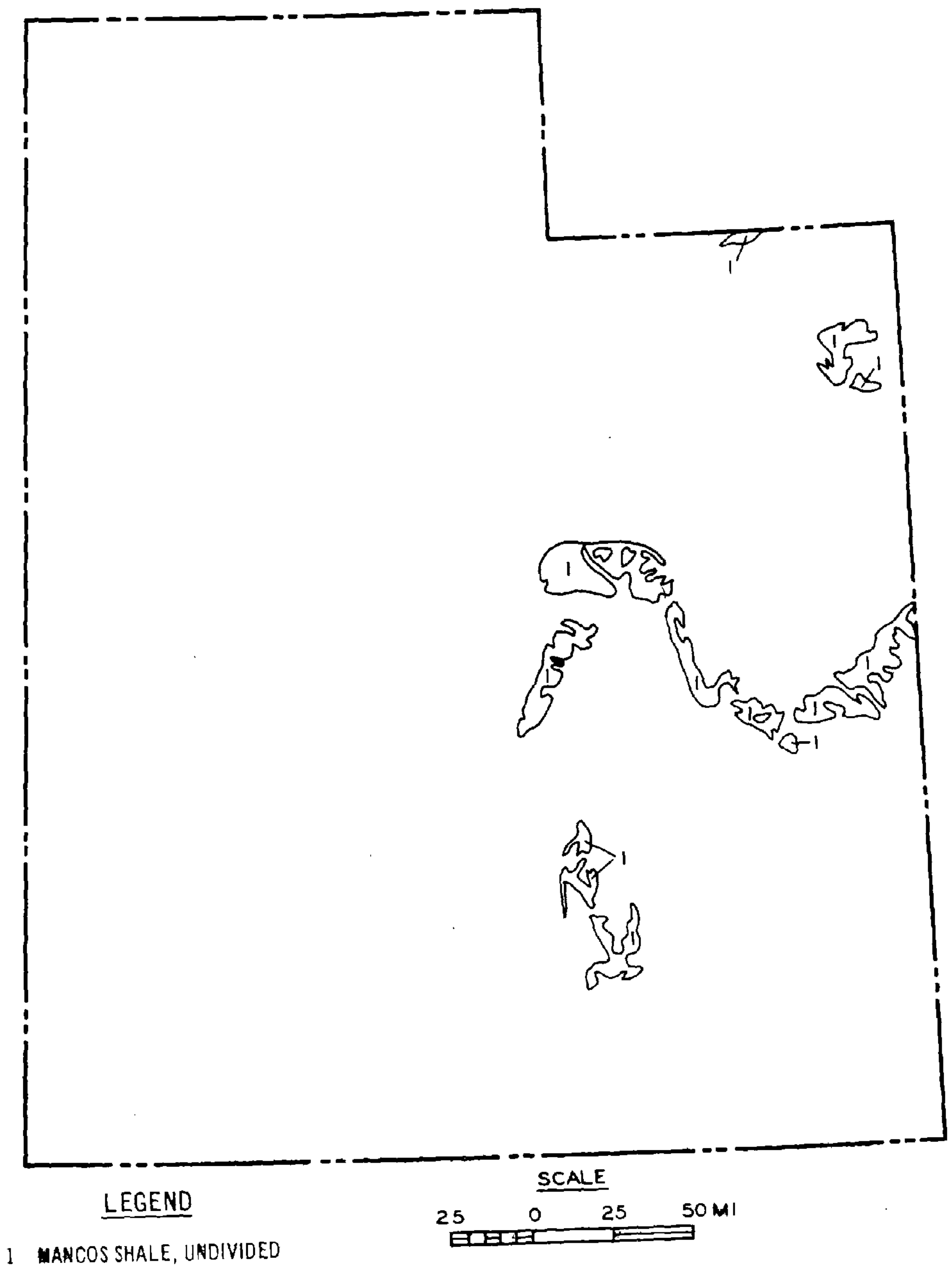


Figure 17. Occurrence of problem shale, Utah.

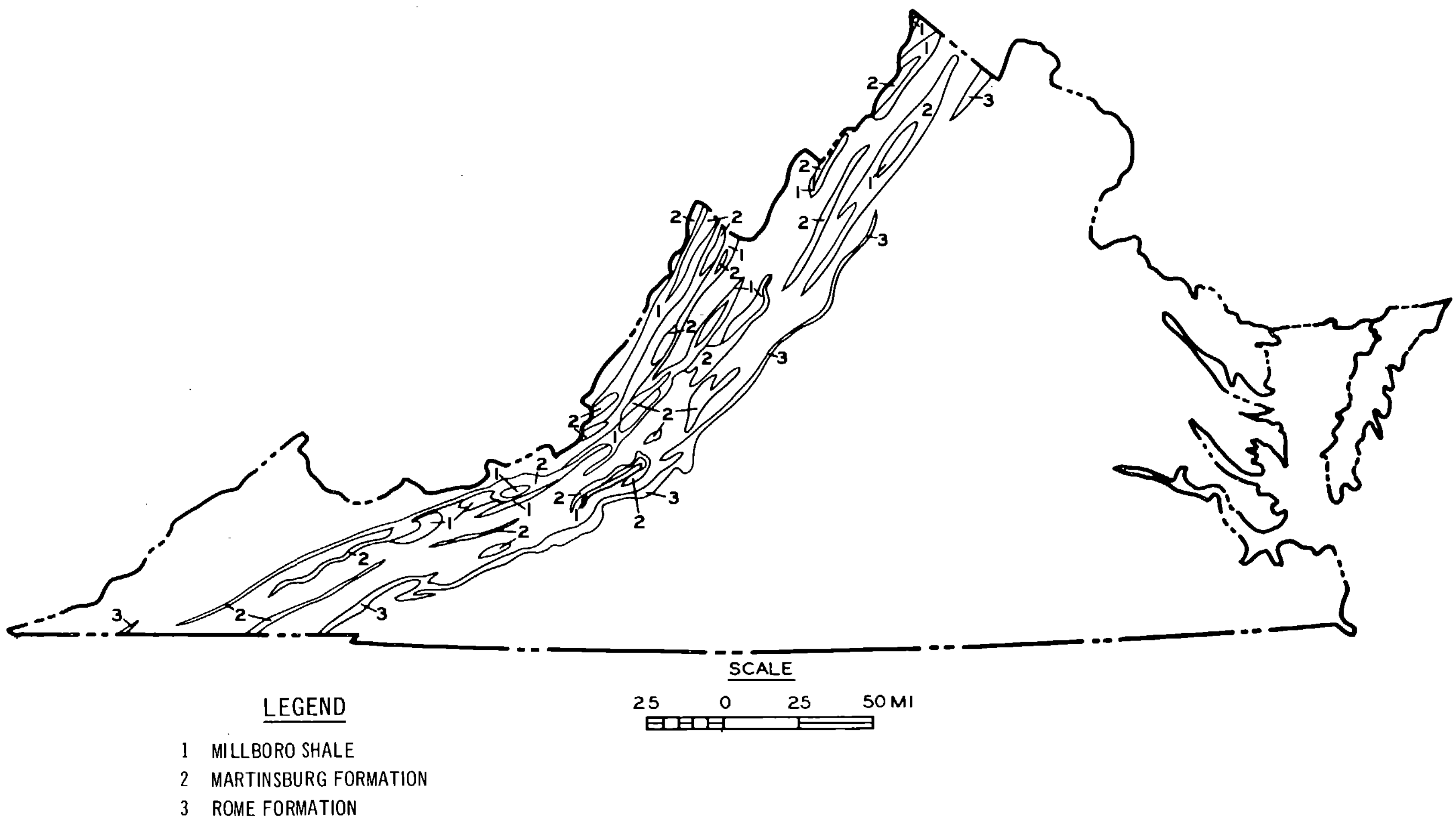


Figure 18. Occurrence of problem shales, Virginia.

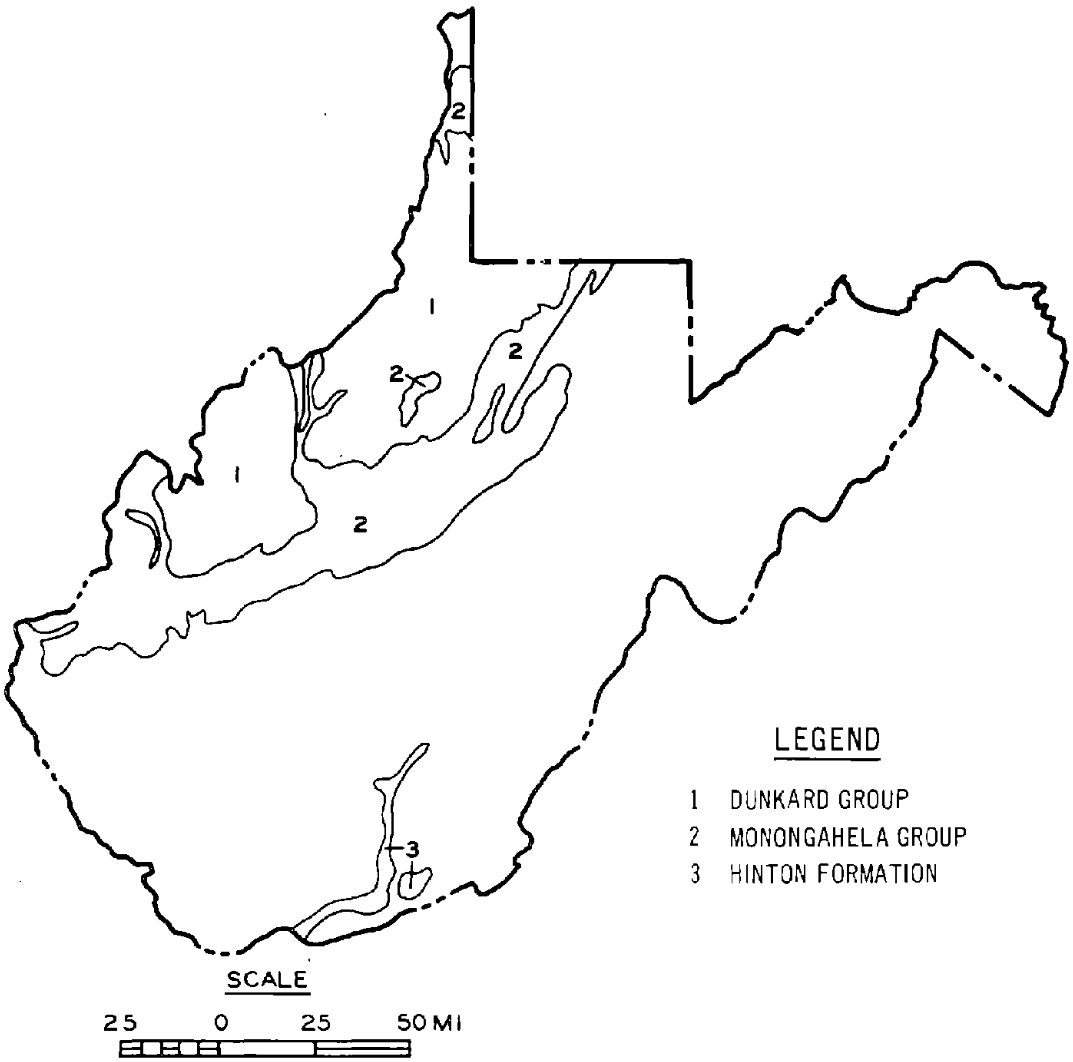


Figure 19. Occurrence of problem shales, West Virginia.

was identified by a highway department by a new name which does not appear on older geological maps. Exact correlations or subdivisions were therefore not possible in all instances. For example, when the original map was compiled, a group or formation may have been identified; but later geological studies resulted in subdivision of a group into two or more formations or subdivision of a formation into members which were given different names. Thus, it was not possible in this study to separate the new formations from the older formations or groups. Hence, the area portrayed on the occurrence maps may be larger than that of the actual problem shale formation.

The northern States (Montana, South Dakota, Missouri, Indiana, Pennsylvania, and Ohio) are partly covered by glacial drift which has masked part of the problem shale outcrop. The thickness of the drift is variable, and consequently the underlying shale may or may not be encountered during highway construction. The southern limit of the glacial line is portrayed on the appropriate maps as a ticked line.

Types of Problems

Types of problems being experienced in the 13 States (3 States indicated that shale embankments were not presenting any problems) were identified during the discussions held with representatives of the highway departments. Summaries of these experiences are presented below. A more detailed description of several problem embankments is presented in Volume 2 (Bragg and Zeigler, 1975), in the sections on embankment problems and remedial treatment. Although not all of the States identified a problem, this part of the discussion includes the experiences of all the states to indicate possible reasons for unsuccessful versus successful performance of compacted shale embankments. Information on current practices is included where appropriate and summarized in the next section for all 16 States.

California. Large embankments higher than 100 ft* were constructed

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page xii.

at Jail Gulch, Pine Creek, and Squaw Creek in the late 1960's. Clay shale materials were successfully incorporated, and these embankments have not exhibited any distress. Generally, following the requirement for a high degree of compaction is cited as the reason for successful performance of shales in high embankments. Only two problem areas were identified. One was on U. S. 80 near Lake Tahoe where silty material was used in fills 50 to 100 ft high. Settlement occurred and cracks extending parallel to the highway were observed in the shoulder area. In a second area along the northern and mid-California coast, fills up to 200 ft high using interbedded sandstone and shale exhibited some cracking and slumping at the outer slope. These problems terminated within a short time and were corrected by routine maintenance.

Colorado. Major problems experienced by the Colorado Highway Department are with expansive soils. Minor settlements have occurred in shale embankments, principally at bridge approaches. The latter are attributed to inadequate compaction rather than to shale deterioration. No failures have occurred in compacted shale embankments. Sliding in foundations and cut slopes has required remedial measures.

Indiana. Indiana has experienced major problems with compacted shale embankments, particularly along a 5-mile stretch of Interstate Highway 74 (I-74) northwest of Cincinnati (DiMillio and Haugen, 1974). The major problems are associated with instability of the slopes, settlement, and longitudinal cracking, which in some cases results in eventual slope failure with encroachment into the shoulder and roadway. Three embankments in this 5-mile stretch failed and required major repairs; several other embankments in the same area are presently exhibiting distress. These embankments were constructed with interbedded limestone and shale during 1964-1965. The failures developed as follows:

a. Settlement and spreading of fill with a sag developing in the roadway 1 to 2 yr after construction.

b. Longitudinal cracking along or near pavement edges, ponding of water in the sag, and possible separation of drainage conduits facilitating infiltration of surface water into the fill.

c. Failure of the fill slope along the shoulder 6 to 7 yr after construction with progressive sliding extending into the pavement lane from rotational slipping or translation of the downhill fill section.

Causes of the problems were believed to be inadequate compaction, excessive settlement, toe erosion, infiltration of surface water and subsurface seepage, deterioration and softening of shale, and improper benching beneath the fill. Investigations of existing fills by test pits to measure in situ density and water content proved inadequate because of the variability and heterogeneity of the fill materials. These characteristics cause major difficulties in trying to assess stability. A consulting engineering firm is under contract to investigate, evaluate, and develop remedial measures where needed for several other fills in the previously mentioned 5-mile stretch on I-74. Shale deterioration or degradation processes are discussed in Part V and a review of the failed embankments is given in Volume 2 (Bragg and Zeigler, 1975).

A classification system based on three types of durability tests (Wood and Deo, 1975) is presently being used to evaluate all shales during the design investigations. Test strips are being used to develop compaction procedures for fill construction on I-64 in southern Indiana.

Kentucky. Compacted shale embankments have presented problems in the form of settlement, cracking, and slope failure. Settlements were noted on I-75 some 2 to 3 yr after construction and failures have been occurring in the last year or two (i.e., some 10 years after construction). Unstable conditions and failures at several other locations have been investigated and remedial measures recommended or implemented. The failures occurred in the central, north central, and southern parts of the State and are not limited to the Interstate Highway System. Settlements have caused rough road surfaces requiring weekly maintenance with the rate of settlement increasing until a lane had to be closed because of hazardous driving conditions. Another problem embankment experienced settlement followed by shoulder slumping and eventual failure. Reports on embankment failures furnished by the Kentucky Department of Transportation and reviewed in the study of remedial treatments (Volume 2, Bragg and Zeigler, 1975) indicated that many of the failures may have started in the foundation.

Bridge approach fills of shale-rock mixtures have been instrumented, and 2 to 4 yr of data on settlement at the tops of the fills have been collected. Repeated profiles taken within 50 ft of the abutment indicated initial heave then up to one foot of settlement. Experience has indicated that slopes of 1V on 2H results in long-term deformation problems while slopes of 1V on 4H are stable. However, there is no apparent trend in deformation with respect to slope height. The relocation of Kentucky Highway 15 at Carr Fork Lake specified rock fill, but shale was mixed with the rock. Slopes of 1V on 2H were used and deformation occurred which also affected the bridge abutments.

Missouri. Current problems with compacted shale embankments are mainly caused by swelling rather than by deterioration of shale material. Past failures of 1V on 2H embankments and differential settlement of highways in the Kansas City area may have been due to shale deterioration. However, these embankments were built in the 1950's and current construction practices (summarized in Table 6) eliminate this type of problem. In the current practices, shale is treated as a soil for compacted fill, and acceptance or rejection criterion is based on the liquid limit of the shale. This criterion and flatter side slopes have apparently solved the problem.

Montana. Variations in the performance of embankments in Montana can be attributed to construction practices, and settlements have occurred in some embankments. Distress usually occurs near the contact between the fill and natural foundation at the cut section where settlements on the order of 3 to 4 ft have occurred in extreme cases. Shear failures have also occurred in three embankments. The main contributor to embankment settlements is the increase in water content. The quality of the pavement influences the amount of water infiltrating into the embankments. Problem areas were identified in the vicinity of Great Falls, Bozeman, Reed Point, and Fort Benton.

A practice on some of the interstate construction has been to build one lane first (e.g. westbound lane) and to construct the other lane (eastbound lane) later. Prior to construction of the second lane, distress or failure in the original lane is studied and designs are revised for the second lane in distressed areas.

North Carolina. Representatives of the State highway department stated that they had not experienced any problems with compacted shale embankments. They also indicated that only a relatively small amount of shale occurs in the State (Figure 6). Where shale is encountered during highway construction, it breaks down on handling and is compacted as soil.

Ohio. Shale embankments constructed after 1963 are not exhibiting any distress. This current situation is attributed to more rigid specifications requiring shale to be placed as soil fill. Prior to 1963 excessive settlement was experienced in a 55-ft and a 70-ft embankment. Both embankments were instrumented with settlement gages. Settlement of the 55-ft-high embankment amounted to 2 to 3 in. per yr with 12 to 15 in. occurring over a 10-yr span. The 70-ft-high embankment at another location experienced a settlement of 26 in. over a 5-yr period. The large settlements were attributed to the use of thick lifts and inadequate compaction which produced a high percentage of voids (10 to 20 percent). Cement grouting was used to stabilize the fills with some 1240 cu yd being used on the 70-ft-high fill.

Following the more rigid specifications, a shale-rock mixture placed in 8-in. lifts and compacted with four coverages of a tamping roller and two coverages of a 50-ton rubber-tired roller performed excellently in a 40-ft-high fill. Although settlement measurements were not made, no maintenance has been required. In another 55-ft-high fill (with four settlement gages at the base of the fill and three at higher elevations) experienced 7 in. of settlement in the foundation 6 months after construction, but the embankment had settled only 1/4 to 1/2 in. during this time. Eight to nine years later, total embankment settlement was only 2 to 4 in.

Special precautions are used near bridges for a distance of six times the abutment height back from the abutment. Water content and density control testing is required for each layer and soil type.

When rock is used for embankments, it is placed in 18-in. lifts (maximum) and rolled to the satisfaction of the project engineer.

Shales are placed as soil in 8-in. lifts, and large rock fragments are broken down to 8 in. or removed. Reduction in strength has occurred in some embankments.

Oklahoma. Stability is not a problem with embankments constructed from shales in Oklahoma since the topography does not require high embankments. Also, the low volume of fill required for embankments allows the use of conservative embankment designs, e.g. slopes of 1V on 3H.

After excavating a shale cut and subsequently placing and shaping the embankment, some shales deteriorate. The deterioration is manifested as undulations in the roadway, rutting, shoving, potholing, and cracking of the roadway surface. Experience has shown that in testing for Atterberg limits, the plasticity index (PI) of such shales may be near zero when first excavated. However, soon after construction, the roadway deteriorates and soil samples from beneath the subgrade exhibit PI's ranging from 15 to 25 or higher.

About 75 percent of the fills do not exhibit any deterioration problems to a significant degree. The problems are mostly in areas of 40 in. or greater rainfall in eastern Oklahoma.

Oregon. Compacted shale embankments in Oregon are not presenting problems that the highway department personnel could attribute only to the embankment. Some failures have occurred which were identified as foundation problems. Failures have occurred in cut slopes, and major landslides have been experienced. Shale in Oregon used for highway construction breaks down upon handling and is placed as soil fill rather than rock fill. Shale is often stockpiled during wet weather and rehandled later during placement; this practice may aid in breaking down the shale.

Pennsylvania. The Pennsylvania Department of Transportation requires shale embankments to be compacted according to soil specifications; significant problems have not occurred in embankments. Occasionally some effort is made to zone embankments. The coarsest material is placed to the outside in the lower part of the embankment and the finer material to the inside and the higher part of the embankment. A minor embankment problem occurred about 15 yr ago in the north central part of the State where shale broke down during wet weather and muddy conditions developed due to exposure of the material to wetting and drying.

South Dakota. Although most of the embankments in South Dakota are less than 30 ft high, some may reach 150 ft in height. Highway department personnel indicated that embankment problems within the State can be attributed principally to foundation conditions but that some settlement occurs within embankments. Problems are being experienced on bridge approaches across the Missouri River north of Pierre.

The major problem is with swelling shales and clays in embankments and foundations. About 2 to 3 yr are required for significant volume changes to develop. A satisfactory method for maintaining a constant water content in the embankments has not been found.

Tennessee. Two major problems are being experienced with shale embankments, excluding foundation failures. These problems involve slope failures due to trapped water and steep slopes probably caused by a rapid breakdown of clay shales to clays, and differential settlement of the pavement structure. A basic cause of settlement is deterioration of shale chunks resulting in fines filling the voids and causing the fill material to retain water. Settlements of up to 4 ft have occurred at the center of some fills with very little differential settlement of the pavement structure. However, wavy surfaces have developed in the roadway on other fills. One sidehill fill indicating distress was investigated by a consulting engineering firm, and it was recommended that the fill be sampled every 6 months to determine reduction in strength. Slope indicators at the shoulder showed movement in the lower 10 ft of this 90-ft-high fill although no water was present in an observation pipe. Samples, apparently of chunks on the order of 3 ft in size, indicated that deterioration had occurred partway into the shale chunks.

Utah. The main problem in Utah with shale embankments involves the swelling characteristics of the shale. Settlements have occurred in embankments sections, but no significant problems have developed. The expansive problems are concentrated at the cut-fill interface, where heave occurs in the winter and sags occur in the summer. Sections of Kentucky Highways 10 and 27 and I-70 in the east central part of the

State are in the Manços shale where severe problems occur.

Virginia. One major problem with compacted shale embankments was indicated in Virginia. At Clifton Forge on I-64, three embankments placed during 1969-1971 are showing excessive settlement and lateral movement. The severest condition is found in two 70-ft-approach fills for twin bridges over Commercial Avenue and Smith Creek. The fills are of shale, blasted mainly from one cut, loaded, and hauled to the fill sections, and spread by D-9 dozers in 2-ft (maximum) horizontal layers. The layers were compacted with a tamping roller. The fill under the bridge abutments has settled approximately 12 in. in 3 years and moved laterally requiring the ends of the bridge to be raised and the girders to be shortened (about 10 in.). The natural ground under the fill slopes steeply toward the bridge. An investigation of the fill has included split spoon sampling, slope indicator readings, and settlement observations. Mineralogical studies of the clay fraction of split-spoon samples from this fill are discussed in Part IX. The acidation and clay weathering are believed to be important by the highway district geologist in the deterioration process which also may be affected by infiltrating deicing salts. Infiltration of rainfall and possibly subsurface seepage from the adjacent cuts are cited as factors contributing to deterioration. No water was encountered in slope-indicator borings until the foundation was reached. The other two embankments are located in the same area and are showing similar distress to a lesser extent.

West Virginia. Settlement of embankments constructed of shale was identified at two locations in West Virginia. The first location involves several fills east of Princeton along a section of U. S. 460, a divided highway completed in 1971. At one site, the fill in front of twin bridge abutments on one side of a creek settled about 8 in., and slope-indicator readings showed slight movement obliquely towards the creek. Cracks 1/2 to 1 in. wide occurred in the fill surface under the bridge.

Along the same highway further east of Princeton, four fills up to 200 ft high have settled at a relatively constant rate amounting to some 12 to 15 in. by 1974. The concrete pavement in the central portion of each fill was replaced by asphalt pavement prior to opening the section to traffic. The central portions were overlaid twice during a 3-yr period. A comprehensive subsurface investigation by the State highway department was initiated in 1972. Settlement data, slope-indicator data, and geophysical logging data obtained periodically are being used to monitor and evaluate the conditions of these fills.

These four fills include a sidehill fill benched into unweathered shale; a through fill (i.e. on a fairly flat foundation); a partial sidehill fill; and a combination sidehill and through fill. The fills were constructed using special provisions developed for shale materials. Over 1,000,000 cu yd were placed in the four fills with one fill being completed in approximately 30 days. Most fill material was classified as hard shale or rock except for some soil placed in the lower portion of one of the fills near the bottom of the long grade. Slope-indicator data from a boring on each side of one fill indicated that the downhill side had deformed outward several inches over the lower two-thirds of the fill height but that the other side had not deformed. Some weathering and deterioration of the shale in the four fills are believed to be occurring. Relatively high water levels exist in cased borings in the fills near the bottom of the long grade.

The other location is at Wheeling where mine waste composed of shales was used for an embankment that failed. The weathered materials were quite wet, and probably had low strengths.

Current Highway Construction Procedures

A second major objective of Task A was to determine current highway department practices for construction of shale embankments and the types of remedial measures used within the States being studied. Current practices were divided into three broad categories: (a) preconstruction, (b) construction, and (c) remedial measures. Categories (a) and (b) were subdivided to cover specific methods and procedures. The sources of information used for compiling the current practices were standard specifications, special provisions, construction manuals, material manuals, case studies, and personal communications with State highway department representatives. A summary of the current practices by States is presented in Table 6. Variations in the practices are discussed in the following paragraphs.

Preconstruction Practices

Preconstruction surveys include the procedures that are followed to identify the surface and subsurface conditions that will be encountered during highway construction. For this discussion, these surveys were separated into soil and geologic investigations, subsurface explorations, and laboratory tests. All State highway departments follow similar approaches with the type of field exploration, sampling, and testing varying somewhat between States. The complexity of terrain conditions within a highway project also influences the amount of detail required during the preconstruction survey. The results of preconstruction surveys are presented in a report in a memorandum and are retained in the project file.

Soils and geologic investigations. These investigations include office studies and field surveys. During office studies, all pertinent information and data are collected for the segment of highway to be constructed. Sources of information include available geological reports published by the State geological surveys or USGS, soil survey bulletins published by the U. S. Soil Conservation Service, and topographic maps from the USGS. Other data generated by previous work in the area are also reviewed. Air photo interpretation techniques are used by some States to identify soil and rock conditions and assist in the field surveys. After all pertinent data have been assembled, compiled, and reviewed, the field survey is initiated. The type of field data collected varies with States and projects. The field surveys generally include a reconnaissance along the planned center line for collection of appropriate data and examination of rock exposures (streambanks, etc.) to develop sections on subsurface geologic and soil conditions. When spacings of borings or test pits are not standardized, the spacing may be decided in the field or adjusted from those selected during the office study of results of the center line reconnaissance.

Table 6. Current highway practices, compacted shale embankments.

Item	California	Colorado	Indiana	Kentucky	Missouri	Montana	North Carolina	Ohio
I. PRECONSTRUCTION								
A. Background data-soil & geol investigation	Geol & soils reports, air & ground photos, topo maps, existing project data	Geol & soils maps, existing test reports & field data	Geol & soils maps, topo maps, existing field data	Geol & soils maps, topo maps, existing field data	Geol & soils reports, existing field data	Geol & soils reports, topo maps, air photos, existing field data	Geol & soils reports, topo maps, air photos, existing field data	Geol & soils reports, field reconnaissance, existing field data
B. Field exploration								
1. Spacing and depth of boring	Determined for each project	Determined for each project	Spacing deter. by project; depth: 6 ft or 2/3 height of fill; through soft to solid material	Min of 200 ft spacing along center line. 5-ft depth below grade or to solid rock	Spacing deter by project; depth: norm 10 ft below grade	Spacing deter by project; depth: 5 to 10 ft below grade or 1.5 times fill height	Avg spacing 100 ft; depth: 5 to 10 ft below ground surface	Deter by found explor; depth to rock if feasible
2. Samples	Disturbed, split-spoon, undisturbed	Disturbed, bulk, split-spoon, cores	Bulk, auger, split-spoon, cores	Auger, split-spoon, undist, cores	Auger, split-spoon, cores	Bulk, split-spoon, undist, cores	Disturbed	Split-spoon, undisturbed, cores
3. Other	Resistivity, subaudible rock noise	None identified	Geophysical test pits, quarry insp, piez, slp ind	Seismic (not used routinely)	Resistivity	Geophysical	Have used resistivity	Have used seismic and resistivity
C. Materials tests								
1. Physical	Grain size, compaction, slaking, swelling, triaxial (R), direct shear, stabilometer	Grain size, Atterberg limits, specific gravity, compaction, load-deform, swelling, slaking, stabilometer	Grain size, Atterberg lim, sp grav, nat wc, slaking, durability, compaction, triaxial, shrinkage lim, CBR, organic cont	Grain size, Atterberg limits, specific gravity, slaking, compaction, unconf comp, triaxial (R), direct shear	Grain size, Atterberg limits, specific gravity, slaking, soundness, direct shear	Grain size, Atterberg limits, specific gravity, water content & density on undisturbed, shrinking-swelling shear, permeability, bearing cap	Grain size, Atterberg limits, special cases: unconf comp, triaxial	Grain size, Atterberg limits, specific gravity, slaking, soundness
2. Chemical	None	pH, sulfate, chloride content	20% H_2O_2 using slake durability residue	Organic content	Insoluable residue test	Organic content	None	Sodium sulfate soundness
3. Acceptance or rejection criteria	Lab tests	Proj engr judgment	Purdue Univ durability tests	Lab tests & inspection of core samples	Lab tests	Field & lab tests	None	Lab tests & proj engr judgment

(continued)

(Sheet 1 of 8)

Table 6. Current highway practices, compacted shale embankments. (continued)

Item	California	Colorado	Indiana	Kentucky	Missouri	Montana	North Carolina	Ohio
II. CONSTRUCTION CRITERIA								
A. Site preparation	Cleared & grubbed of all vegetation & objectional material where grading plane is natural ground; where grading plane is >2 ft above natural ground, stumps etc., are cut 1 ft above natural ground	Cleared &/or grubbed of all trees, stumps, roots, & other protruding objects not designated to remain except undisturbed stumps and roots & nonperishable objects at a min of 2 ft below subgrade or base slope of embankment	Cleared &/or grubbed; sound stumps, roots, & nonperishable objects will be 3 ft below subgrade and not more than 4 in. above natural ground surface. No shale, weathered rock, or rock. Shall be within 2 ft of subgrade	All vegetation shall be cleared and grubbed except as designated. Areas not grubbed, trees, and stumps 6 in. or larger in diameter measured 6 in. above ground are cut within 3 in. of ground line. Other trees cut at ground surface. In grubbed areas all trees, roots, etc., grubbed except undisturbed stumps, roots, and nonperishable objects are at a min of 3 ft below subgrade or embankment slope and extend not more than 6 in. above ground line	All trees, stumps, brush, and hedge not to remain are grubbed, or cleared and grubbed as required; undisturbed stumps not greater than 6 in. above natural ground that are 3 ft or deeper below finished earth grade may remain	Trees, stumps, brush, shrubs, and other vegetation cut off 6 in. above grnd and removed; stump roots, logs, or timber greater than 3 in. in diameter shall be removed to a depth of not less than 12 in. below original ground surface	Cleared & grubbed of all trees, stumps, roots, etc., except where fills are greater than 3 ft in which case no stumps will be left within 3 ft of finished subgrade; all unsound stumps, etc., removed to depth of 2 ft below original ground; embankments less than 3 ft grubbed within limits of clearing or special provisions followed	Excavation or embankment areas are scalped (removal of vegetation, etc., from ground surface) except where embankments are greater than 4 ft in height to subgrade elevation. Where natural ground will be a minimum of 3 ft below subgrade of embankment, undisturbed stumps, roots, and nonperishable solid objects can be left provided they do not extend greater than 6 in. above ground surface
B. Emplacement								
1. Compacted as	Soil & rock	Soil & rock	Soil & rock	Soil & rock	Soil	Soil	Soil	Soil
2. Emplaced as soil								
a. Lift thickness	8-in. loose	8-in. loose	8-in. loose (maximum)	12-in. loose	8-in. loose	8-in. loose	6-in. loose	8-in. loose (larger rocks broken down or removed)
b. Compaction equipment	Segmented pad, sheepsfoot, grid rollers & other approved types	Sheepsfoot, grid rollers, and other approved types	Approv vibratory tamping roller (55,000 lb) plus static roller (60,000 lb); min foot length = loose lift thick; min 2 cov w/vib plus 3 cov w/static per lift; procedure checked by test pads and No. passes adjusted to obtain desired density	Sheepsfoot, pneumatic-tired rollers or other approved type	Sheepsfoot (16- to 20-ton), vibratory rollers or other approved equipment	Metal, smooth wheeled, self-propelled rollr meeting weight specifications, towed steel-drum, pneumatic-tired, grid, vibratory rollers & other meeting weight specifications	Flat-wheel powered rollers, sheepsfoot tamping roller or pneumatic-tired rollers approved by proj engr	Determined by the aontractor, adequate to obtain specified compaction, sheepsfoot & 50-ton rubber-tired roller have yielded good results

(continued)

(Sheet 2 of 8)

Table 6. Current highway practices, compacted shale embankments. (continued)

Item	California	Colorado	Indiana	Kentucky	Missouri	Montana	North Carolina	Ohio
3. Emplaced as rock								
a. Lift thickness	Max., in. / % Rock >6 in. Max. rock size / >50 36 / 25-50 8 / <25	Engr may permit maximum layer thickness of 36 in. and emplacement of occasional larger size boulders	Not to exceed top size of material being placed but will not exceed 24 in.	12-in. maximum desirable; 18-in. maximum acceptable	NA	NA	NA	NA
b. Compaction equipment	Sheepsfoot rollers	Sheepsfoot rollers	Sheepsfoot rollers	Sheepsfoot rollers	NA	NA	NA	NA
4. Soil compaction requirements								
a. Moisture-density specification	Relative compaction of not less than 95% within 3 ft of grade; below 3 ft relative compaction not less than 90%; moisture content as required to obtain relative compaction requirements	Soils A-1, A-3, A2-4, A2-5 ≥ 100% (T99) or ≥95% (T180) of maximum dry density, all others ≥95% (T99) or ≥90% (T180) of maximum dry density	At least 95% of maximum density on dry side of optimum moisture (-3% to -1%); values will change due to different locations and shale formations	Not less than 95% of max density near optimum moisture as practicable	At optimum moisture or above	Not less than 95% of maximum density; optimum moisture content ±2% unless modified by engr	Not < 90% of density obtained by compacting a soil sample, required to obtain maximum density for other moisture content satisfactory to engineer	Lab Max Dry Den pcf / Min % Comp; Fill Ht <10' / >10' 90-104.9 / 100 / 102 105-119.9 / 98 / 100 ≥120 / 96 / 98 moisture not >3% points above optimum or reduced to optimum to obtain required density
b. Comp tests for material	Relative compaction, CA test method 216	AASHO T99 method A or AASHO T180 method A	AASHO T99 as modified in 203.34; method A for soil, method C for granular	AASHO T99 method A or C	AASHO T99-7D, method C	MT test method: MT 210 or AASHO T99, method A or C	AASHO designated method (T99-57) method C	AASHO T99 or other approved methods
c. Frequency of field density tests	Determined by proj engr or his staff	Minimum 1 test every 2000 cu yd emplacement	No established procedure, frequency of tests to meet engr's approval	Determined by proj engr	Recommended procedure--a minimum of 4 tests per day for each grading spread	1 test per 2000 cu yd for A4-A7 material; 1 test per 4000 cu yd for A1-A3 material	Every 500 ft per lift	Determined by proj engr

(continued)

Note: NA = not applicable

Table 6. Current highway practices, compacted *shale embankments*. (continued)

Item	California	Colorado	Indiana	Kentucky	Missouri	Montana	North Carolina	Ohio
d. Shale embankment slopes								
(1) Soil								
(a) Slopes	1V on 1.5H 1V on 2H 1V on 3H	Fill Ht,ft / Slope <10 / 1V on 6H 10-25 / 1V on 4H >25 / 1V on 3H	1V on 2H to 1V on 4H	1V on 2H to 1V on 4H	1V on 2H prefer 1V on 3H	Fill Ht,ft / Slope <10 / 1V on 6H 10-25 / 1V on 4H >25 / 1V on 3H	1V on 1.5H 1V on 2H or flatter	1V on 2H (normally)
(b) Height	5-250' typical 80-100'	Average 10-30' maximum 50'	<10-150'		Less than 50'	Maximum 80' average 30-40'	No data obtained	5-60'
(2) Rock								
(a) Slopes	1V on 1.5H or flatter	Same as soil, special cases 1V on 2H	1V on 2H or flatter	1V on 2H to 1V on 4H	NA	NA	NA	NA
(b) Height	5-250'	Average 10-30' maximum 50'	<10-150'		NA	NA	NA	NA
C. Remedial treatment (other than *overlaying* pavement)	Stripping, widening, and flattening *slopes*; installing subdrains and horizontal drains	Perforated pipe subdrain, stripping impervious foundation materials, drainage blanket	Remove and *flatten slope*, add drainage blanket or horizontal drains and/or berm. Reconstruct embankment with benching on sidehill, drainage systems (including transverse drains along low points), flatter slopes and/ or berms. Stabilizing keyways (stone and "B" borrow), isolated, drilled, reinforced piers	Remove and *flatten slope*, add drainage blanket or horizontal drains and/or berm. Reconstruct embankment with benching on sidehill, drainage systems (including transverse drains along low points), flatter slopes and/ or berms. Crib walls, sand shear keys	Weather walls, flatten slopes, install drainage ditches, and subdrains	Install: subdrains, relief wells, horizontal drains, interceptors. Stripping; placing longitudinal and transverse subdrains in sidehill fills	NA	Remove, add drainage blanket, reconstruct embankment with benching on sidehill with flatter slopes and/or berm. Grouting has been used in special cases where voids in embank*ments* have been suspected

(continued)

Table 6. Current highway practices, compacted shale embankments. (continued)

Item	Oklahoma	Oregon	Pennsylvania	South Dakota	Tennessee	Utah	Virginia	West Virginia
II. PRECONSTRUCTION								
A. Background data-soil & geol investigation	Geol & soils publications & other available information (not performed routinely for every project)	Published geol & soils reports & other applicable data	Published geol & soils reports, topo maps, field reconnaissance, air photos	Published geol & soils reports, borings, physical property tests, air photos	Published geol & soils reports, field data, topo maps	Published geol & soils reports, topo maps, borings, air photos	Published geol & soils reports, topo maps, air photos, field reconnaissance	Published geol & soils reports, topo maps, borings, air photos, field reconnaissance
B. Field exploration								
1. Spacing and depth of boring	Set up by geologist on project requirements	Determined by regional geologist	Spacing depends upon geological conditions. Depth depends upon height of fill	Spacing determined by geologist and engineer. Depth is 4 to 5 ft below grade	Spacing determined by proj requirements; depth: 4 ft below ditch or 1-1/2 times fill or refusal	1000 ft spacing along center line to 5 ft below grade	No standard spacing, depth 3 ft below grade	No standard spacing, depth to 5 ft below grade
2. Samples	Shelby tube, disturbed, cores to be taken in near future	Auger & split spoon	Disturbed, undisturbed, and cores	Disturbed & cable tool sampler	Disturbed, split spoon, undisturbed cores (AN, NX, BX)	Disturbed and undisturbed	Disturbed, bulk, and cores	Bulk, split spoon, shelby tubes, and cores
3. Other	Seismic, but not routinely	Seismic	Seismic and resistivity	Resistivity	Seismic and resistivity	Resistivity	Resistivity	Seismic
C. Materials tests								
1. Physical	Grain size, Atterberg limits, slaking, compaction, unconf comp, direct shear, soundness, bearing cap	Grain size, Atterberg limits, specific gravity, slaking, absorption, soundness, abrasion	Grain size, Atterberg limits, specific gravity, compaction, direct shear, bearing cap	Grain size, Atterberg limits, specific gravity, swelling	Grain size, Atterberg limits, specific gravity, compaction, unconf comp, triaxial comp, bearing cap	Grain size, Atterberg limits, specific gravity, compaction, triaxial comp, bearing cap	Grain size, Atterberg lim, specf gravity, water content, compaction, abrasion, soundness, swelling, CBR	Grain size, Atterberg lim, specf gravity, water content, compaction, density, triaxial comp, direct shear, CBR, bearing cap
2. Chemical	None routinely, pH on occasions	None	None	None	None, except iron content sometimes	Soluble salt content	pH on occasions	None identified
3. Acceptance or rejection criteria	Lab tests	Field tests	Lab tests; material with >35% passing No. 200 seive: min dry density is 95 lb/cu ft, max LL = 65	Proj engr judgment	Proj engr judgment	Field tests	Proj engr or geologist judgment	Lab & field tests; proj engr judgment

(continued)

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Table 6. Current highway practices, compacted shale embankments. (continued)

Item	Oklahoma	Oregon	Pennsylvania	South Dakota	Tennessee	Utah	Virginia	West Virginia
II. CONSTRUCTION CRITERIA								
A. Site preparation	Trees, logs, stumps, brush, and objectionable material removed and disposed of within right-of-way. Stumps and roots removed except undistrubed stumps and roots and nonperishable solid objects which will be a minimum of 3 ft below subgrade. Where grubbing is not required, stumps cannot extend greater than 6 in. above ground	Clearing of trees, high stumps, and vegetation. If not to be grubbed, stumps will not extend greater than 6 in. above ground. Where grubbing is required & embankments less than 4 ft high, stumps removed within 3 ft of subgrade or slope surface. Roots and embedded wood removed if less than 1 ft below subgrade or slope	Embankments less than 5 ft: cleared and grubbed to greater than 8 in. below ground level. Embankments greater than 5 ft: cleared and stumps cut to 6 in. above ground	Cleared &/or grubbed except for undisturbed stumps, roots, and nonperishable solid objects which will be greater than 3 ft below subgrade or slope surface. When authorized, stumps, etc., may be left which do not extend greater than 6 in. above natural ground line	Cleared and grubbed. In swampy areas undisturbed stumps cut off 6 in. above ground surface or at low water level if approved by engr. Depressions resulting from grubbing, filled and compacted with suitable material	Cleared and grubbed. Trees, stumps, etc., removed to a depth less than 3 ft below subgrade or embankment slope except no stumps, etc., left greater than 1 ft above natural ground surface	Cleared and grubbed. Undisturbed stumps, roots, and nonperishable objects which will be a min of 5 ft below top of earthwork or slope of embankments may be left. In no case more than 6 in. above eixsting ground surface or low water level	Cleared and grubbed. Where embankment is greater than 5 ft, undisturbed stumps, nonperishable objects need not be grubbed provided they are cut off not to protrude more than 6 in. above original ground surface
B. Emplacement								
1. Compacted as	Soil	Soil	Soil	Soil	Soil and rock	Soil	Soil and rock	Soil and rock
2. Emplaced as soil								
a. Lift thickness	8-in. loose	8-in. loose	8-in. loose	8-in. loose	10-in. loose	12-in. loose	8-in. loose	8-in. loose
b. Compaction equipment	Rubber tired, vibratory and smooth wheel rollers	Type required to obtain density	Rubber tired, conventional and vibratory sheepsfoot rollers	Sheepsfoot roller or other types approved that will obtain required density	Rubber tired, sheepsfoot rollers or other approved types	Smooth wheeled, sheepsfoot rollers	Type approved to obtain required density	10-ton, 3-wheel, rubber tired, grid, and sheepsfoot rollers

(continued)

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Table 6. Current highway practices, compacted shale embankments, (continued)

Item	Oklahoma	Oregon	Pennsylvania	South Dakota	Tennessee	Utah	Virginia	West Virginia
3. Emplaced as rock								
a. Lift thickness	NA	NA	NA	NA	36-in. maximum	NA	2 to 10 ft below grade: maximum 24 in.; >10 ft below grade: max 48 in.	24-in. maximum (hard shale) or less depending on size of excavated material
b. Compaction equipment	NA	NA	NA	NA	No type specified	NA	No type specified	Vibratory compactors, grid, paddle foot or other roller types approved by the engineer
4. Soil compaction requirements								
a. Moisture-density specification	Not less than 95% of standard density unless otherwise specified. Moisture content will be that required to obtain density	Within 3 ft of subgrade: not <95% of relative max density. Below 3 ft: not <90% of relative max density; (+)2 or (-)4 percentage points of optimum moisture	Not less than 97% dry weight density below 3 ft. Top 3 ft not less than 100%. Moisture content in top 12 in. not more than 2 percentage points above optimum except under specified conditions	95% of max dry density. Moisture content not lower than 4% points below optimum moisture and shall be such to produce required density	Not less than 95% of maximum density unless otherwise specified. Moisture range to obtain required density. Not to exceed optimum to extent material pumps under construction equipment	Not less than 96% of maximum lab density. Moisture shall be that required to obtain specified density	Minimum density of not less than 95% of theoretical max density. Optimum moisture (+) or (-) 20% as required to obtain specified density	Not less than 95% of max dry density (+)3% or (-)4% points from optimum at time of compaction.
b. Comp test for material	AASHO T99, method C or D	AASHO T99, method A or C, adjust for oversize: No. 4 sieve AASHO T224	Penn. test, method 106, method B	Test No. SD 104 (AASHO T99)	AASHO T99, method A	AASHO T99, method D	Virginia test, method 1	West Virginia Dept of Highways test procedures MP 712.21.22 MP 712.21.25
c. Frequency of field density tests	1 test per 500 ft of lane; 1 test per 1000 ft of lane on shoulder	1 test per 5000 cu yd	No standard criteria	Specified criteria for various depths below top of fill	1 test every 1500 ft or 5000 cu yd; 1 per lift as a minimum	Average of 4 tests in an area of 2500 ft or less	1 test per 500 ft of lane	Embankment: 1 test per layer each 1/2 mile on each working width or 1 test per each 5000 cu yd. At pipe installations, 1 test per day per installation

(continued)

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Table 6. Current highway practices, compacted shale embankments. (concluded)

Item	Oklahoma	Oregon	Pennsylvania	South Dakota	Tennessee	Utah	Virginia	West Virginia
d. Shale embankment slopes								
(1) Soil					Fill Ht,ft Slope			
(a) Slopes	1V to 1/4H to 1V to 3H	1V on 2H (standard) in some instances 1V on 1.5H	Up to 1V on 1.5H	High fills 1V on 3H low fills 1V on 5H	<10 1V on 4H 10-20 1V on 3H >20 1V on 2H	Variable from 1V on 2H to 1V on 4H	1V on 2H and flatter	1V on 2H
(b) Height	Average 15-30' maximum 135'	20->200' 100' common	<10-120'	4-30', up to 150' at some locations	<10-180'		5-75'	5 to ±200'
(2) Rock								
(a) Slopes	NA	NA	NA	NA	1V on 1.5H	NA	1V on 2H or steeper	1V on 2H special cases 1V on 1.5H
(b) Height	NA	NA	NA	NA	<10-180'	NA	5-100'	Few feet to ±200' (may be a combination of soil and rock)
C. Remedial treatment (other than overlaying pavement)	Removal and replacement. Lime is used with replacement material	NA	NA	Subdrains	Buttress fills, removal, drainage systems including drainage blankets	Stripping, subdrains	Subdrains, drainage trenches, horizontal drains, cut-off steel bridge beams	Replacement, drainage blankets, drainage trenches, berms, H-piles

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Field explorations. The boring program during preconstruction varies with the individual States. A few States have standardized spacings ranging from 100 to 1000 ft along the center line; however, in most States, the spacing is determined by the geologist and/or engineer after appropriate information from the highway center line reconnaissance and the project requirements have been considered. The depths of borings are usually specified in terms of either feet below ground surface, below grade, or to rock.

Most of the highway departments have several types of drilling rigs. The type of rig used is dependent upon the type of sampling that will satisfy the soil test requirements. For example, auger borings are adequate to obtain samples for grain-size analysis. Undisturbed samples are taken in unconsolidated material for which shear strength or consolidation tests are required. Cores of various sizes are taken in rock formations.

Geophysical techniques used in the preconstruction surveys by some States are seismic refraction and/or resistivity. The surveys are not routinely performed by all States having the capability.

Material tests. The States have a variety of physical soil testing capabilities, and the types and number of tests performed depend on the conditions determined for each project. The one exception is that soil classifications (grain size and Atterberg limits) are determined for all projects. The shales are broken down with a pulverizing apparatus or manually to sizes for which limits can be determined. The amount of effort required to break the shale down depends on the hardness of the shale (i.e., cementation, degree of consolidation, etc.). Other physical tests specified for project investigations tests include for water content, density, slaking, specific gravity, swelling, absorption, compaction, durability, permeability, unconfined compression, direct shear, triaxial compression, soundness, abrasion, bearing capacity, CBR, and stabilometer. The types of tests performed by each State are indicated in Table 6. Standard AASHTO test procedures are used in some States for tests such as water content, specific gravity, and Atterberg limits. However, most States have modified different AASHTO test methods in different ways and have developed original procedures which are published in some form of a materials test manual.

Chemical tests are performed by about half of the States, though some of these States do not perform them routinely. The types of chemical tests vary between States and include pH, sulfate content, chloride content, organic content, iron content, insoluble residue, soluble salts, and chemical slaking. The States that perform chemical tests and the type(s) of test performed are identified in Table 6.

Acceptance or rejection criteria for placement of shale in an embankment differ considerably between States. The term "acceptable" means that the material is acceptable in its natural condition or acceptable after special treatment. The special treatment may be a reduction in lift thickness, water content control, density requirements and control, segregation, or placement of shale in a specified zone of the embankment. Acceptance criterion varies from fairly rigid requirements to a subjective judgment. Most of the States use laboratory and/or field tests but do not define the criteria for acceptance or rejection. The project engineer or geologist makes the judgment for using shale in some of the States. Indiana uses laboratory durability tests (Wood and Deo, 1975) for accepting or rejecting shale for use as rock fill. Missouri uses liquid limit values in rejecting shale or requiring special treatment. Pennsylvania has requirements based on liquid limit and density for rejecting shale for embankment construction.

Most of the States did not indicate that any large quantities of shale had been rejected. In fact, most States cannot follow this practice since additional borrow as well as spoil areas for the reject shale would be required.

Construction Criteria

The construction criteria (Table 6) are subdivided into requirements for site preparation, placement, compaction, and embankment side slopes.

Site preparation. The specifications for this phase of construction for all States are similar. Table 6 presents only a summary of information contained in the specifications. The detailed requirements are presented in the specifications, and are not necessarily restricted to embankments.

Placement. An important criterion is whether shale is to be placed as soil or rock. Nine of the States require shale to be placed as soil, and seven States allow shale to be placed either as soil or rock, depending upon the characteristics of the shale. The placement criteria are shown in the latter part of Table 6 according to placement as soil only or as soil or rock.

Shale placed as soil. The maximum loose lift thickness allowed when materials are placed as soil varies from 6 to 12 in.; the majority of States (12) specify an 8-in. loose lift thickness. Two States specify a maximum loose lift thickness of 12 in.

The type and often the weights of compaction equipment are identified in the specifications. Although some States do not place restrictions on the type of compaction equipment which can be used in constructing a compacted embankment, the equipment must meet the approval of the project engineer and be capable of achieving the required density. The number of passes may or may not be identified in the specifications, and the phrase "or until the required density is obtained" is frequently used. Speed is sometimes specified for types of compaction equipment and may include maximum and minimum limits during operation.

Shale placed as rock fill. The maximum loose lift thickness of shale placed as rock fill varies from 18 to 48 in. Some States specify the desired maximum rock size and stipulate a maximum allowable size. Presplitting is practiced to reduce the size of rocks. Three States have special provisions for shale, and the criteria for using the special provisions are identified in the specifications. Indiana requires shale to be compacted in 8-in. lifts, with a special provision of 24 in. maximum lifts for compacting rocklike shale (determined from durability tests, see Figure 27) that does not break down to sizes smaller than 8 in. West Virginia classes shale as soft or hard depending upon whether it can be broken down with specified compaction equipment, weight, and number of passes. These criteria are used to determine lift thickness during placement. There has been a tendency for States having problems with shale to reduce the lift thickness. Kentucky has reduced the allowable lift thickness for problem shales to 12 in. desirable and 18 in. maximum. California specifies the allowable maximum lift thickness, for materials containing rock, on the basis of the percent of rock having a size greater than 6 in. and requires 90 percent relative compaction. The type of equipment and number of coverages may or may not be identified; however, both items are usually specified on a project basis, with the approval of lift thickness left to the engineer who bases his decision on the capability of the equipment and number of passes required to obtain the specified density.

Soil compaction requirements. The moisture-density specifications vary somewhat among the 16 States, but all are specific concerning the minimum density required. The compacted densities required by the States are identified as percentages of the optimum density determined by laboratory methods under specified methods. Test methods used by the States to determine optimum density are identified in Table 6. The AASHTO T 99 method (AASHTO, 1974) is the most generally used; however, four States specify their own test methods which are variations of the AASHTO T 99 method. In addition, California uses the term "relative compaction" instead of "density" and has its own method to determine the relative compaction (equivalent to AASHTO T 180). The moisture required to obtain the specified density is identified as either a percent below or above optimum and/or that moisture required to obtain the specified density. The density required in the specifications is normally higher in the top 3 ft of an embankment than below this depth. Density of embankments is also required to be higher at bridge approaches and specified distances away from bridges.

The frequency of field density tests during embankment construction is also identified by States in Table 6. The criteria for frequency and spacing of tests are variable, and four States rely on the project engineer's judgment. Other States use one of the following criteria: (a) a test for a given number of cubic yards of material placed, which may or may not be identified by material types; (b) a test per vertical interval and linear distance; (c) a minimum number of tests per day for each grading spread; or (d) a combination of (a) and (b).

Embankment side slopes. Side slopes were identified as a critical factor for constructing shale embankments. One State highway representative said that when shale embankments are constructed with 1V on 2H side slopes, some deformation is experienced within 3 years but that where the side slopes are constructed to 1V on 4H, a stable condition results. It was pointed out that some locations prohibit the construction of flatter slopes such as in urban areas where land cannot be acquired economically. Side slopes for shale embankments vary among the States and also may vary for soil or rock type construction. The shale placed as soil embankments has side slopes that vary from 1V on 1.5H to 1V on 6H. In many States, the lower the embankment the flatter the slope. Four States specify side slopes on the basis of embankment height ranges. Side slopes of 1V on 2H are the most commonly used; a normal range is 1V on 2H to 1V on 4H. The height of embankments within the States surveyed ranges from 4 ft to over 200 ft. The ranges and/or averages are identified by States in Table 6. Slopes for rock-fill embankments vary from 1V on 1.5H to 1V on 4H. However, most of the rock-fill embankments have slopes of 1V on 2H or 1V on 1.5H, and only two States require flatter side slopes. The range in height of shale placed as rock fill in embankments varies from 5 to 180 ft in the 16 States.

Remedial Treatment

Some remedial measures fall into the category of routine maintenance which may or may not be reported to the State office until the problem has reached a point beyond the capability of the maintenance crew. Table 6 presents a brief list of the remedial measures used by the States. A detailed discussion of remedial measures is presented in Volume 2 (Bragg and Zeigler, 1975). Two types of remedial measures are overlaying the problem area and removal and replacement of the failed material. The remedial measures that have been employed by one or more States are listed below.

a. Widening and/or flattening of embankment slopes.

b. Installation of surface and/or subsurface drainage measures.

c. Reconstruction involving removal and replacement of failed material.

d. Grouting (only a few cases and not used recently).

e. Addition of berms, buttresses, or retaining walls.

Virginia reported that an embankment was settling and pushing against a bridge abutment resulting in compression of the bridge superstructure. The temporary solution to date has been to raise and shorten the steel superstructure. Some States have investigated the problem areas either through their own resources or through contracts. These investigations also include recommendations for remedial measures to correct the problem.

Discussion

Problems with compacted shale embankments vary considerably within the 16 States. The State highway representatives identified causes that they felt contributed to shale embankment distress or failure. These include the following (which are not necessarily listed in order of importance):

a. Excessive lift thicknesses.

b. Inadequate compaction practices (specifications and control).

c. Shale deterioration (mechanical and/or chemical).

d. Expansive characteristics of shale.

e. Excessive steepness of side slopes.

f. Infiltration of water.

g. Lack of sidehill benching.

h. Inadequate drainage.

i. Random mixing of shale with harder rock types (limestone and sandstone, etc.).

j. Failure to consider all geological conditions.

k. Lack of established tests and criteria to reliably predict shale behavior after emplacement.

One or any combination of these factors could contribute to unstable conditions in compacted shale embankments, and one or more of these factors could contribute to or initiate the occurrence of others. Other unidentified factors could also be contributors to embankment problems.

The above causes except c, d, and k have rather obvious solutions, such as using flatter slopes, thinner lifts, increased compaction, and more extensive drainage. However, a requirement for the general use of all these measures may not be warrented and would be expensive, both to the design and construction of a project. Consequently, criteria are needed to determine when specific measures are actually required.

The most critical problem is the lack of tests and criteria to predict shale performance in all instances when shale is used as construction material. Because of the natural variability of shale, it is difficult to predict the alterations that will occur during excavation (ripping and/or blasting), hauling, dumping, spreading, and compaction. Further alterations can occur after construction through loading and surface erosion, infiltration of surface and groundwater, and shale deterioration. Mineral composition of shale has not been studied in detail to date, and some of the highway representatives have stated the need for such study.

Many of the States have developed special provisions for construction and compaction of shale embankments. How closely the provisions (especially lift thickness and density controls) are followed during construction was not determined. The present practice by some States of relying on the project engineer's judgment for density control is difficult to evaluate. More objective practices need to be considered. Also, the present requirements for constructing shale embankments are not believed to be sufficient to prevent all problems after emplacement. Embankments constructed with shale placed as soil fill and adequately compacted have not generally been a problem. Segregation of material types may not be practical for highway construction. However, some State highway representatives have reported that intermixing of shales with limestone and sandstone has resulted in embankment problems.

There are many variables in the construction of shale embankments that can result in unstable embankment conditions. The required data must be furnished prior to construction, and appropriate control must be established and adhered to during construction. The project engineer must be aware of potential problems and account for them during design investigations, construction, and postconstruction phases of highways that require building embankments out of shale. The States with problems are working to find solutions with such approaches as instrumenting and monitoring the embankments under distress, studying and using remedial measures, and modifying the specifications for future construction. Solutions to all problems have not been found, and some of the States identified their needs for assistance in finding ways of constructing stable shale embankments.

Corps of Engineers Experiences and Practices

Although relocation of highways and railroads (including costs for design and construction) is an integral part of many Corps of Engineers (CE) projects, the design and/or construction of the relocation is usually accomplished by the owner. When construction plans and specifications are prepared by the Corps, the standards and criteria of the owner are used. The main CE experience with compacted shale embankments has been in the construction of earth and rock-fill dams. Early experience in this field is summarized in this section first and is followed by a discussion of current practices pertinent to highway embankment design and construction. Recent data on the shear strength of compacted shale materials are summarized, and the usefulness of such data is discussed.

Early experience. Compacted shale materials were used in a number of CE earth and rock-fill dams constructed in the 1950's and 1960's. The material usage and compaction procedures for several of these dams are summarized in Table 7. The successful compaction and performance of the shale materials are attributed to the use of heavy compaction equipment and procedural (or method) type specifications developed for each material to produce a dense fill with adequate shear strength. For weathered or soft shale materials, which are easily broken down, thin

Table 7. Examples of compacted shale used in CE dam embankments (1949-1967).

State	Name of Dam (and Reference)	Year Completed	Location	Formations and (Age)	Material and Use	Compaction Procedure
PA	East Branch (WES EDCR No. 41)	1952	Clarion River near Johnsonburg	Allegheny Pottsville Mauch Chunk Pocono (Pennsylvanian & Mississippian)	Nondurable shale and weathered sandstone from spillway, crushed to 6-in. maximum size. One part crushed material mixed with 1-1/2 parts overburden soil used in US and DS random zones. DS slope varied from 1V on 2H to 1V on 3H.	Spread in 6-in. lifts. Compacted near optimum water content by six passes of tamping roller to 90% of T180 dry density (110 pcf minimum)
VA	John W. Flannagan (WES EDCR No. 18)	1963	Pound River near Haysi	Pennsylvanian age "Coal Measures"	Shale and weathered sandstone, sized to pass 8- by 16-in. opening, used in interior US and DS random zones.	Spread in 12-in. lifts. Compacted by six passes of 50-ton rubber tired roller.
VA	North Fork of Pound (WES EDCR No. 36)	1961	Pound River near Pound	Wise Gladeville Norton Lee (Pennsylvanian)	Shale, siltstone, and sandstone, sized to pass 8- by 16-in. opening used for US and DS random rock shells. DS slope of 1V on 2.3H.	Spread in 12-in. lifts. Compacted at natural water content (~6%) by 50-ton rubber tired roller
KY	Buckhorn (WES EDCR No. 10)	1960	Middle Fork Kentucky River near Buckhorn	Breathitt (Pennsylvanian)	Sound sandstone and shale used for US and DS random rock shells with outer slopes of 1V on 2.5H.	Spread in 4-ft lifts. Compacted by hauling and spreading equipment.
KY	Fishtrap (WES EDCR No. 66)	1967	Levisa Fork Big Sandy River near Pikeville	Wise Gladeville Norton Lee (Pennsylvanian)	Shale, siltstone, and sandstone mixture used for US and DS random rock zones. DS slope 1V on 2H above el 770.	Spread in 24-in. lifts and compacted at natural water content by three passes of 50-ton rubber tired roller. D-8 tractor used to break accumulations of 12-in. shale chunks into gravel sizes.
KY	Grayson	1967	Little Sand River near Grayson	Breathitt (Pennsylvanian)	Shale, siltstone, and sandstone used in DS random rock zone with 1V on 3H to 1V on 3.5H outer slope.	Spread in 12-in. lifts. Compacted by three passes of 50-ton rubber tired roller.
IN	Monroe	1964	Salt Creek near Bedford	Harrodsburg LS Edwardsville SiS (Mississippian)	Limestone and shaley siltstone used in US and DS random rock zones with 1V on 3H outer slopes.	Spread in 12-in. lifts. Compacted by four passes of 50-ton rubber tired roller.
KA	Milford	1966	Republican River near Junction City	Towanda LS Holmsville Sh Ft. Riley LS (Permian)	Shale and limestone used in US shell with 1V on 4.5H outer slope.	Spread in 18-in. lifts. Compacted by two or more passes of 50-ton rubber tired roller. Oversize removed or broken down by spike tooth roller or 30-ton tractor.
KA	Tuttle Creek (WES EDCR No. 4; Lane and Fehrman, 1960; Bennett, 1958)	1961	Big Blue River near Manhattan	Stearns Sh Cottonwood LS Eskridge Sh Neva LS (Permian)	Shale-limestone mixture used in several zones of US and DS shells. DS slope 1V on 3H above berm.	Spread in 18-in. lifts. Oversize (+18 in.) removed and remaining chunks broken down with spike tooth roller. Compacted by three passes of 50-ton rubber tired roller.
KA	Wilson (WES EDCR No. 39)	1965	Saline River near Wilson	Greenhorn Graneros Dakota (Cretaceous)	Soft shale, lean clay, sandy silt, and silty sand used for interior random fill section.	Spread in 6-in. lifts and compacted by six passes of tamping roller, or spread in 12-in. lifts and compacted by three passes of 50-ton rubber tired roller.
OK	Wister (WES 1959)	1950	Poteau River near Wister	McAlester Sh Hartshorne SaS Atoka Sh (Pennsylvanian)	Harder shale and sandstone used in US and DS berms. Weathered and softer shale used in DS section for a 100% shale zone (1V on 2.75H outer slope) and underlying 20% shale zone.	Berms: Spread in 3-ft lifts; shale broken down with a spike tooth roller; compacted by hauling and spreading equipment. DS sec: Spread in 6-in lifts, watered and disked, compacted by eight passes of tamping roller.
SD	Oahe (WES EDCR No. 42)	1962	Missouri River near Pierre	Pierre Sh (Cretaceous)	Shale used in US and DS flat berms with 1V on 15H outer slope.	Spread in 18-in. lifts. Compacted by controlled routing of hauling and spreading equipment
ND	Garrison (WES EDCR No. 54)	1954	Missouri River near Riverdale	Tongue River (Tertiary)	Soft shale used in random zones, 1V on 2.5H steepest DS outer slope.	Chunky shale (several cubic feet) broken down with spike tooth roller (to 8-in. maximum size) Water added on fill to restore natural water content. Grader used to smooth surface. Compacted in 8-in. lifts with eight passes of tamping roller.

Note: LS = limestone
Sh = shale
SiS = siltstone
SaS = sandstone
US = upstream
DS = downstream

lifts (6 or 8 in.) and a heavy tamping roller were used. For harder shale materials containing chunks and fines, thicker lifts (12 to 24 in.; maximum particle size limited to two-thirds of the lift thickness) and 50-ton rubber-tired rollers were used. For the sound sandstone and shale rock (based on slaking tests) at Buckhorn Dam, 4-ft lifts were used. At Wister Dam, 3-ft lifts of shale were used in the flat berms which were not critical to the stability of the embankment slopes. It is of interest to note that only one of the problem shales listed in Table 4, the Pierre, appears in Table 7.

The specifications for each material stipulated the compaction equipment, lift thickness, number of passes, and water content limits. For shale materials easily broken down, these procedural type specifications were designed to obtain a desired percentage (usually 95) of standard effort maximum density (equivalent to AASHTO T 99). Laboratory compaction and shear strength tests were used to develop the desired percent compaction and required water content limits (related to optimum water content) to produce the required design strength. Field density test results, compared with appropriate laboratory compaction results, were used to check the adequacy of field compaction during construction. For harder materials containing numerous large chunks, test fills (often in conjunction with a test quarry) were used during design or at the start of construction to develop the compaction procedures. Test pits were dug to visually determine whether a dense, well compacted fill was being obtained. Several large-scale field density tests were usually performed to determine in-place dry densities for use as a check during construction. The construction of test fills at Tuttle Creek Dam, and the importance of (a) breakage during blasting, (b) conditioning the fill with a shale breaker roller, and (c) removal of oversize chunks as described by Bennett (1958) and Lane and Fehrman (1962) provide a valuable insight on compaction of shale materials. An extract from Lane and Fehrman (1962) is reproduced in Appendix C.

Current practices. Guidance and procedures for field investigations, field tests, laboratory tests, design studies and analyses, construction control, and performance evaluation are contained in a number of Engineering Manuals (EM) listed in Table 8. Although a number of these manuals (recently published or revised) are primarily for earth and rock-fill dams, the principles and tests procedures are applicable to highway embankments.

Geology and materials investigations. The need to use all possible materials from required excavations, especially for large spillways, has increased the use of shale materials in recent CE earth and rock-fill dams in areas such as the Appalachian region. This increased use of shale emphasized the importance of a thorough investigation to determine the location, extent, quantity, and physical properties of the soil and rock strata in the spillway area.

A knowledge of the regional and local geology is essential in developing a plan of subsurface investigations for a site to interpret conditions between and beyond boring locations and to reveal possible sources of trouble. General requirements for geologic and subsurface investigations are given in EM 1110-2-2300, and geological investigations are discussed in EM 1110-1-1801.

The extent of the foundation exploration program is governed principally by the complexity of the geology and size of the project. Exploration of the borrow and excavation areas is accomplished early in the field investigations program under the direction of the project soils design engineer and project geologist. This early evaluation enables the quantity and properties of the soil and rock available for embankment construction to be determined before detailed studies of embankment sections are made. Geophysical investigations and subsurface investigations for soils are described in EM 1110-2-1802 and EM 1110-2-1803. Soil sampling equipment and procedures are described in EM 1110-2-1907 and in Hvorslev (1948).

Subsurface explorations of required excavations in shale materials proposed for use in an embankment are usually accomplished by continuous drive sampling of the overburden (using a standard split-spoon sampler) and continuous NX-size rock core drilling. In relatively firm soils, split-spoon drive samples and blow counts are sometimes obtained using a 300-lb hammer falling 30 in. instead of the standard 140-lb hammer

Table 8. List of applicable CE EM's.

EM No.	Title*	Date
1110-1-1801	Geological Investigations	1 Nov 1960
1110-2-1802	Subsurface Investigations Geophysical Explorations	Sep 1948
1110-2-1803	Subsurface Investigations-Soil	Mar 1954
1110-2-1902	Stability of Earth and Rock-Fill Dams	1 Apr 1970
1110-2-1906	Laboratory Soils Testing	30 Nov 1970
1110-2-1907	Soil Sampling	31 Mar 1972
1110-2-1908	Instrumentation of Earth and Rock-Fill Dams; Part 1: Ground Water and Pore Pressure Observations	31 Aug 1971
1110-2-1911	Construction Control for Earth and Rock-Fill Dams, Draft	15 May 1972
1110-2-2300	Earth and Rock-Fill Dams, General Design and Construction Considerations	1 May 1971
1110-2-3504	Chemical Grouting	31 May 1973
1110-2-3800	Systematic Drilling and Blasting for Surface Excavations	1 Mar 1972

* Listed in References under OCE.

falling 18 in. NX core drilling is performed using standard double-tube rotary core barrels (5- and 10-ft-long core barrels) and bottom discharge bits. Maximum spacing of borings or core holes is of the order of 200 ft. An example of a boring plan for a spillway site is shown in Figure 20. In this example, core borings along the center line of the spillway (Figure 21) extended 20 ft into the interbedded siltstone and sandstone considered suitable for the spillway foundation. The type of information obtained is shown on example boring logs and a section in Figures 21 and 22. Special 6-in.-diam cores are often obtained of the rock (shale and sandstone in this example) to provide material for laboratory testing. All cores not selected for laboratory tests are retained in their core boxes and stored at the project site until the project is completed and any claims are settled.

Overburden and borrow area soils for impervious compacted fills are investigated by test pits and auger borings. Jar samples are taken continuously with depth to determine visual classification, soil type, areal extent, and natural water content. Analyses for gradation and Atterberg limits are determined on representative samples of each soil type. Bag samples of each major soil type (often composite samples of similar soils from comparable depths in different test pits) are obtained for visual classification, for tests of specific gravity and Atterberg limits, for grain-size analyses, for standard effort compaction tests, and for triaxial shear tests. Test procedures for these tests are given in EM 1110-2-1906.

The information presented in the remainder of this section deals mainly with shales east of the Mississippi River. Recent CE earth dams constructed with shales in the western States have involved weaker bonded shales which tend to break down and are treated as soil fill. Use of instrumentation and observations during and after construction of embankments is discussed in Volume 2 (Bragg and Zeigler, 1975) and is not included in this section.

Laboratory tests on shales. Laboratory tests performed on shale proposed for compacted fill include the following:

a. Petrographic analyses.

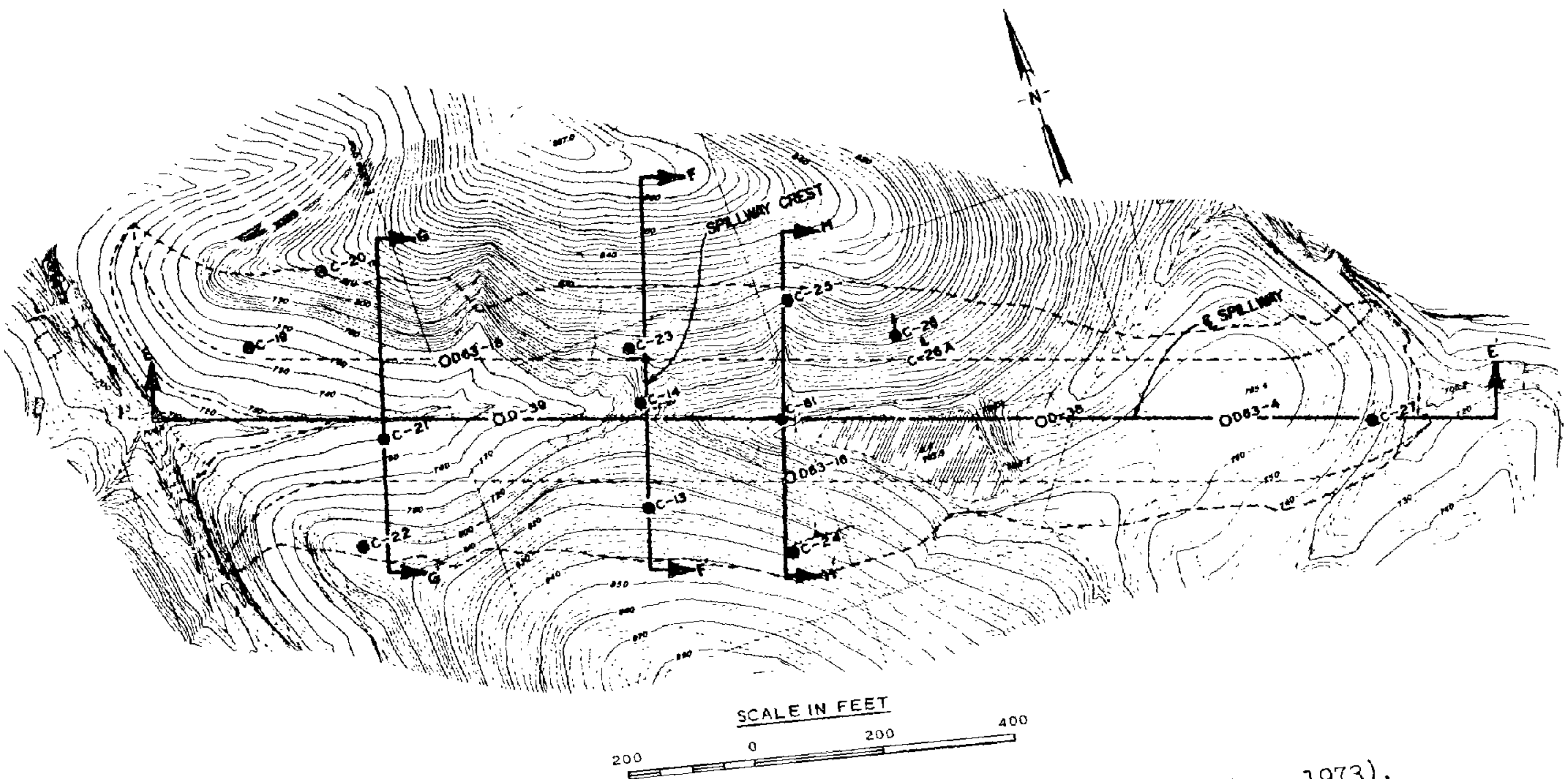

Figure 20. Example of boring plan for spillway site (USAE, Huntington, 1973).

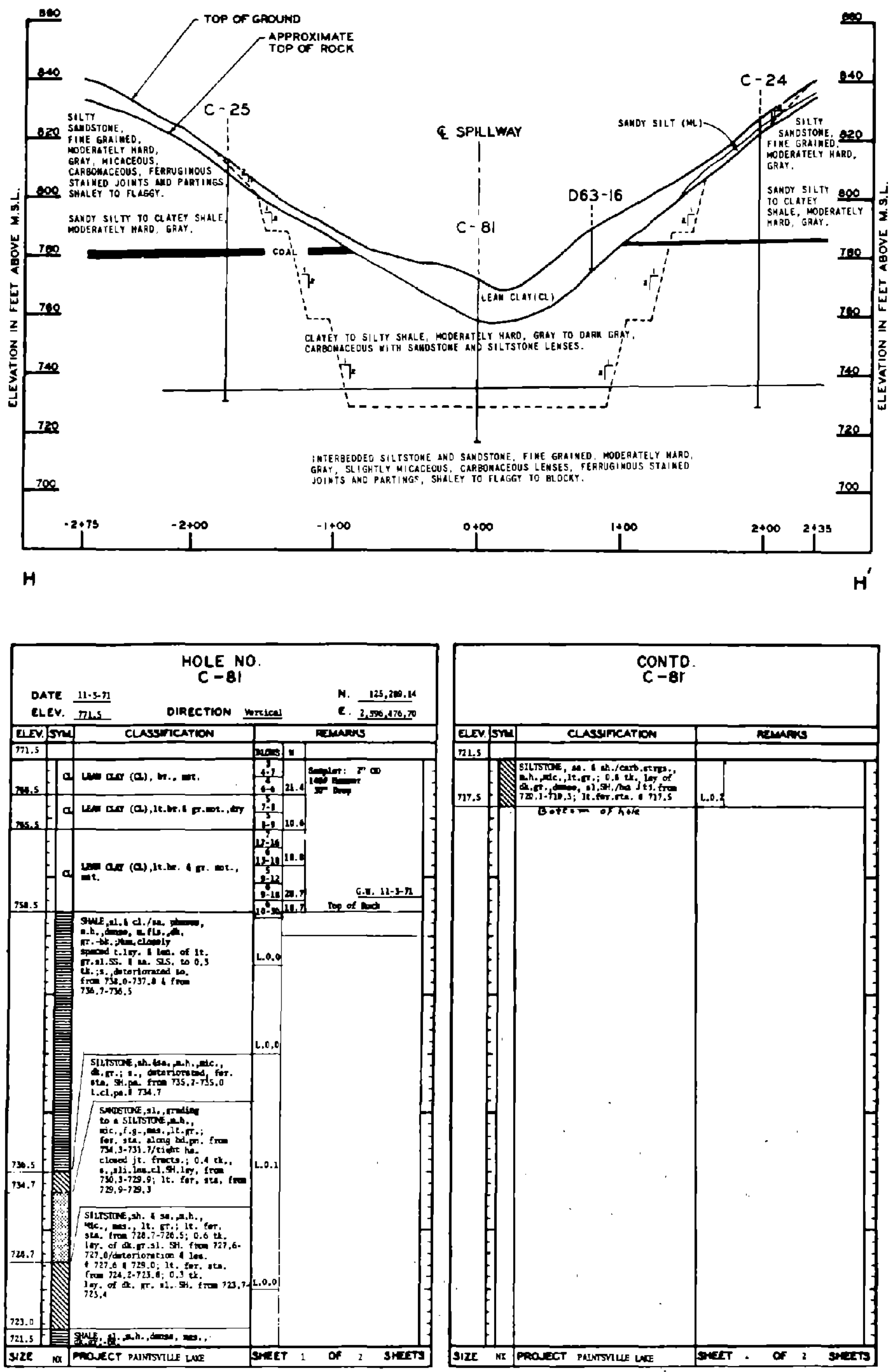

Figure 21. Example of spillway section and NX boring log (USAE, Huntington, 1973).

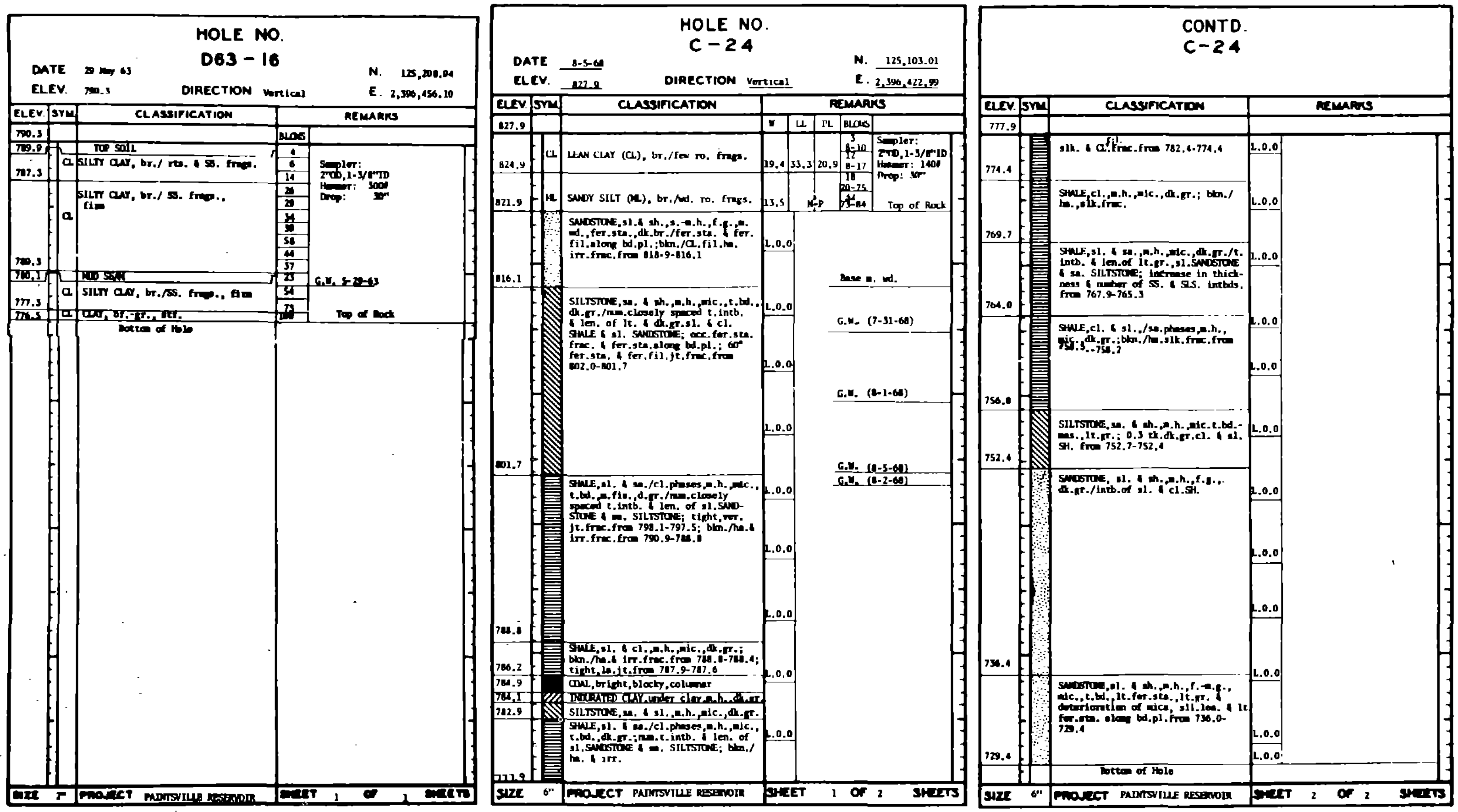

Figure 22. Example of 2-in. drive sampler boring log and 6-in. core log (USAE, Huntington, 1973).

b. Wet-dry weathering tests.

c. Gradation tests.

d. Specific gravity of solids.

e. Atterberg limits tests.

f. Large-scale compaction tests.*

g. Permeability tests.

h. Large-scale triaxial tests.*

Petrographic analyses (in special cases also mineralogical studies and cation exchange capacity tests) and weathering tests are performed on representative cores to aid in the evaluation of durability. Wet-dry weathering tests are performed on either NX or 6-in.-diam cores preserved at their natural water content in plastic sheeting. These tests have been performed at the U. S. Army Engineer Division, Ohio River (ORD), Cincinnati, Ohio, by alternately soaking cores in water at 50°F for 6 hr and drying under infrared lamps at 140°F for 6 hr. The tests are conducted in a concrete tank with the cores resting on wire frames. The test tank is flooded and dewatered and the heat lamps turned on and off automatically. This setup enables two cycles of wetting and drying to be completed each 24 hr. Photographs taken periodically during the tests indicate the type and degree of weathering and provide a qualitative basis for evaluating the suitability of the shale for use as rock fill. These tests have been used for several projects in Kentucky, Ohio, West Virginia, and Pennsylvania. Because fairly severe weathering has often occurred after 4 to 12 cycles, the shales or shale and sandstone or limestone mixtures have generally been restricted to nonfree-draining compacted random-rock zones.

A similar type of wetting and drying test is performed in the U. S. Army Engineer Division Laboratory, South Pacific (SPD), at Sausalito, Calif. Samples are soaked for 16 hr at room temperature and dried at 140°F for 8 hr.

The size of current large-scale compaction and triaxial compression

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* Large-scale compaction and triaxial tests are used mainly for stronger shales east of the Mississippi River.

test apparatus located at two CE division laboratories limits the maximum particle size for test specimens as indicated below.

Division Laboratory	Specimen Diameter for Test Apparatus	Maximum Particle Size, in.
South Atlantic (SAD)	15-in. triaxial	3
	12-in. compaction	2
South Pacific (SPD)	12-in. triaxial	2
	12-in. compaction	2

Materials for these tests are usually obtained by crushing NX or 6-in.-diam cores to a maximum size of 3 in. (or 2 in. at SPD) in a laboratory jaw crusher or by removing the oversize rocks from material obtained from test fills constructed during design studies. The gradation of the as-received material is determined using a large-sieve shaker and 12 to 14 different sieves (3 in. down to the No. 200 sieve). This gradation or a gradation specified by the soils design engineer is used as a basis for segregating the material (2000 to 3000 lb) into different size fractions (down to No. 4 sieve) for use in reconstituting samples having the same gradation. The gradation after a test is determined to evaluate particle breakdown during the test. The gradation test is usually determined by first separating the sample on a No. 4 sieve. A sieve analysis (5 min of shaking) is performed on the plus No. 4 ovendry material. The minus No. 4 fraction is washed over a No. 200 sieve, dried, and a sieve analysis performed. The retained portion on the No. 200 sieve is often soaked overnight and then dried before its gradation is determined. The separate gradation results are used to calculate the total sample gradation.

Atterberg limits tests, as described in EM 1110-2-1906, are determined on the minus No. 40 sieve material using a process of grating or crushing, slaking, and blenderizing to disaggregate shale into basic particle sizes (i.e. silt and clay sizes). The wide range in Atterberg limits that can be obtained by different processing methods is described by Townsend and Banks (1974). The specific gravity of solids is determined (in accordance with EM 1110-2-1906) for the plus No. 4 sieve and minus No. 4 sieve fractions separately, and a weighted average is used.

Large-scale compaction tests are performed in a 12-in.-diam mold using a standard effort (AASHTO T 99) hand-compaction method as described in EM 1110-2-1906. The maximum particle size is 2 in. with replacement when the plus 2-in. material exceeds 10 percent of the total sample weight. Otherwise, the plus 2-in. material is scalped. Usually, a complete compaction curve is developed. However, in some project studies, only a one-point test is performed at the expected field compaction water content (generally when the shale is to be compacted at its natural water content). The design dry density for field compaction is usually 98 percent of standard effort maximum dry density. Constant head, vertical permeability tests are performed on compacted samples. The results are used to estimate the degree of drainage to be expected in the field (free draining, semipervious, or impervious).

Large-scale triaxial compression tests are performed on 15-in.-diam by 32-in.-high specimens in the SAD (12-in.-diam specimens in SPD) compacted to the design density and expected field compaction water content (usually the natural water content for shales). Specimens are compacted by using the same procedures as for the standard compaction test but reducing the number of blows as the specimen is built to achieve a uniform density. Three types of controlled strain tests are usually performed.

a. Unconsolidated-undrained (Q) test to determine the shear strength for conditions of no drainage during or at the end of construction (short term) for relatively impervious materials and total stress conditions.

b. Consolidated-undrained (R) test to determine the shear strength after consolidation of relatively impervious materials for load increases without further drainage (e.g. increase in the embankment weight).

  (1) Without back pressure saturation (R test) for field conditions where saturation will not occur (e.g., area in an embankment above a drainage blanket).

  (2) With back pressure saturation ($\overline{R}$ test) for field conditions where saturation will occur (e.g., increase in the water level in an embankment). Pore pressure measurements are usually made to determine effective stresses during the test.

c. Consolidated-drained (S) test to determine the drained strength of relatively free-draining materials.

For some shales, an S triaxial compression test is impractical because of the impervious nature caused by a high percentage of silt and clay sizes. In this case, standard direct shear tests (in accordance with EM 1110-2-1906 using 3- by 3-in. specimens) are performed on the minus No. 4 fraction compacted to the design density.

Because large-scale triaxial tests are expensive, usually only a few are performed. The zone of placement in the dam (i.e., expected saturation conditions) and drainage characteristics of the shale determine the type of test (see page 87). Composite samples of shale and other rock are tested where a mixture is to be used in a zone of the dam. When separate zones of different material types are feasible, a separate series of tests is performed on each material.

Shale test fills. When previous experience is not available on the compaction characteristics of a particular shale or mixture of shale and other rock, test fills are constructed to develop adequate compaction methods and control procedures. Data for 14 CE rock test fill projects, including three involving shale, are summarized by Hammer and Torrey (1973). Six of the 14 are analyzed in detail and include shale test fills for Beltzville Dam, Penn. This study recommends procedures for test fills covering (a) planning and design, (b) construction, (c) measurements and observations, and (d) evaluation of results. It also includes a detailed procedure recommended for large in situ field density tests using a displacement method with water and plastic sheeting.

Because large-scale compaction tests and field density tests are expensive and time-consuming, only a limited number are used to aid in establishing the field compaction method. Cumulative settlement measurements and test trenches to observe denseness, interlocking, and distribution of coarse rock and fines are the main techniques for evaluating the variables studied in rock test fills (rock types, mixtures and gradations, lift thicknesses, types of compaction equipment, and number of passes).

For relatively strong nonshale rocks, a vibratory roller generally

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results in the best compaction. However, this type roller also causes substantial surface breakage for most rock types. Better results have been obtained with sizes less than about 3 in. removed. For weaker rocks, including most shales, the best results are obtained with a heavy tamping roller to facilitate breakdown (especially when shale and limestone or shale and sandstone mixtures are used) followed by compaction with a 50-ton rubber-tired roller. Using a heavy tracked dozer (D-9 or equivalent) to tow the roller helps break down and push larger pieces into the lift being compacted.

At Beltzville Dam, Penn., test fills were constructed using partly weathered shale and relatively unweathered shale. Surface breakage of these materials for different types of compaction equipment is summarized in Tables 9 and 10. The gradations of the material used are summarized in Figure 23 which illustrates the large breakdown during compaction. Results of field density tests summarized in Table 11 illustrate the significant increase in dry density for 12-in. lifts with increases in the compaction effort. The nuclear method (back scatter apparatus) indicated somewhat higher water contents and lower dry densities (depending on the roughness of the compacted surface) than those from field density tests using the 36-in. template (for volume measurements). The effectiveness of the different compaction procedures is shown in Figure 24 for the two types of shales.

Based on the rock test fill results, the specifications for compaction of the shale at Beltzville called for the following:

a. Loose lift thickness of 12 in.

b. Partially weathered shale (for relatively impervious interior zones) to be compacted by two passes of a shale breaker followed by four passes of a 50-ton rubber-tired roller.

c. Relatively unweathered shale (for relatively free-draining outer shells) to be compacted by two passes of a 10-ton vibratory roller.

Pertinent information for the Beltzville test fills and test fills for three other projects, summarized in Table 12, indicate the predominant use of the 50-ton rubber-tired roller. Compaction with this heavy roller is usually preceded by processing the material or by breakdown

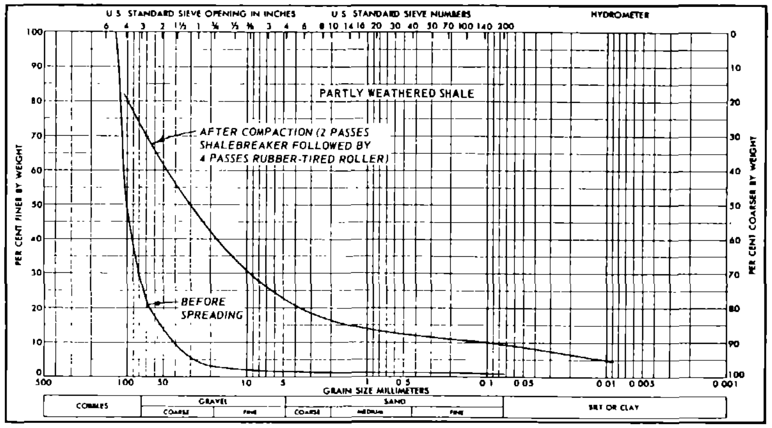
Table 9. Visual evaluation of particle breakage, Beltzville test fill.

Operation	Breakage Evaluation
Partly Weathered Shale	
Spreading with tracked bulldozer	Extensive surface breakage
Compaction with shale breaker roller	Larger pieces reduced
Compaction with rubber-tired roller (following shale breaker)	Additional breakage on surface
Relatively Unweathered Shale	
Spreading with tracked bulldozer	Limited surface breakage
Compaction with rubber-tired roller	Limited surface breakage
Compaction with vibratory roller	Extensive surface breakage

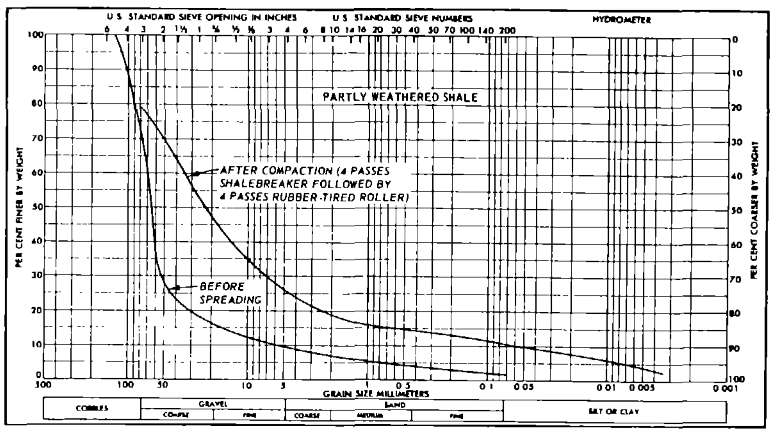
Table 10. Summary of compaction equipment, Beltzville test fill.

Towing Speed mph	Compaction Equipment		
	Type	Make and Model	Data
3	Shale breaker	Ferguson-Gebhard Model 22	Chisel tips with 1.5-sq-in. face area, 3,750-psi tip pressure
3	50-ton rubber-tired roller	Bros 50 ton	Four pneumatic tires, 25,000-lb wheel load
1-1/2	10-ton steel-vibratory roller	Ferguson Model 230	23,500-lb static weight; operating frequency of vibration varied from 1,100 to 1,300 vibrations per minute
	Tracked bulldozer	Allis-Chalmers HD-20	Not given

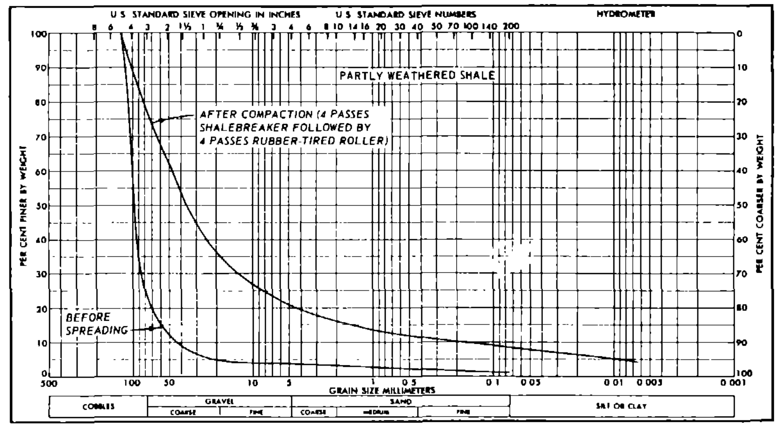
Note: Allis-Chalmers HD-20 tracked bulldozer used for spreading and towing.



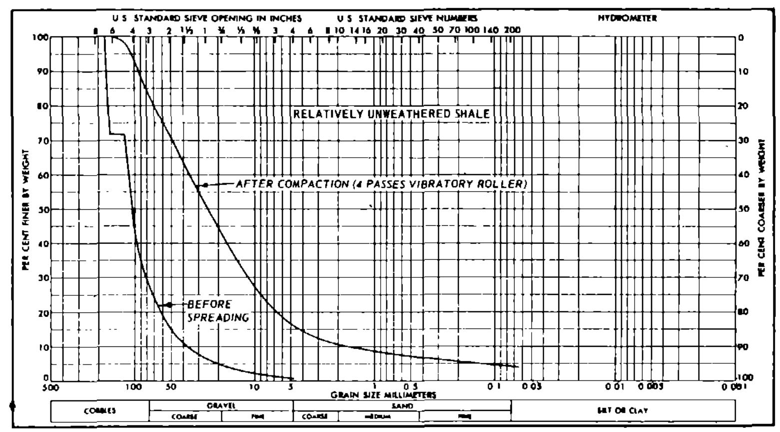
a. TEST ZONE C, AREA 19, LIFT 3



b. TEST ZONE C, AREA 21, LIFT 2



a. TEST ZONE C, AREA 22, LIFT 1



b. TEST ZONE D, AREA 23, LIFT 3

Figure 23. Gradation of shale, Beltzville test fills.

Table 11. Beltzville test fill, field density test data.

Test Area	Compaction	No. of Lifts	Lift Thickness in.	Field Density Test Method	Water Content w, %	Dry Density pcf
Zone C, partly weathered shale						
19	2 passes shalebreaker plus 4 passes 50-ton rubber-tired roller	4	12	36-in. template* - sand	8.7	119.7
				36-in. template - water	9.0	122.4
				Nuclear	13.8	105.0
				Nuclear	12.0	112.0
20	4 passes shalebreaker plus 2 passes 50-ton rubber-tired roller	4	12	Nuclear	11.6	107.5
21	4 passes shalebreaker plus 4 passes 50-ton rubber-tired roller	4	12	36-in. template - sand	8.9	120.0
				36-in. template - water	8.6	126.6
				15-in. template - sand	8.8	117.8
				Nuclear	10.0	109.5
				Nuclear	13.6	99.0
22	4 passes shalebreaker plus 4 passes 50-ton rubber-tired roller	4	12	36-in. template - sand	10.5	123.8
				Nuclear	11.8	102.0
Zone D, relatively unweathered shale						
23	4 passes vibratory roller	3	18	Nuclear	9.2	125.0
24	4 passes vibratory roller	4	12	36-in. template - sand	3.9	127.1
				Nuclear	8.8	125.0
25	2 passes dozer plus 2 passes 50-ton rubber-tired roller	2	24	Nuclear	11.3	111.5
26	2 passes dozer plus 2 passes 50-ton rubber-tired roller	3	18	Nuclear	8.2	110.5

* Steel frame used in the volume measurement of the excavated hole.

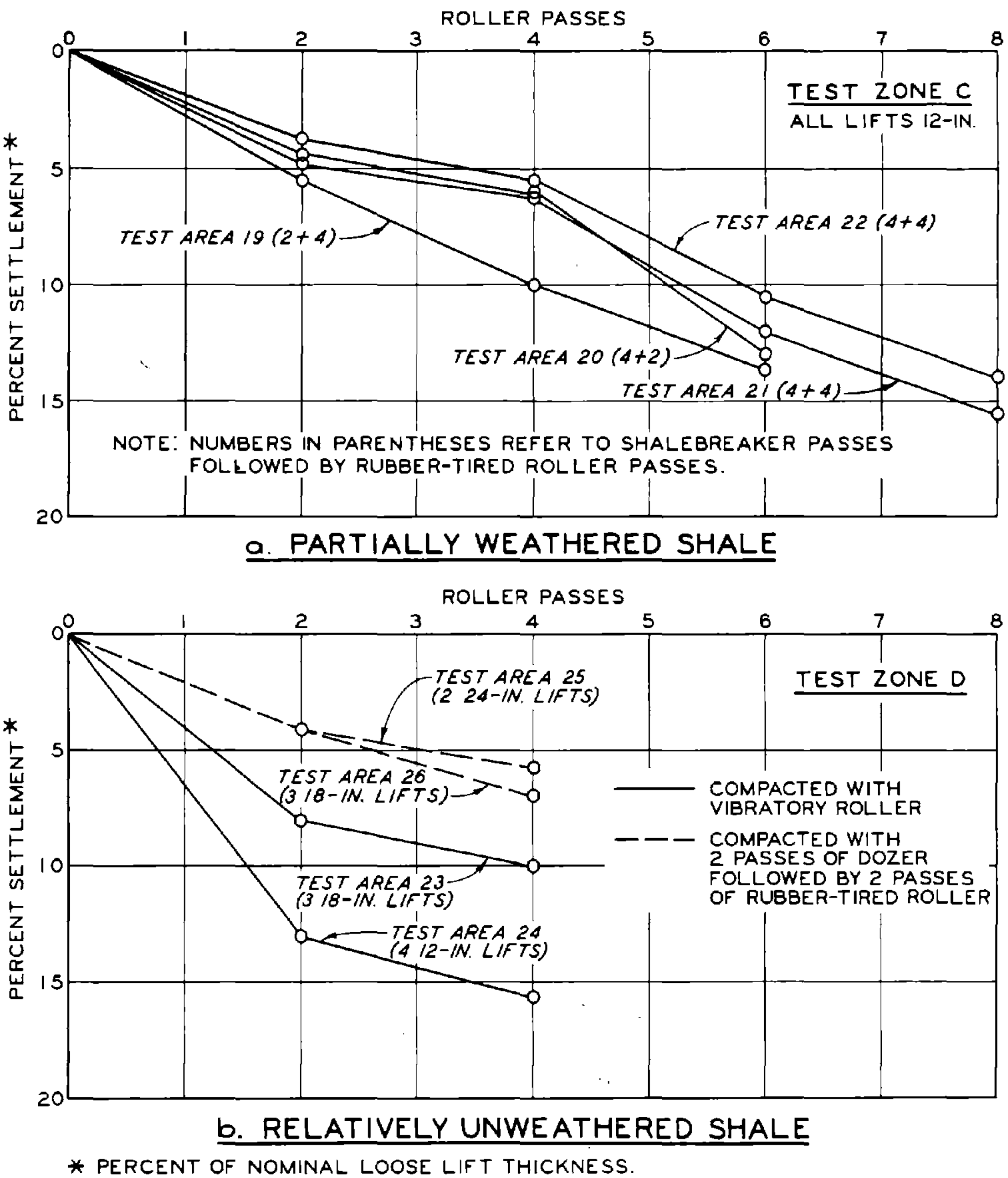


Figure 24. Comparison of percent settlement and compaction effort, Beltzville Dam.

Table 12. Summary of CE shale test fills.

Dam and Location	Beltzville, Leigh River, PA	Raystown, Juniata River, PA	Bloomington, Patomic River Basin, MC and WV	R. D. Bailey Lake Project, Guyandot River, WV
ROCK DATA				
Formation and (age)	Trimmers Rock and Mahantango (Devonian)	Catskill (Devonian)	Conemaugh series, Franklin coal measures (Pennsylvanian)	Kanawha (Lower Pennsylvanian
Type material used in test fills	(1) Brown-green-gray, partly weathered shale (2) Dark gray, dense, relatively unweathered shale	(1) Sandstone (2) Siltstone (3) Shale, various mixtures used	(1) Siltstone, shale, and sandstone (2) Shale and claystone (3) Shale, claystone, and some siltstone	(1) Weathered shale (2) Unweathered shale (sandy with sandstone stringers)
Gradation used in test fill	(1) Minus 5 in., 10% minus No. 4 (2) Minus 8 in., 5% minus No. 4	Minus 30 to 10 in., 5 to 15% minus No. 4	(1) Unprocessed. minus 15 in., processed: plus 3 in. and minus 3 in. (2) Unprocessed: minus 12 in. (3) Unprocessed: minus 12 in., processed: plus 3 in. and minus 3 in.	Maximum size approximately 16 in.
Breakage during rolling	Limited to extensive, depending on type of roller	(1) Blocky pieces (2) and (3) Small, blocky plus numerous fines	Limited	Extensive
Gradation after compaction	Minus No. 4 increased to 15 to 25%	Minus 18 to 8 in., 5 to 25% minus No. 4	Moderate increase in No. 4 fraction	Minus 9 in. with approximately 45% minus No. 4
COMPACTION EQUIPMENT				
Type, make, and model	Shale breaker. Ferguson-Gebhard Model 22; 50-ton RTR. Bros., 10-ton vibratory roller' Ferguson Moderl 230	D-9 crawler tractor, Cat.; 50-ton RTR: Bros.; 10-ton vibratory roller: Bros.; 32-ton truck. Euclid	D-9 dozer, 5-ton RTF grid roller 3.5 to 7.5 ton (to condition minus 3-in. material)	Tamping roller (TR), 10-ton vibratory roller (vib): Buffalo-Bomag, 35-ton truck, loaded, Cat. 769B end dump
Data	Shale breaker: chisel tips with 1.5 $in.^2$ face, 3750 psi tip pressure, 50-ton RTR: four tires, 25,000 wheel load 10-ton vibratory roller: 23,500 lb weight, 1100 to 1330 vib/min	D-9. 41-ton, 1640-psf track pressure; 50-ton RTR: four tires, 25,000-lb wheel load; 10-ton vibratory roller: 24,000 lb weight, 1400 vib/min	Not reported	Not reported
Rolling speed, mph	1 5 to 2.0 estimated	Not reported	Not reported	Not reported
COMPACTION				
Uncompacted lift thickness	12, 18, 24	8, 12, 18, 24, 36	4, 8, 12, 24	8 (tamping roller) 12 (vib. roller and truck)
Number of coverages	2 to 8	2 to 8	2, 4, 6	3 (tamping and vib. roller) 6 (truck)
FIELD DENSITY TESTS				
Method	3- by 3-ft template with sand or plastic sheet and water to measure volume	6-ft-diam ring with plastic sheeting and water to measure volume	3- by 3-ft or 6- by 6-ft template, plastic sheeting and water to measure volume	4-ft-diam by 36-in.-deep pit, lined with plastic sheeting and filled with water to measure volume
No. of tests and average or range of dry density, pcf	(1) 6, 122 pcf (2) 1, 127 pcf	24, 115 to 139 pcf	(1) 9, 127 pcf (2) 1, 130 pcf (3) 4, 130 pcf	TR 2, 116 and 124 pcf Vib 1, 124 pcf Trk 1, 126 pcf
SETTLEMENT DATA	Settlement vs No. of passes (foundation settlement also measured)	Settlement vs No. of passes; settlement vs layer thickness	Not reported	Not reported
SPECIFICATION PROCEDURES	(1) 12-in. lifts, 2 passes of shale breaker followed by 4 passes of 50-ton RTR (2) 12-in. lifts, 2 passes with 10-ton vibratory roller	Processed into plus 4 in. and minus 4 in., plus 4 in.. 24-in. lifts by D-9 dozer compacted by hauling and spreading equipment; minus 4 in.: 12-in. lifts, 4 passes, 50-ton RTR	Unprocessed minus 18 in.: random fill 24-in. lifts, 4 passes, 50-ton RTR	Spread with D-9 dozer, 12 in. lifts, 2 passes tamping roller followed by 4 passes of 50-ton RTR, maximum size rock ≤ 12 in.

Note: Numbers in parentheses refer to material type.

with a tamping roller or shale breaker.

Embankment design considerations. The primary objective of embankment design is to develop the most economical cross section having adequate stability during and after construction. Achieving this objective requires a study of different trial sections for zoning the embankment to obtain the maximum and best use of available materials, especially from required excavation, considering the sequence of availability.

Geologic information important to embankment design includes the following:

a. Groundwater and seepage conditions in the foundation and abutments.

b. Lithology, stratigraphy (including variations in thickness), and geologic details disclosed by borings and geologic interpretation (important in determining the quantities and sequence of different materials in excavation areas).

c. Structure, including bedding, folding, jointing (amount, systems, and character), and faulting (important in estimating fragmentation during blasting).

d. Depth of weathering and effect of weathering on exposed outcrops of sedimentary rocks.

e. Field evidence relating to slides, movement along bedding and faults, tension jointing, and earthquake activity.

Drainage zones (such as inclined drains and blanket drains) are an important feature of the design and are included to control seepage and prevent excessive pore water pressures, piping, sloughing, removal of material by solution, and erosion of material by loss into cracks, joints, and cavities. Transition or filter zones are also included between materials of widely differing gradations to prevent erosion (or piping) of finer material into coarser drainage material and possible clogging of drainage zones. The importance of drainage features in remedial work on compacted shale highway embankments is discussed in Volume 2 (Bragg and Zeigler, 1975).

Results of stability analyses (using procedures in EM 1110-2-1902) are used to compare the relative merits of trial cross sections and evaluate the influence of foundation conditions, as well as the effects of possible changes in material properties during and after construction.

The value of stability analyses depends on the validity of selected design shear strengths, and results are reviewed for compatibility with analyses for other embankments of similar materials where construction and operating experience are known. The final choice of embankment slopes is influenced by the (a) strength and compressibility of the foundation, (b) cost for removal of strengthening of weak foundation materials as opposed to the use of berms, and (c) total quantity of each type of available embankment material.

The intended use of compacted shale or shale and limestone (or sandstone) mixtures in a particular embankment zone depends on the shale durability (evaluated from wetting and drying tests), breakdown during compaction in test fills and/or laboratory tests, and permeability. The compaction specifications are designed to achieve a greater density (usually 98 percent of standard effort maximum density) than is normal for soils to insure adequate shear strengths. For shales which tend to break down or deteriorate, the blasting and excavation procedure, processing and/or prerolling with a tamping roller or shale breaker, and the specified compaction method are designed to produce a dense relatively impervious material. The use of shale or shale and rock mixtures for compacted free-draining rock-fill embankments is limited to shales expected to be durable and which do not break down significantly during compaction in test fills.

Design shear strengths. Only a limited number of large-scale compaction and triaxial shear tests are economically feasible for shale and composite mixtures of shale and other sedimentary rocks. This limited testing makes it imperative that high densities be attained during construction of the embankment to insure adequate strengths for stability.

For shales used as compacted rock fill or as relatively impervious compacted random fill in zones above the expected seepage line, the design shear strengths are conservatively estimated from Q, R, and S triaxial compression tests on specimens tested at the expected placement water content. For relatively impervious zones below the expected seepage lines, the R and S strengths are conservatively estimated from triaxial compression tests with back pressure saturation. The S strength is often determined using effective stresses from R tests with back pressure saturation and pore water pressure measurements. For relatively impervious shale materials, the S strength is determined from direct shear tests on the minus No. 4 fraction. The design shear strengths for some projects are also based on laboratory tests and construction and performance experience with similar shales used in other projects.

Design shear strengths are used in the form of Mohr strength envelopes for stability analyses. As outlined in EM 1110-2-1902, composite strength envelopes are used in stability analyses for cases other than the end-of-construction case. An example is the steady seepage case where the S and R strength envelopes are combined to form a composite envelope with the S strength used at effective normal stresses below that at the intercept of the two envelopes. An envelope intermediate between the S and the R strengths is used for effective stresses above that at the intercept of the two envelopes. When R strengths are determined from triaxial compression tests with back pressure saturation and the measured pore water pressure (during shear) decreases below the initial value, the resulting high cohesion intercept is unrealistic and is one reason that the S strength is used at low normal stresses.

Data on rock cores. Readily available test data on shale cores was extracted from several CE design memoranda to compare formation ages, unconfined compressive strengths, and results of wet-dry weathering tests. The limited data summarized in Table 13 indicate only general trends.

a. For younger age (Tertiary to Jurassic) shales at two projects (Warm Springs and Chatfield), the unconfined compressive strengths were lower than those for the older (Pennsylvanian, Mississippian, and Devonian) harder and silty to sandy shales. However, older (Ordovician) shales interbedded with limestone (Brookville, Caesar Creek, and East Fork projects) also had low unconfined compressive strengths.

b. The results of wet-dry tests indicated initial separation of cores along bedding and varying degrees of deterioration unrelated to age or unconfined compressive strength. In one case (Raystown project), shales of relatively high unconfined compressive strength disintegrated after a long period of 50 cycles of wet-dry testing.

Table 13. Summary of shale rock core data from selected CE earth and rock-fill dams.

Project	Formation and (Age)	Description	Sampled Material	Unconfined Compressive Strength psi	Modulus E 10^6 psi	Poisson's Ratio μ	Natural Water Content	Specific Gravity: Solids	Specific Gravity: Mass	Dry Density lb/cu ft	Direct Shear: Peak Strength	Direct Shear: Sliding Resistance	Weathering
Warm Springs Dam, CA Dry Creek, Russian River Basin, 6 miles northwest of Healdsburg	Great Valley and Franciscan (Jurassic and Cretaceous)	Franciscan formation south consists of a heterogenous assemblage of sandstones and interbedded shales, altered volcanic rocks, ultramatic rocks and serpentinites and minor amounts of glaucophane and related schists, all of which have been faulted and folded. The Great Valley sequence consists mainly of bedded carbonaceous shales interbedded with arkose sandstones and cobble conglomerates	Boring 1F-19 (soft shale)	17	-	-	8.5	-	-	132.3			Wet-dry tests on harder shale pieces showed breakdown into granular particles and severe flaking after two cycles in fresh water. In salt water, shale pieces began to break down after second cycle and could be broken by finger pressure after 15 cycles
			Boring 1F-27 (soft shale)	10	-	-	6.8	-	-	143.7			
				9	-	-	7.6	-	-	136.7			
			Boring 1F-32 (soft, medium, and hard shale)	19	-	-	5.4	-	-	147.4			
Chatfield Dam and Reservoir, CO South Platte River, 8 miles south of Denver	Dawson (Tertiary)	Composed of sandstone, siltstone, and clay shale. Sandstones are light gray to dark green-gray; soft, silty, clayey, relatively uncemented. Siltstones are soft, dark gray-green; sand and clay size particles contained as impurities; essentially uncemented although quite firm due to inclusion of fines and compaction. Clay shales are generally dark gray-green, soft, sandy, silty, uncemented, and compacted	Boring 105 (sandy shale)	39	-	-	13.8	-	-	123.6			No weathering test performed Swell tests Tests conducted on clay shale samples from undisturbed box samples under a restraining load of 0.5 tsf, 0.68 and 0.10 percent swell occurred
				26	-	-	14.7	-	-	117.9			
			Boring 106 (sandy shale)	147	-	-	9.2	-	-	132.2			
				65	-	-	15.0	-	-	106.7			
			Boring 88 (weathered sandy shale)	12	-	-	24.7	-	-	103.0			
			Boring 31 (shale w/ sand)	45	-	-	13.4	-	-	120.7			
			Boring 175 (weathered shale)	9	-	-	23.0	-	-	98.5			
				221	-	-	12.4	-	-	122.1			
			Boring 55 (hard shale)	1048	-	-	8.9	-	-	132.3			
Bear Creek Dam, MO Bear Creek, 2 miles southwest of Hannibal	Hannibal Shale (Miss.)	Medium hard to hard laminated shale	Not reported	2265	-	-	8.2	-	-	156.0			Slaked readily upon exposure to air

(continued)

(Sheet 1 of 4)

Table 13. Summary of shale rock core data from selected CE earth and rock-fill dams. (continued)

Project	Formation and (Age)	Description	Sampled Material	Unconfined Compressive Strength psi	Modulus E 10^6 psi	Poisson's Ratio μ	Natural Water Content	Specific Gravity Solids	Specific Gravity Mass	Dry Density lb/cu ft	Direct Shear Peak Strength	Direct Shear Sliding Resistance	Weathering
Paintsville Lake, KY Paint Creek, Big Sandy River Basin, 4 miles west of Paintsville	Lee (Penn.)	Thinly bedded to blocky sandstone containing conglomerate zones that are interbedded with dark gray to black carbonaceous shale	Boring C-77 (hard, brittle shale)	-	0.070	-	2.4	-	-	153.2	Carbonaceous shale c = 11 tsf ϕ = 17 deg		Clay shale core sample subjected to 12 cycles of wet-dry tests Severe weathering had taken place after 4 cycles with complete separation along several bedding planes
Yatesville Lake, KY Blaine Creek, 4 miles south of Louisa	Coalburg Sandstone (Penn.)	Massive, moderately hard, medium-grained, generally gray sandstone with interbedded siltstones, claystones, coals, and shale	Boring C-71 (clayey shale)	3765 3461	0.317 0.336	0.44 0.34	2.0 2.0	- -	- -	161.4 160.8	Silty claystone c=6.5 tsf ϕ=14 deg	c=5 8 tsf ϕ=10 deg	Weathering test on sandstone only
Brookville Reservoir, IN East Fork of Whitewater River, Ohio River Basin, Brookville	Cincinnati (Ord.)	Gray, medium-hard to hard calcareous clay shale with soft, fossiliferous seams, interbedded with gray, hard, medium-grained crystalline, fossiliferous, highly jointed limestone. Clay shale layers are laminated by thin limestone lenses	Boring DC-210 (weathered shale) (medium-hard shale)	40 - 427 365 -	- - - - -	- - - - -	17.0 11.8 8.3 8.2 6.4	- - - - -	- - - - -	110.5 127.3 142.1 140.9 149.6			No weathering tests performed
Alum Creek Reservoir, OH Alum Creek, Scioto River Basin, 10 miles southeast of Delaware	Ohio Shale (Devonian)	Brown-black silty, carbonaceous shale, moderately hard, thinly laminated, platy. Average laminae thickness is 2 in. Contains pyrite inclusions throughout. Thin beds of light gray, moderately hard, silty to clayey shale is interbedded throughout	Boring C-146 Boring C-147	5675 5665 5815	0.66 1.00 0.65	- - -	2.9 - 2.8	- - 2.55	- - 2.39	148.7 - 151.5	c=28.8 tsf ϕ=46 deg Q: c=117 tsf ϕ=29.1 deg	rock on conc c=28.8 tsf ϕ=48 deg	Wet-dry cycle tests indicate shale is moderately susceptible to air slaking Hairline bedding fractures visible after 5 cycles. Complete separation along bedding planes prior to 39 cycles
Caesar Creek Reservoir, OH Caesar Creek, about 30 miles northeast of Cincinnati	Richmond (Ord.)	Gray, moderately soft to moderately hard calcareous shale with 10 to 15 percent gray, hard, crystalline, fossiliferous limestone interbeds. Limestone beds increase to as much as 65 percent above el ±885 in some areas of the reservoir	Boring DC-51 (gray, silty shale)	150 290 475 -	- - - -	- - - -	7.1 5.8 3.8 3.9	- - - -	- - - -	131 9 151.7 156 6 162.5	c = 1 4 tsf ϕ = 52 deg		No weathering tests performed

(continued)

(Sheet 2 of 4)

Table 13. Summary of shale rock core data from selected CE earth and rock-fill dams. (continued)

Project	Formation and (Age)	Description	Sampled Material	Unconfined Compressive Strength psi	Modulus E 10^6 psi	Poisson's Ratio μ	Natural Water Content	Specific Gravity Solids	Specific Gravity Mass	Dry Density lb/cu ft	Direct Shear Peak Strength	Direct Shear Sliding Resistance	Weathering
East Fork Lake, OH East Fork of Little Miami River, 4 miles southeast of BataVia	Maysville (Ord.)	Gray, moderately hard to moderately soft calcareous shale, interbedded with hard, fossiliferous, crystalline limestone. Limestone beds vary from 0.1 to 1.0 ft in thickness. Limestone composes 20 to 35 percent of the formation	Boring DC-68 (weathered shale)	28	-	-	18 9	-	-	111.4	c = 8.8 to 15 tsf ϕ = 18.3 deg		No weathering tests performed
			Boring C-93B (medium-hard shale)	583	0.111	-	5.4	-	-	147.0			
				879	0.180	-	4.2	-	-	151.5			
				1101	0.200	-	4.6	-	-	152.1			
			Boring C-206 (soft shale)	488	-	-	5.7	-	-	151.8	c = 1.9 to 5.1 tsf ϕ = 10 to 24 deg		
Fishtrap Dam, OH Levisa Fork, Big Sandy River, 7 miles southeast of Pikeville	Norton (Penn.)	Shale is gray with numerous thin laminae of light colored sandstone and siltstone. Shale is fissile, highly micaceous and contains sand and silts. Parts easily along the closely spaced bedding planes intersected by high angle opened joints which are heavily stained with iron. The laminae of sandstone becomes thinner and more widely spaced with depth	Boring C62-34 (sandy shale)	5124	0.863	0.070	1.4	2.75	2.63	167.8	Tests were conducted but data were too erratic to plot	conc on rock ϕ = 31.0 deg rock on rock ϕ = 29.5 deg rock on rock ϕ = 26.1 deg	
				6994	1.629	0.244	0 9	2.73	2.66	143.5			
				-	-	-	0.9	2.74	2.66	166.7			Wet-dry tests: 4-13 cycles indicated hairline partings; no change through 28 cycles; hairline cracks along bedding plane at start of test. 18 cycles produced increase in partings; 28 cyles indicated 4 complete openings
				-	-	-	0.8	-	-	167.3			
				-	-	-	1.4	2.77	2.67	167.5			
				3354	0.480	-	1.9	2.79	2.65	143 2			
	Gladeville (Penn.)	Interbedded light gray fine-grained sandstone and dark gray shale characterized by the partings along the shale seams. The laminae vary from a fraction of an inch to several inches in thickness with no joint or fracture pattern. No evidence of weathering is present	Boring C62-25 (interbedded sandstone and silty shale)	7457	3.528	-	1.0	-	-	166.5			Wet-dry tests indicated: no change after 28 cycles
				-	1.298	-	-	-	-	167.0			
										155.1			
				7584	1.420	-	0.8	2.75	2 66	167 8			4-18 cycles indicated several hairline partings along bedding plane. No change after 18 or 28 cycles
				-	-	-	0.9	-	-	167.1			
				5667	1.838	-	1.1	2.74	2.66	143.6			
				-	-	-	0.9	-	-	166.7			
	Norton (Penn.)	Light to dark gray shale varying in amounts of sands and silts, compacted and medium hard, open joints and vertical fractures occur in the upper portion but decrease with depth	Boring C26-47 (medium-hard shale)	412	0.061	-	3.7	2.71	2 43	144.7			Wet-dry tests indicated that after 4-13 cycles produced hairline partings but through 28 cycles there was no further breakdown
				-	-	-	2.9	-	-	161.0			
				246	-	-	1.2	2.74	2.63	166.8			
				-	-	-	0 9	-	-	161.9			
				-	1.096	-	0.8	2.73	2.64	167.6			

(continued)

(Sheet 3 of 4)

Table 13. Summary of shale rock core data from selected CE earth and rock-fill dams. (concluded)

Project	Formation and (Age)	Description	Sampled Material	Unconfined Compressive Strength psi	Modulus E 10^6 psi	Poisson's Ratio μ	Natural Water Content	Specific Gravity Solids	Specific Gravity Mass	Dry Density lb/cu ft	Direct Shear Peak Strength	Direct Shear Sliding Resistance	Weathering
R. D. Bailey Lake, WV Guyandot River, 1 mile north-east of Justice	Kanawha (Lower Penn.)	Gray, moderately hard sandy to clayey shales with coal beds and sandstone stringers underlain by sandstone members	Boring C-71 (black, soft, shale)	- -	- -	- -	1.5 1.5	- -	- -	166.0 164.0	c = 22.7 tsf ϕ = 29 deg		Some of clayey shales air-slaked when exposed in the field
			Boring C-71 (gray, soft shale)	215 266	- -	- -	- -	- -	- -		c = 13 7 tsf ϕ = 19 deg		Wet-dry tests indicated: 4-10 cycles produced cracks with a thin lamination flaking off end of sample After 24 cycles. half of 6-in. core broken into fragments: other half remained intact
Raystown Reservoir, PA Raystown Branch, Juniata River, 3 miles south of Huntingdon	Catskill (Dev.)	Predominantly red, interbedded shale, silt-stone, and sandstone. Shale and siltstone are soft and silty. Lamellar bedding is common in most of the shale and siltstone but is not readily evident unless these rocks are weathered. Beds vary laterally in thickness and lithology	Boring FDH-181 (soft shale) (soft siltstone)	2816 1325 4225	- - -	- 0.32 -	1.6 1.4 1.0	2.95 2.85 2.78	2.82 2.74 2.69	169.3 169.5 169.5			Wet-dry tests indicated: shale cracked parallel to bedding planes on second and third cycles and completely disintergrated by the end of 50 cycles. By the end of 50 cyles the siltstone samples were badly cracked but intact
			Boring BDH-198 (soft shale) (silty shale)	2859 2203 4026	- - -	1.4 0.93 -	0.8 1.5 1.4	2.83 2.81 2.83	2.73 2.69 2.70	169.5 169.6 169.2			
			(silt-stone)	5041	-	0.32	0.8	2.82	2.73	169.0			
			Boring BDH-196 (shaly siltstone)	10088	-	0.31	0.4	2.75	2.70	170.4			

(continued)

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It appears that bedding and macrostructure in the harder shales of the eastern States have a predominant influence on the degree of breakdown. The unconfined compressive strength of cores at their natural water content does not appear to be a good indicator of possible deterioration.

Compacted shale strength data. To provide a useful reference on shear strength of compacted shales, data were extracted from available design memoranda for CE earth and rock-fill dam projects located in the states of interest in the study. Since results of tests on large-scale samples were given priority to include tests on large particle sizes (2 to 3 in.), most of the data were for projects in harder shales east of the Mississippi River. Data from 14 projects considered pertinent are summarized in Table 14 (fold out tables at the end of this report). For convenience, data for siltstone at Cachuma Dam from tests by the U. S. Bureau of Reclamation (USBR) are also included. Data for large-scale triaxial tests on a mixture of weathered sandstones and shale for the Jail Gulch embankment in California (conducted at SPD) are not included since this information is summarized by Hall and Smith (1971).

The data in Table 14 include a wide variety of shale materials and shale-limestone and shale-sandstone mixtures. However, it should be noted that only two of the problem shales identified in Table 4, the Franciscan and Maysville groups, appear in Table 14. The gradations indicate a significant amount of breakdown during testing with a resulting change in classification to a more clayey material in many cases. The specific gravity of solids ranged widely from 2.69 to 2.82. Maximum dry densities and optimum water contents from standard effort compaction (AASHTO T 99) tests varied widely and do not appear to be related to the type of material, except for the clay shale at Bear Creek, Colo., and the siltstone at Cachuma, Calif. The limited vertical permeability data indicate that the materials were relatively permeable at standard effort density.

The triaxial tests were performed on material crushed in the laboratory except for three projects (Nos. 12, 13, and 14) which included tests on material from test fills. Triaxial test samples were compacted either at the expected field water content or at a water content related to optimum and at a dry density related to the maximum standard effort density.

For several of the Q tests (all controlled strain), the deviator stress increased throughout the test with an increase in axial strain, and the samples exhibited a bulging type failure. The Q strengths, as might be expected, were significantly influenced by initial water contents. As shown in Figure 25, materials compacted at low water content (2 to 6 percent) exhibited ϕ values of 28.5 to 38.5 deg with c values ranging from 0.5 to 2.0 tsf. Materials at higher water contents (11.1 to 27) exhibited ϕ values from 0 to 22.5 deg with c values ranging from 0.4 to 2.6 tsf.

The majority of R tests summarized in Table 14 were back pressure saturated and pore water pressures were measured. Thus, R strengths based on total stresses and $\overline{R}$ strengths (equivalent to S strengths) based on effective stresses were determined. For many of the tests (all controlled strain), the deviator stress continued to increase with increases in axial strain for the duration of the test (noted by an asterisk in the consolidation stress column). Measured pore water pressures increased initially during the test and then leveled off or decreased slightly during the remainder of the test. Failure was predominantly of the bulging type. A large decrease in pore water pressure, possible under a high back pressure and indicative of a tendency for dilation and erroneously high deviator stresses, did not occur in any of the tests. Consequently, the increase in deviator stress with increasing strain is believed to be valid. However, a number of the R and $\overline{R}$ envelopes were slightly curved indicating that the interpreted value of cohesion may be high.

The R strength data indicate that tests on samples saturated by back pressure generally have a lower shear strength than samples tested at initial water content. Strengths for a majority of the tests on saturated samples indicated ϕ values of 14 and 20 deg and c values as large as 0.5 tsf. The exceptions to these ranges were for tests on shale from Beltzville (No. 13) which indicated a wide range on values of

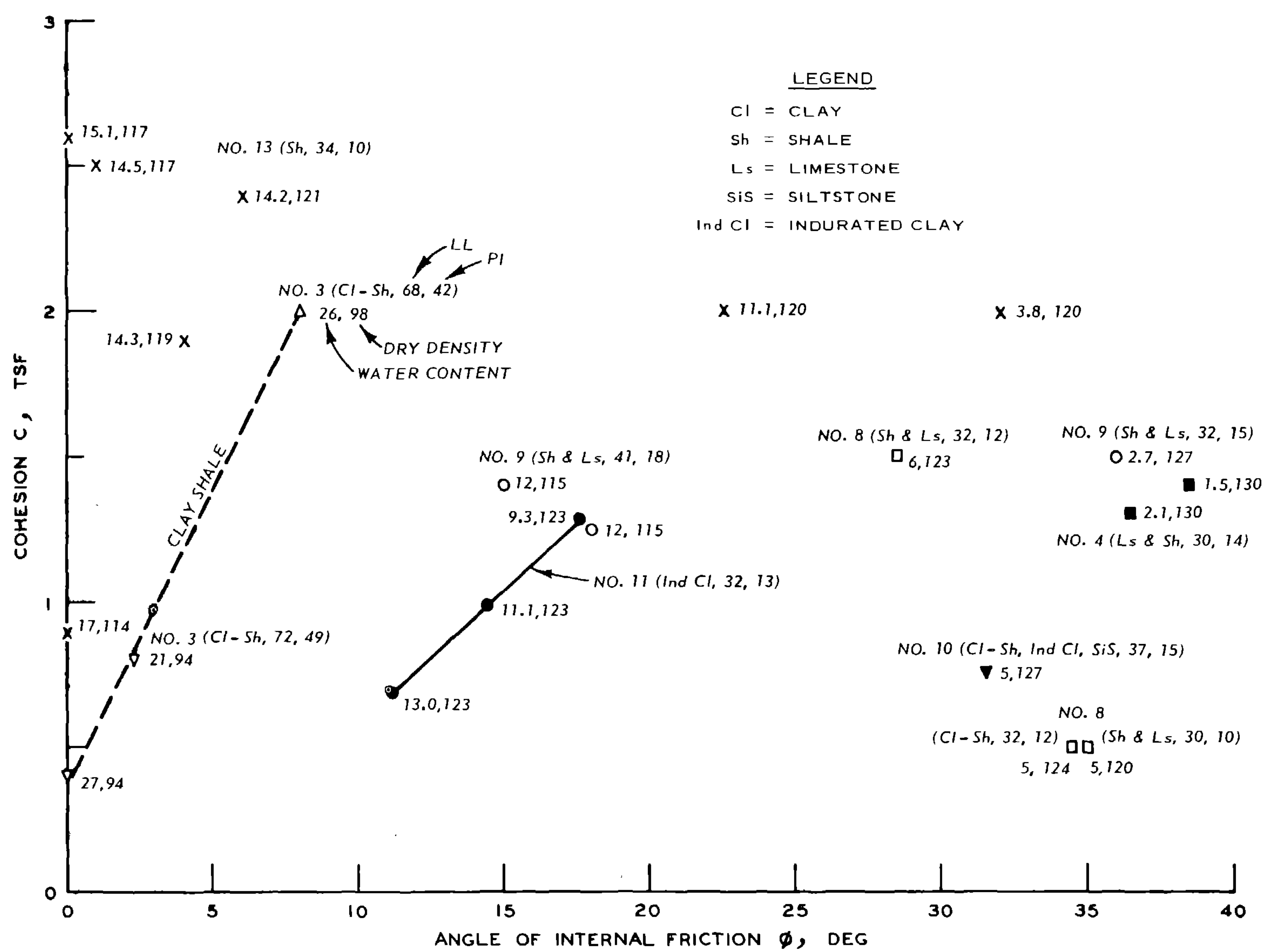

Figure 25. Comparison of Q strength data from Table 14.

both ϕ and c and for tests on the shale-limestone from Taylorsville (No. 9) and East Fork (No. 4) which had c values as high as 0.9 and 1.5 tsf, respectively. However, for these latter two materials, the strengths of the saturated samples were still significantly lower than the strengths of unsaturated samples. Higher ϕ and c values for higher initial densities were noted in some cases (Fishtrap Dam, No. 6; Beltzville, No. 13; and Trexler Lake, No. 15). Similar R strengths were obtained for tests on crushed material and test fill material when both types of material were tested (R. D. Bailey, No. 12, and Beltzville, No. 13). It should be noted that the siltstone at Cachuma Dam had the highest shear strengths.

The S strength data also indicate curved strength envelopes, and the listed c' values should be discounted (especially for Fishtrap Dam, No. 6, and Beech Fork Dam, No. 11). The ϕ' values ranged from 31 to 45 deg with one low value of 25 deg for the clay shale at Bear Creek Dam, Colo. The ϕ' values do not appear to be related to the age of the formation or physical properties. Where both crushed material and test fill material were tested (Raystown, No. 14), similar S strengths were obtained.

The design values for Q, R, and $\overline{R}$ and S strengths listed in Table 14 indicate the conservatism used in selecting strength values for stability analyses. Current CE guidance suggests that design shear strengths should be selected such that two-thirds of the laboratory test values (for tests at different dry densities and water contents) exceed the design values. This conservative selection is done to offset possible field compaction at the wet side of the specifications and at densities no higher than the design dry density.

The data presented in Table 14 appear to provide a useful design guide and illustrate that relatively low Q and R strengths exist even when standard effort maximum densities are achieved. It is evident that compacted shale materials at water contents above 10 percent have relatively low Q strengths and that saturation such as by seepage can significantly reduce R strengths.

<u>Residual shear strengths</u>. The residual shear strength of shales,

especially that of clay shales in natural and cut slopes, has received much attention and research. The residual strength is the minimum strength attainable upon sliding and has application to shear plane failures in compacted embankments. A recent study by Townsend and Gilbert (1974) presents a summary of residual strengths for 15 different shales and correlations of residual shear strengths with classification indexes. The correlations have a limited usefulness in predicting residual strengths from index properties such as liquid limit and plastic limit.

USBR Practices

Shales from required excavations have been used successfully in several earth dams constructed by the USBR. The use of shales has been restricted either to berms at the toe, where only additional weight is needed to increase the stability of a weak foundation, or to a special zone in the downstream section of the dam. In the latter case, the suitability of the shale material as compacted fill is thoroughly investigated by laboratory tests such as large scale compaction and triaxial compression and field test fills. When used in a downstream zone, the shales are well compacted and surrounded by more pervious materials. This practice is used to prevent seepage into the shale zone and to relieve excess pore water pressures that might develop should the heavy compaction produce an impervious mass.

At Cachuma Dam (constructed during 1950-1953 near Santa Barbara, Calif.), some 700,000 cu yd of siltstone from the spillway excavation in the Monterey formation was used in the downstream zone 3 section. This zone was completely surrounded by pervious sand, gravel, and cobbles to protect it in case the siltstone degraded under compaction into an impervious mass. Piezometer measurements in 1971 indicated slightly negative pore pressures in the zone 3 section. These readings imply that free water is being held in the soil by capillary tension.

A test fill was constructed during final design (Hilf, 1957a). The material in the Monterey formation used for the test fill consisted of partially laminated, fine-grained, soft siltstone which was light in weight and virtually saturated. The average in-place properties were:

Wet unit weight	112.5 pcf
Water content	33 percent
Dry unit weight	85 pcf
Specific gravity of grains	2.48
Void ratio	0.82
Porosity	45 percent
Liquid limit	40 to 50 percent
Plasticity index	8 to 12 percent

The results of the test fill showed that the siltstone, containing varying amounts of rock fragments larger than 6 in. (USBR, 1959), could be readily broken down and compacted at the natural water content with the USBR heavy tamping roller (minimum of 4000 lb per foot of length of drum, Hilf, 1957b). Results of triaxial shear strength tests (Gibbs et al., 1961) on 3-1/4-in. diam by 9-in.-high specimens of minus 3/8-in. material compacted from crushed cores showed a high effective shear strength with cohesion c' of 0.6 tsf and ϕ' of 37 deg (Table 14).

Although much of the siltstone in the spillway excavation could have been excavated by shovel without blasting, the contractor found it economical to drill and shoot. A 4-cu-yd shovel and a 2-1/2-cu-yd shovel were used to load the siltstone. The excavated siltstone was delivered to the zone 3 section in bottom-dump trucks and placed in layers about 18 in. thick (USBR, 1959). Although the siltstone was excavated in fairly large chunks (as large as 2 to 3 cu ft), the haul trucks and spreader tractor broke down the blocks reasonably well. After it was spread, the lift was thoroughly wetted and compacted by 12 coverages of the USBR heavy roller. Water was added to maintain a near-saturated condition in the absorptive siltstone chunks (short of creating a spongy fill). The coarse gradation of the fill (about 6-in. maximum size after compaction) produced a relatively pervious and stable fill (due to interlocking of angular pieces under rolling).

An unusual criterion was used for compaction control because of the low percentage of minus No. 4 fraction material (about 30 percent or less) in the compacted fill. Standard effort laboratory compaction tests (using a 4-in.-diam mold) gave erratic results because of continuous breakdown of particles during compaction. To overcome this effect, the natural dry density of the siltstone was used as a reference. The dry unit weight of the natural siltstone was determined by measuring the bulk specific gravity of the plus No. 4 fragments in the field density sample. For compaction control purposes, it was established that the dry density of the compacted fill material should not be less than 85 percent of the dry density of the natural siltstone. A total of 174 field density control tests showed that the fill water content averaged 31 percent and the fill dry density averaged 79 pcf (or 89 percent of the natural siltstone dry density).

Shales from the Mancos formation were used with pervious soils in the downstream section of Jackson Gulch Dam (constructed in 1947-1949 west of Durango, Colo.). In this case, the shale was compacted using rubber-tired rollers to minimize the amount of fines produced by compaction. This procedure provided a well-compacted material that was still relatively pervious. Trinity Dam (constructed in 1957-1959 near Redding, Calif.) incorporated weathered shale (particle sizes up to 3 in.) from the Bragdon formation in an interior upstream and downstream section (zone 2). Materials in zone 2 were compacted into 12-in. layers by 12 coverages of the USBR heavy tamping roller. Blocks over 12 in. were broken down by extra rolling. Shale was also used in a small zone in the downstream portion of Heron Dam (recently constructed in northern New Mexico). No detailed information has been published on these dams.

The experience of the USBR with compacted shales indicates that:

a. The use of 12 coverages of the heavy USBR tamping roller and removal of oversize rocks produce a well-compacted fill. Compacted layers 12 in. or less in thickness and proper water content and density control are required to achieve the required compaction and shear strength.

b. Shales used within embankments should be limited to downstream zones and should be surrounded by a zone of well compacted and more pervious material.

Large-scale triaxial compression tests are often used to determine the shear strength of compacted shale materials containing large particle sizes (up to 3 in.). When swell pressures measured from consolidation tests on the finer fraction are relatively high, the shale materials generally are not used in earth dam embankments. Details of USBR field investigation and laboratory test procedures are described in the USBR "Earth Manual" (USBR, 1963) and "Design of Small Dams" (USBR, 1965).

Application of CE and USBR Practices to the Construction of Highway Shale Embankments

The CE and USBR practices applicable to highway shale embankments are discussed briefly below.

a. The use of 12- to 18-in. loose lifts and heavy tamping rollers to aid in breaking down harder shales or mixtures of shale and limestone or sandstone followed by compaction with a heavy roller such as the 50-ton rubber-tired roller, appears warranted in obtaining densities needed for stability of slopes as steep as 1V on 2H. Well compacted shale materials appear to resist deterioration because of initial breakdown during compaction with heavy equipment.

b. Very few shales appear to be durable enough for use as compacted rock fill in thick lifts (2 to 3 ft) without using heavy rollers (50-ton rubber-tired or 10-ton vibratory steel wheel).

c. The use of procedural-type specifications appears to have an advantage over end result density specifications which require frequent field density tests for enforcement. Mixture of shale and harder rock particles (as large as 8 to 10 in.) require large-scale field density apparatus and the tests are expensive and time-consuming to perform. On the other hand, the need for field test fills in developing procedural type specifications may be reduced by using available experience (such as in Tables 7 and 12).

d Segregation of shales from limestone and sandstone where feasible would allow these durable and relatively free-draining materials to be used as drainage blankets at the base of the embankment and as stronger protective rock fill in exterior zones.

e. Reduction in shear strengths of compacted shales caused by seepage saturation could be prevented by good drainage at the base of the embankment, especially on sidehill and abutment slopes. Where saturation is prevented, higher shear strengths of unsaturated shale materials would permit more economical design of compacted shale embankments and remedial repairs to failed embankments.

f. Large-scale compaction and triaxial tests are applicable to highway embankments but would be extremely expensive for a highway project. Consequently, the strengths summarized in Table 14 may be a useful guide for similar materials and could be supplemented by triaxial strength testing of minus No. 4 material in smaller size apparatus accommodating 3-in.-diam specimens.

g. Use of combined strength envelopes in stability analyses is applicable to large, high embankments (generally 100 ft or higher), but for smaller embankments more simplified procedures are usually considered adequate.

IV. CLASSIFICATION AND COMPOSITIONAL ASPECTS OF FINE-GRAINED CLASTIC SEDIMENTARY ROCKS

Classification

The fine-grained sedimentary rocks investigated during the course of this study have been described as shale, clay shale, indurated clay, siltstone, mudstone, etc. These names have been applied to different types of materials for several different reasons. Generally, the names reflect physical characteristics and/or physical properties important in classification or engineering. Most classification schemes have been based upon grain size, rock fabric, degree of induration, or engineering properties. These aspects are discussed below using mudrock as a general name denoting the above types of sedimentary rocks.

Grain size. The terms listed above apply to sedimentary rocks composed mainly of particles finer than approximately 1/16 mm and corresponding to silt- and clay-size fractions. These size fractions should comprise at least 50 percent of the total rock according to some schemes. Even so, the size composition of these rocks may be quite variable and may include a considerable amount of sand-size particles. The mudrock classes are gradational with sandstones and to a limited extent with calcareous rocks, including limestones. General aspects of this gradation are shown in the compositional tetrahedron in Figure 26.

Rock fabric. Fabric refers to the spatial orientation of the particulate material and cement (if any) in sedimentary rocks. This aspect becomes important if the rock contains appreciable platey clay minerals which may contribute to the fissility of the rock which results in the splitting up of the rock along closely spaced parallel surfaces. Rocks which break into large, equidimensional blocks are described as massive. Mineralogy, grain-size, and postdepositional history appear to determine whether fine-grained sedimentary rocks are fissile or massive.

Degree of induration. This factor describes the extent to which the material in question is rocklike or sedimentlike. Theoretically, rocklike material should be hard, durable, and nonfriable. Sedimentlike

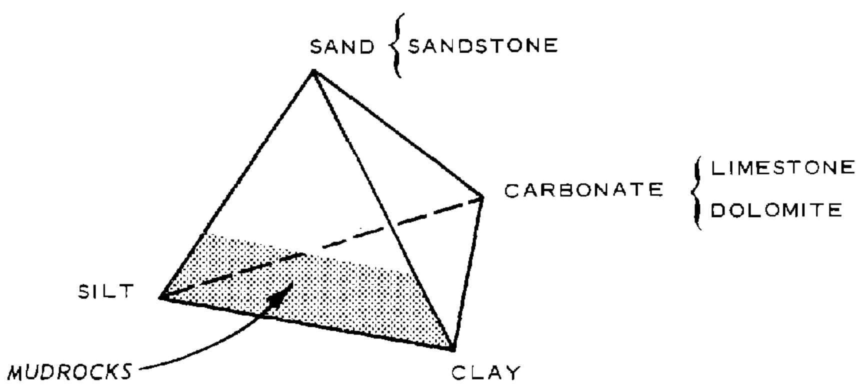

Figure 26. Grain size composition of mudrocks.

material should be soft, nondurable, and friable. This factor depends upon postdepositional history as well as the environment of the material. The postdepositional history determines the amount and type of cement and the maximum overburden pressure under which the sediments have been loaded. The environmental conditions such as degree and depth of weathering, position of groundwater table, and sample depth all influence the extent to which a material is rocklike. Mudrocks exhibiting induration derived by mineral cements are called "cementation shales," whereas induration produced by compaction results in mudrocks termed "compaction shales."

Engineering properties. Classification schemes based upon engineering properties account for such attributes as swelling potential, plasticity, consolidation history, durability and soundness, and shear strength in categorizing mudrocks. These attributes are usually related to grain size, rock fabric, degree of induration, and also the mineralogy of the rock.

Geological Classification Schemes

Ingram (1953). This classification recognizes grain size and extent of fissility in categorizing mudrocks. The degree of induration is not specifically stated, but apparently Ingram excludes materials that will easily break down to their particulate constituents. Table 15 summarizes the classification.

This scheme is relatively descriptive and is amenable to field as well as laboratory use. However, no statement is made as to how thick a shale fragment can be and still be considered fissile. In this same paper, Ingram summarizes the causes of fissility in shales; these will be discussed in the next section.

Folk (1968). Folk follows Ingram's lead and includes all fine-grained argillaceous rocks under the term "mudrock." A mudrock must contain more than 50 percent silt and/or clay. Further subdivision is based upon relative amounts of silt and clay and the presence or absence of fissility. Folk's classification is shown in Table 16.

Table 15. Ingram's classification of mudrock.

Relative amount of Si and Cl	Breaking Characteristics: Unknown or Not Stated	Massive	Fissile
Unknown or not stated	Mudrock*	Mudstone	Mud shale
Si is greater than Cl	Siltstone	Siltstone	Silt shale
Cl is greater than Si	Clay rock	Claystone	Clay shale

* 75 percent silt (Si) and clay (Cl) sizes.

Table 16. Folk's classification of mudrock.

Grain Size of Mud Fraction	Massive	Fissile
Subequal silt and clay	Mudstone	Mud shale
Over 2/3 silt	Siltstone	Silt shale
Over 2/3 clay	Claystone	Clay shale

As with Ingram's classification, there is no size definition of fissility. Folk recommends that mudrocks containing over 10 percent sand should be prefixed by the sand size, e.g., "very fine sandy claystone."

The classification schemes described above are general in nature and are primarily useful for field descriptions, geologic mapping, and petrologic purposes. The classifications which follow were intended to describe rock properties and relate to selected test parameters.

Underwood (1967). Underwood discusses the in situ engineering properties of shales and their relationship to rock characteristics and constituency. This paper is not addressed specifically to shale deterioration. Although no formal engineering classification is presented, Underwood reviews geological classification schemes and discusses the problems of formulating a classification based upon engineering properties. He concludes that the "cementation shales" pose the least engineering problems while the compaction or soillike shales pose the greatest problem. Also, he states that there are insufficient test data as well as agreement on test procedures to develop an engineering classification. This conclusion still appears to be true.

Deo (1972). Deo developed an engineering classification which considers shale durability. The index classification scheme (Figure 27) is based on slaking, slake-durability, and sulfate soundness tests, all of which are briefly discussed in Wood and Deo (1975). These three tests measure shale durability and are discussed in Section VI. The categories of classification include soillike shale, two types of intermediate shales, and rocklike shale. The rocks tested to provide the basic data were all from Paleozoic deposits in Indiana, thus the results are applicable to these older rocks. Even so, the proposed classification scheme is being used by the Indiana Highway Commission in evaluating shales for use in rock fills (see Parts VI and VII).

Gamble (1971). The most comprehensive study of shale durability was conducted by Gamble. The materials investigated included a wide range of rocks of different ages, locations, and constituencies. The laboratory work also included a variety of standard classification tests

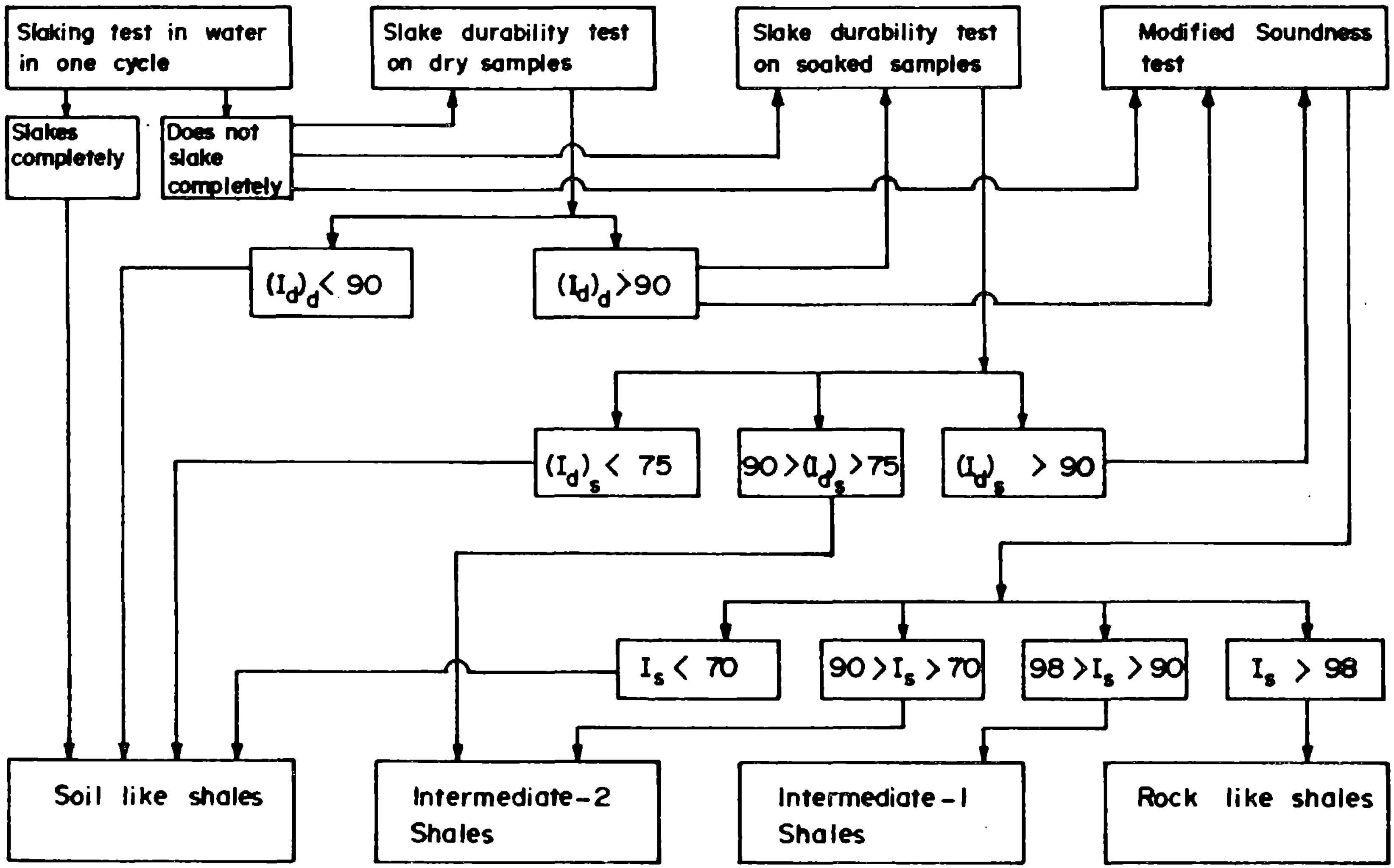

Figure 27. Proposed classification of Indiana shales for embankment construction (Deo, 1972).

as well as strength and durability evaluations. Gamble's classification relates two-cycle slake-durability to plasticity index (Figure 28). He concluded that more work is necessary to correlate laboratory results with field performance.

Morgenstern and Eigenbrod (1974) presented an engineering classification of shales based upon rate of jar slaking, liquidity index, and undrained shear strength (Figure 29). The rocks examined were exclusively Tertiary and Mesozoic in age; thus, the older more highly indurated rocks were not analyzed.

Composition

The following discussion relates engineering properties to the composition of mudrocks in general, with emphasis on the siltstones and clay shales. These two mudrock types represent materials with significantly different compositions and behaviors such that they may be considered end member types. Intermediate types such as mudstones are believed to have engineering properties ranging between the end members. The major properties considered are plasticity, swell potential, durability, and strength. These properties are related to texture, mineralogy, geochemistry, and rock fabric and/or structure.

Mineralogy. The mineral constituency of mudrocks may be quite variable and is dependent upon several factors, including sedimentational aspects such as environment of deposition and sediment source area, and diagenetic aspects. The environment of deposition and source area control the type of original detrital minerals, and diagenesis affects changes in the original mineral suite as well as the introduction of authigenic* cements.

It is convenient to categorize the rock constituency as to mineral origin in discussing the mineralogy of mudrocks. Mudrocks consist of two general types of materials: detrital, particulate minerals and authigenic minerals. The detrital minerals may be sand-, silt-, or

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* Formed after deposition.

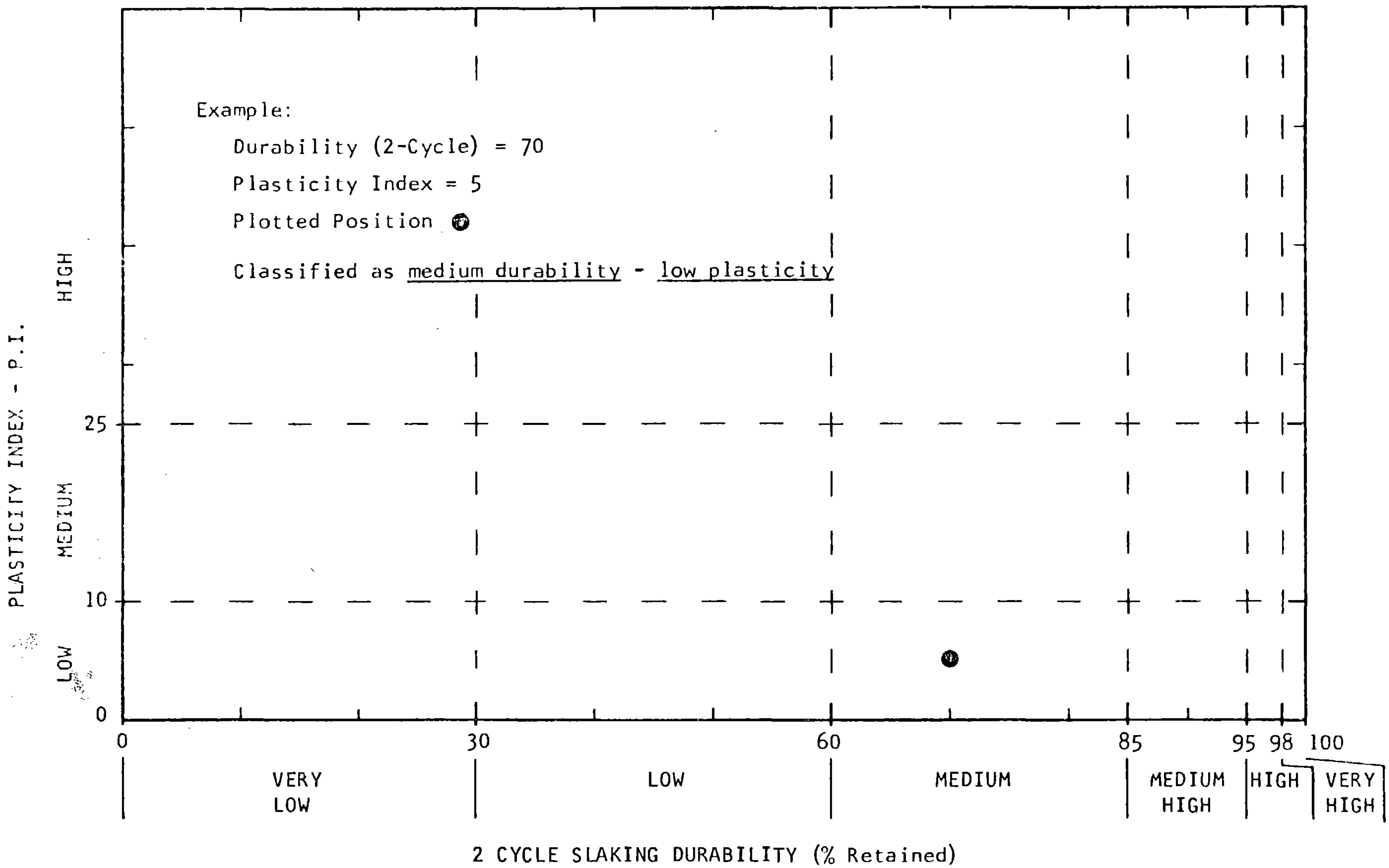
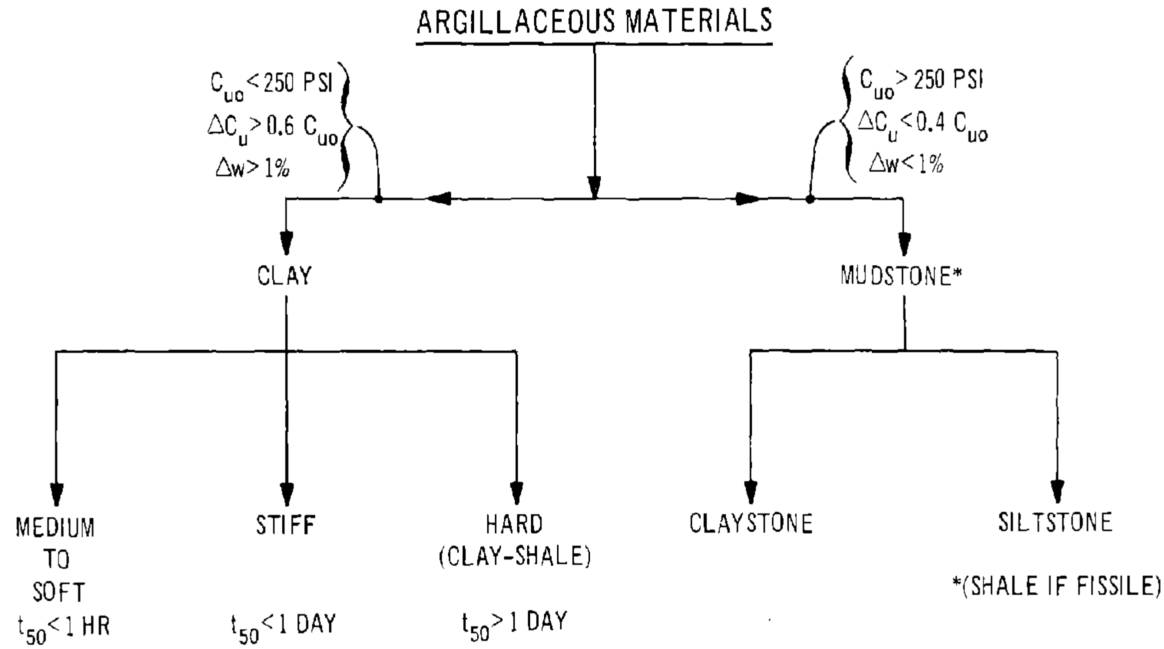


Figure 28. Durability-plasticity classification for shales and other argillaceous rocks (Gamble, 1971).



C_{uo} = UNDRAINED SHEAR STRENGTH AT NATURAL WATER CONTENT (Q TEST USING 50 PSI CHAMBER PRESSURE)

ΔC_u = STRENGTH LOSS AFTER SOFTENING TO EQUILIBRIUM WATER CONTENT

Δw = CHANGE IN WATER CONTENT AFTER SOFTENING

t_{50} = TIME OF SOFTENING FOR LOSS OF 50% OF C_{uo}

a. ENGINEERING CLASSIFICATION OF ARGILLACEOUS MATERIALS

w_L = LIQUID LIMIT

w_s = MAX. WATER CONTENT DUE TO SLAKING

I_L = LIQUIDITY INDEX

ΔI_{L1} = CHANGE IN LIQUIDITY INDEX

		AMOUNT OF SLAKING $w_s = w_L$				
		VERY LOW VL $w_L < 20$	LOW L w_L BETWEEN 20 & 50	MEDIUM M w_L BETWEEN 50 & 90	HIGH H w_L BETWEEN 90 & 140	VERY HIGH VH $w_L > 140$
RATE OF SLAKING: $\Delta I_{L1} = I_{L1} - I_{L0}$ (2 HR WATER IMMERSION)	SLOW, S $\Delta I_L < 0.75$	VL S	L S	M S	H S	VH S
	FAST, F $0.75 < \Delta I_L < 1.25$	VL F	L F	M F	H F	VH F
	VERY FAST, VF $\Delta I_L > 1.25$	VL VF	L VF	M VF	H VF	VH VF

b. CLASSIFICATION IN TERMS OF SLAKING CHARACTERISTICS

Figure 29. Classification of argillaceous materials (Morgenstern and Eigenbrod, 1974).

clay-size. Table 17 illustrates one method of categorizing the mineral constituency of mudrocks.

The detrital minerals consist chiefly of quartz and the clay minerals. Ordinarily, the quartz is restricted to the sand- and silt-size fraction, but occasionally it will occur in the clay-size fraction. The presence of appreciable quartz or amorphous silica in the clay-size fraction may be indicative of an authigenic origin of the quartz. The clay minerals normally occur in the clay-size fraction but some clay minerals such as kaolinite may occur in the silt-size fraction. The type of clay mineral present often controls the engineering properties of mudrocks due to the ability of some clay minerals to imbibe large quantities of water.

The authigenic mineral suite consists principally of cements with minor amounts of noncementing minerals, many of which may have originated by diagenesis of the original detrital minerals and may be related to the geologic age of the rock. The cements are perhaps the main elements of rock strength in siltstone, but they may also be vulnerable to the effects of weathering. Cements play a less important role in the durability and strength of clay shales.

The extent to which the particular mineral constituents individually affect the engineering behavior of siltstones is dependent upon the extent to which cementing minerals (or the cementing effects of diagenetic bonds) continue to bond the particulate, detrital minerals together and, once broken apart, upon the type and amount of clay minerals. The effects of weathering and the destruction of cement bonds are discussed in Part V, and the effects of the clay minerals are discussed below.

The clay minerals are hydrous aluminosilicates generally smaller than 2 micrometres (μm) in size and usually platty but occasionally tubular in shape. Ionic substitutions within the lattice, broken bonds on lattice edges, and London-van der Waals bonds on lattice faces usually result in electrical charge deficiencies and attractive or repulsive forces on individual clay mineral grains.

The fine-grained clay minerals are classed as phyllosilicates and

Table 17. Common mineral constituents of mudrocks.

Detrital			Authigenic	
Sand size	Silt size	Clay size	Noncementing	Cements
Quartz	Quartz	Quartz	Clay minerals	Calcite
Feldspars	Feldspars	Feldspars	Zeolite	Dolomite
Micas	Micas	Micas	Feldspars	Siderite
Heavy minerals	Chlorite* Heavy minerals**	Clay minerals	Pyrite	Gypsum
(Rock fragments)	Clay minerals	Kaolinite Halloysite Illite/mica † Smectite Chlorite* Vermiculite Mixed-layer types		Quartz/silica
(Volcanic glass)	Kaolinite			Iron oxides and hydroxides
(Shells)				Aluminum oxides and hydroxides
(Organic matter)		(Organic matter)		Pyrite
	(Rock fragments)	(Allophane)		
	(Volcanic glass)			
	(Shells)			
	(Organic matter)			

( ) detrital rock fragments; amorphous, nonmineral glass, and biological constituents.

* The properties and composition of silt-size chlorite are significantly different from clay-size chlorite.

** Heavy minerals may also occur as clay size.

† Micaceous minerals exhibiting 10 Å d-spacings and clay size are also called clay-mica.

possess structural similarities to coarse-grained minerals such as talc, mica, and chlorite.

A first-order classification of clay minerals is based upon their structural configuration. The structures are distinguished by the arrangement of two basic components: a sheet of silicon atoms tetrahedrally coordinated with oxygen and a sheet of aluminum or magnesium atoms octahedrally coordinated with the hydroxyl radical. Further classification is based upon the chemical population and ionic substitutions within these two basic structures. The clay mineral classification is given below and shown in diagram form in Table 18.

The clay minerals are classified as follows:

a. Two-layer clays. These consist of one silicon tetrahedral layer bonded to one aluminum octahedral layer. Kaolinite is the common mineral when the octahedral layer contains mainly aluminum; serpentine consists of a magnesium-rich octahedral layer.

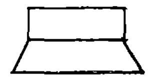
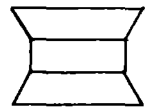
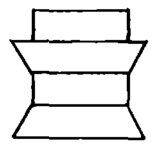
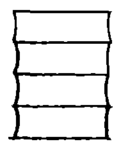
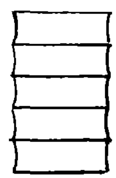
b. Three-layer clays. These clays have one octahedral layer bonded between two tetrahedral layers; examples of this type are illite, vermiculite, and montmorillonite.* These minerals may occur dioctahedrally or trioctahedrally.

c. Mixed-layer clays. These clays consist of an interstratification of tetrahedral, octahedral, two- and three-layer combinations. The mixing may be regular or random. An example of a regular-mixed layer clay is chlorite, a three-layer plus octahedral layer repetition; another common one is montmorillonite-chlorite. The randomly mixed layer clays consist of any of many possible combinations.

When the strength due to cement or diagenetic bonds is lost, either by field compaction or by weathering, the mudrock loses its original identity and behaves as a soil. The resulting plasticity, expansiveness, shear strength, etc., are controlled by the amount and type of clay and nonclay minerals present. Examples are given below for the relationship between clay mineralogy and engineering behavior. These examples are mainly for generally monomineralic materials, but they do illustrate the influences imposed by particular clay minerals.

* The term "montmorillonite," as used here, indicates the dioctahedral magnesium-bearing member of the smectite group.

Table 18. Structural classification of clay minerals.

Category	Thickness	Configuration		Example
2-layer clay minerals	7 Å		Octahedral Tetrahedral	Kaolinite
3-layer clay minerals	10 to 15 Å		Tetrahedral Octahedral Tetrahedral	Illite Vermiculite Montmorillonite
Mixed-layer clay minerals:				
Regular	14 Å		Octahedral Tetrahedral Octahedral Tetrahedral	Chlorite
	26 to 29 Å		Montmorillonite Chlorite Montmorillonite Chlorite	Interlayered montmorillonite and chlorite
Random	Variable		Montmorillonite Chlorite Chlorite Montmorillonite Montmorillonite	Mixed-layer montmorillonite and chlorite

Plasticity (Grim, 1962). Average values of Atterberg limits for montmorillonite, illite, and kaolinite clays are listed below:

Clay	Plastic Limit	Liquid Limit	Plasticity Index
Sodium-montmorillonite	91	441	350
Calcium-montmorillonite	68	155	87
Illite	39	86	47
Kaolinite	31	54	23

The other clay minerals, chlorite, vermiculite, halloysite, and the mixed-layer types possess much more variable plasticity. Vermiculite and chlorite may have properties similar to those of montmorillonite or illite. Halloysite has a plasticity somewhat greater than that of kaolinite. The mixed-layer clays exhibit properties that may fall between those of illite and montmorillonite.

Expansiveness (Grim, 1962). The extent to which an argillaceous material will imbibe water and thus increase in volume is directly related to the clay mineral present and is an important consideration in engineering. The following table illustrates average free-swell data for the common clay minerals:

Clay	Average Free Swell, Percent	Range, Percent
Sodium-montmorillonite	1500	1400 to 1600
Calcium-montmorillonite	102	65 to 145
Illite	89	60 to 120
Kaolinite	28	5 to 60

It can be seen that the montmorillonite minerals exhibit the most severe expansion, with kaolinite and illite exhibiting a lesser amount. Vermiculite and occasionally chlorite are also expansive but generally not to the extent of montmorillonite. Halloysite may be as expansive as illite.

Strength (Grim, 1962). The strength of argillaceous materials is a function of mineralogy as well as loading history, water content, and amount of nonclay mineral constituents. The examples below represent nearly pure monomineralic materials.

Unconfined Compressive Strength, psi

	Wet Specimen	Air-dried Specimen
Sodium-montmorillonite	56	195
Kaolinite	100	66

Shear Strength, psi

Montmorillonite	960
Kaolinite, illite, halloysite	680

Grim concluded that the strength of the monomineralic material is not particularly meaningful since the strength of a mixture of clay and nonclay components is more dependent on the amount and type of nonclay constituents and also on the loading history.

Fabric

The fabric of sedimentary rocks refers to the orientation of the particulate material and cement (see page 111). The overall fabric of mudrock is a function of the orientation of the platey clay minerals. The platey nature of these minerals permits their accumulation such that the individual platelets lie one upon another with face-to-face contact between platelet faces. This contact is also called dispersed orientation. Mudrocks having this orientation usually exhibit some degree of fissility and are classed as shales. Under certain conditions of sedimentation, the individual platelets do not accumulate in a face-to-face condition but rather in random orientation such that the contacts are between edges and faces. This orientation is called edge-to-face contact and represents the fabric of the mudstones, siltstones, and claystones.

The development of a particular type of orientation is based upon the sedimentational environment, diagenesis, and loading history.

Specifically, dispersed orientation and the resulting fissility are aided by the following apparent factors (Ingram, 1953):

a. Appreciable clay minerals. Since clay minerals are platty, they contribute to the overall platty character of the rock, whereas silt contributes more to the massiveness.

b. Slow sedimentation accompanied by low salt concentration. Slow sedimentation and low salt concentration prevent flocculation. The type of salt present is also important. For similar concentrations, Na^{+} is more dispersive than Ca^{++}.

c. Absence of burrowing organisms. The presence of burrowing organisms causes disruption of sediment fabric.

d. Presence of organic matter. The role of organic matter in enhancing fissility is not known. It may, however, complex the flocculating salts.

e. Presence of illite. Illite apparently tends to be more easily dispersed; also, illite is a more common constituent of the older and consequently more deeply buried mudrocks.

f. Lack of cements. Cements may cause a disruption of dispersed fabric; however, cement is more commonly present in silty rocks due to their higher permeability and thus may not directly affect the fabric.

g. Deep burial. Deep burial results in high overburden pressures which permit the reorientation of flocculated clays and enhancement of dispersed fabric.

Geochemistry and hydration characteristics. The chemistry of mudrocks is principally expressed by the mineral suite, i.e., mudrocks composed of illite will possess a chemistry quite different from mudrocks composed of kaolinite. The different properties of the rocks, however, are mainly caused by the differences in mineralogy. The engineering properties of mudrocks may be influenced by the interlayer cations on the clay minerals and by the ions in the pore water.

Substitutions within the crystal lattice and broken bonds at lattice edges result in an effective charge deficiency on the lattice which is compensated by ions bonded to the crystal. Substitutions generally result in a negative charge unbalance, whereas broken bonds may cause either positive or negative deficiency. Hydrogen bonds at the surface of the octahedral layer of kaolinite will also attract ions to this surface.

Thus, the clay mineral is surrounded by a layer of ions which (with exception of the hydrogen bonds) satisfies the electrical balance of the system. These ions may significantly affect the hydration characteristics of the mineral.

V. FACTORS CONTRIBUTING TO MATERIAL DEGRADATION

Geological and Environmental Considerations

The extent to which shales exhibit properties suitable for embankment construction is not only a function of the nature of its constituents but is also dependent upon geologic history and present-day environment, including the as-constructed environment. Although the rock's constituency itself is sometimes dependent upon the geologic events to which the rock has been subjected, this part of the report will discuss the geological and environmental conditions which by themselves may determine the suitability of these materials.

Geologic age. The geologic age of a rock may contribute to its properties through diagenesis, depth of burial, and the introduction of cements. The factor of age is relative and not exclusively independent of other factors such as the character of the original sediments, metamorphism, or loading.

Diagenesis. Probably the most important attribute of diagenesis is its effect upon the constituent clay mineralogy. Paleozoic rocks usually exhibit minor amounts of montmorillonite, whereas this mineral is abundantly present in the younger Mesozoic and Tertiary rocks. This characteristic is due to the relative instability of montmorillonite which leads to the development of illite, chlorite, and mixed-layer clay minerals in its place. This diagenetic change is apparently enhanced by time, burial, and increase in temperature. Also, older rocks may have time for more effective precipitation of cements; however, this factor may not be very important for the more impermeable clay shales.

Depth of burial. The extent of rocklike attributes in older rocks that have been more deeply buried is quite variable. The deeply buried rocks have been subjected to high overburden pressures which cause overconsolidation and orientation of clay mineral platelets.

Tectonic history and metamorphism. Folding, faulting, and the effects of metamorphism produce characteristics which are considerably different from the attributes of the original rock or sediments.

Folding, usually accompanied by elevated temperatures, results in hard, dense shales. Shales so produced may approach slates in their degree of induration. In composition, this change is accomplished by mineral recrystallization and grain growth. The suitability of these hard shales or slates may be controlled by the extent that these rocks have been sheared, faulted, or jointed. Structures such as the last three may impart weaknesses to the rock such that loading and compaction will result in excessive degradation. The effects of metamorphism itself may be quite variable. Thermal metamorphism produces a rock called "argillite" which exhibits some degree of recrystallization without appreciable grain growth and usually is fairly dense. Regional metamorphism on a limited scale may also produce slatelike rocks which possess a hardness and density suitable for construction purposes requiring durable material.

Weathering. All rocks lying near the earth's surface, including rocks placed in embankments, are subject to the processes of weathering. The susceptibility of a particular rock or stratigraphic unit to weathering will exert considerable influence on whether or not the material is acceptable for placement in an uncompacted rock fill and whether the material will maintain its rocklike characteristics after placement. Susceptibility to weathering is dependent upon several factors such as intrinsic properties of the rock, topography, groundwater table position, and climate.

The weathering processes are conveniently classed as physical, chemical, or biological. This classification is somewhat arbitrary since some weathering processes involve two or more types of action. Physical processes include stress release caused by unloading, freezing and thawing, pressure of crystallization (also a chemical process), and forces exerted by plants and organisms (biological processes). These physical processes cause rock degradation by breaking the bonds of contact between the particulate materials in the rock.

The chemical processes include hydrolysis, hydration, carbonation, oxidation, and various types of ion exchange. These processes involve breakdown of existing minerals and formation of new minerals from the existing chemical constituents (usually with additional components introduced from outside the system), alteration of existing minerals, and the removal of the existing mineral accompanied by the removal of its chemical components. These processes occur in and because of the aqueous environment near the earth's surface and are controlled by the ionic content of the surface and groundwater and by the organic environment in the zone of weathering whether in situ or in a fill (Keller, 1962).

Shales intended for use in embankments may occur in various stages of weathering and, therefore, may exhibit varying degrees of soundness, durability, and strength. It would appear that the selection of relatively unweathered shale would be best for construction of rock fills. However, this shale (probably removed from <u>below</u> the zone of weathering) will be exposed to the weathering processes when placed in the embankment. Thus, it is necessary to determine, if possible, the weathering effects the shale will exhibit after construction. The effects will largely depend upon the intrinsic properties of the rock and the nature of the embankment.

The postconstruction changes produced by weathering of the fill material are time dependent. The extent of weathering in a given embankment material during a given time period may be difficult to determine, but there are indicators that may be helpful. Aside from laboratory tests discussed in later sections of this report, one of the best means of determining relative weathering rates is through observation of the material in outcrops and recent exposures, if available. If the rock is highly susceptible to weathering, the recent exposures and outcrops will exhibit several feet of altered rock or soil developed upon the fresh material.

Although embankment problems caused by weathering will be discussed in the next section, it would be well to list here the changes brought about by weathering of shales; these changes are discussed below.

<u>a</u>. <u>Increase in fines</u>. The weathering processes (as well as compaction) tend to disaggregate the rock, resulting in a relative increase in the silt- and clay-size fractions. These fundamental particles, no longer bound together, may then react further

with water to develop higher plasticity, greater swelling and shrinkage, more dispersiveness, higher pore pressures, and more susceptibility to freezing and thawing.

b. Mineralogical changes. The weathering processes may alter existing nonclay minerals and from these produce new minerals. These new minerals will generally be slightly lower in specific gravity, are often hydrous, and as a result, exhibit higher volumes which cause an expansion or volume increase in the material. The clay minerals usually undergo little mineralogical change but respond to weathering by developing higher plasticity which may be drastically increased if the weathering is accompanied by cation exchange.

The driving force of the chemical weathering processes, whether in situ or in an embankment, is water. Water is the medium by which the chemical reactions occur and is also the carrier of dissolved ionic species which drive the reactions to completion. Water is a more effective weathering force if it can percolate through the material with ease. The chemical system ensures complete reactions and will thereby weather more material in the embankment. Thus, it is important to consider groundwater, drainage, and permeability aspects of the embankment and how these will affect the embankment material.

Material Degradation Factors

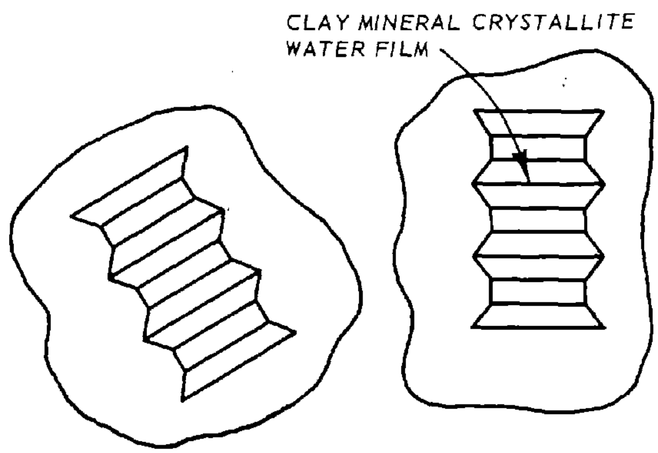
Information gathered from the States during the initial phase of the study was not complete enough to determine specific factors responsible for problems in shale embankments. Shale deterioration is believed to be a major factor, but lack of information on construction conditions, mixing of soil and rock, and inclusion of weathered soil layers masked this aspect. Consequently, possible intrinsic causes of deterioration of shales are covered in this section. The factors discussed below represent geological or mineralogical conditions or processes which may contribute to the failure of fills and embankments. These conditions and processes lead to failure by effecting changes in the material properties of the embankment that result in either deformation or loss of stability. The factors are not generally independent nor mutually exclusive; they may operate concurrently; and one factor may trigger another.

Expansive clay minerals. The small grain size, large surface area, and lattice substitutions of the clay minerals result in strong attractive forces capable of bonding with water molecules. The water molecules bonded to the clay minerals surround either the individual clay mineral crystallites or an aggregate of crystallites. The water molecule surrounding the clay tends to develop an orientation which possesses some degree of crystallinity and which may be several molecular layers thick. The development of these layers of oriented water results in a reduction of the van der Waal's attraction between the clay platelets (or aggregates), repulsion, and an increase in the volume of the clay. Most clay minerals exhibit this type of expansiveness. Thus, any poorly cemented argillaceous rock composed mainly of clay minerals will, if adequate moisture is available, be capable of swelling.

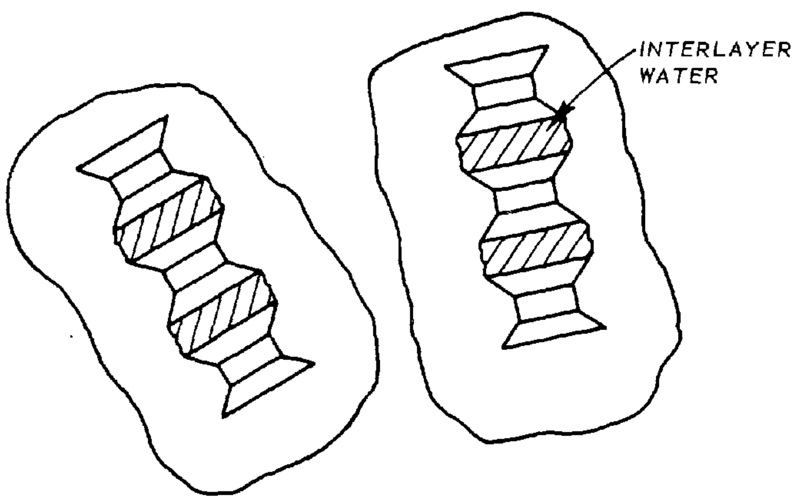
On the other hand, certain clay minerals such as montmorillonite, vermiculite, and varieties of chlorite also possess the property of attracting water to interlayer positions. Since this results in an increase in swelling potential, and these minerals (mainly montmorillonite) may pose serious pavement problems on the embankment.

The two general types of swelling behavior are illustrated in Figure 30. Figure 30a shows clay mineral crystallites with the surrounding water film. Figure 30b represents clay mineral crystallites such as montmorillonite which exhibit both the surrounding water film as well as interlayered water between the crystal lattices. Usually, clay minerals which possess interlayer water are subject to greater swelling pressures and expansion.

Montmorillonite-rich clays occur in shales and other sedimentary rocks of Mesozoic and Tertiary age in large areas of the United States and generally have originated from the devitrification of volcanic ash. Locally, where thick deposits of volcanic ash have accumulated, the devitrification produces beds or lenses of bentonite ranging in thickness from a few centimetres to 1 m. Those rocks which do not contain bedded



a. CLAY MINERAL CRYSTALLITE
WITH PERIPHERAL WATER ENVELOPE



b. CLAY MINERAL CRYSTALLITE WITH
PERIPHERAL WATER ENVELOPE
AND INTERLAYER WATER

Figure 30. Relationships between clay minerals and sorbed water.

or lenticular bentonite often have montmorillonite disseminated throughout in smaller concentrations.

Expansive clays may produce undesirable effects in an embankment by two processes. First, the clay in the rock may actually expand or shrink producing a distortion of the embankment surface which results in the breakup of the pavement. Second, the intake of water onto clay surfaces or into interlayer positions may cause softening of the material and a large decrease in shear strength.

Dispersive clay minerals. Dispersive clays consist of clay minerals (usually montmorillonite) that tend to disaggregate or disperse when in contact with water. The disaggregation or dispersion results in the effective erosion of the material and a reduction in the size of the particles placed in the fill. Shales composed of dispersive clay minerals usually slake freely when placed in an aqueous environment.

It is somewhat arbitrary to discuss dispersive and expansive clay minerals separately since their mineralogy is so similar and since dispersion and expansion may occur simultaneously. However, the processes are different. In the case of expansion, there is little loss of material, whereas in dispersion there may be significant erosion and material loss.

The exchangeable cation on the clay appears to control the amount of dispersion or expansion. Sodium produces higher rates of dispersion than calcium, magnesium, or potassium. Therefore, montmorillonite clays carrying sodium as the exchangeable ion should be regarded as potentially troublesome.

Sherard (1972) and others have found that the percentage of exchangeable sodium present in shale material is directly related to the amount of dispersion the materials exhibit. The relationship is referred to as the exchangeable sodium percentage (ESP), which may be calculated from the following equation:

$$ESP = \left(\frac{Na}{CEC}\right)100 \qquad (1)$$

where

CEC = total cation exchange capacity of the material, milliequivalents per 100 g

Na = sodium concentration, milliequivalents per 100 g

It is believed that dispersive clays will pose greater problems in embankments which have been placed as rock fills. The greater pore space and higher permeability of these embankments will result in the more effective movement of water through the fill and, hence, more erosion. Fills placed as soils will be less permeable and will, if sodium montmorillonite is present, exhibit those problems associated with expansive clays.

Clay mineral weathering. Clay minerals which are the products of weathering are usually quite resistant under most weathering environments. However, the addition of water to the clay mineral in an embankment may result in a certain amount of mineralogical change which may be significant to the understanding of embankment deterioration. The changes which the clay minerals may undergo involve increased hydration of individual clay crystallites and cation exchange. Although montmorillonite and some chlorites and vermiculites exhibit more potential for these changes, it is believed that all clay minerals are thus affected to a certain extent. Since both hydration and cation exchange involve the addition and movement of moisture to and through the embankment, rock fills are more subject to these processes than embankments placed as compacted soils. Also, it would appear that shales which are adequately cemented and remain so during the life of the embankment would exhibit less alteration by clay mineral hydration or by cation exchange.

An increase in the hydration state of the clay minerals will result in increased plasticity and, hence, decreased shearing resistance of the clay. Cation exchange, which may be initiated by the ionic content of the natural infiltrating water and increased by deicing salts (see page 142), changes the original character of the clay by disrupting clay-water bonds or by permitting a larger amount of water to be oriented about the clay crystallites, thereby leading to a more plastic and lower strength condition or to one which will make the clay minerals more expansive or more dispersive.

Even though the clay minerals are relatively stable in most weathering environments, some clay minerals, particularly illites and chlorites, may be affected by strong acids.* If an illite rock is subjected to sulfuric acid derived from the oxidation of pyrite (which also may be a constituent of the rock), the acid tends to dissolve the clay minerals in the process. Equation 2 below describes such a reaction.

$$\underset{\text{illite**}}{4KAl_3Si_3O_{10}(OH)_{2(c)}} + \underset{\text{iron sulfate}}{12FeSO_{4(aq)}} + \underset{\text{water}}{54H_2O_{(\ell)}} + \underset{\text{oxygen}}{3O_{2(g)}} \longrightarrow$$

$$\underset{\text{jarosite}}{4KFe_3(OH)_6(SO_4)_2} + \underset{\text{aluminum hydroxide}}{12Al(OH)_{3(aq)}} + \underset{\text{silica hydroxide}}{12Si(OH)_{4(aq)}} + \underset{\text{sulfuric acid}}{4H_2SO_{4(aq)}} \qquad (2)$$

where

c = crystalline phase

aq = ionic phase in aqueous solution

ℓ = liquid (aqueous) phase

g = gas phase

In this idealized equation, the illite reacts with oxygen and acid-producing iron sulfate to form the new mineral jarosite plus aluminum and silica hydroxides which may be in solution or in a solid form. The iron is a common constituent of shales and can occur as an oxide, a sulfate as shown, or within the lattice of the clay.

A similar reaction can occur in the absence of iron whereby the clay mineral illite can alter to the aluminous mineral alunite, this is shown in Equation 3 below.

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* The formation of acids during weathering discussed in the section on cement weathering.

** The idealized formula for muscovite is used to represent illite in this equation for simplification.

$$\underset{\text{illite}}{KAl_3Si_3O_{10}(OH)_{2(c)}} + \underset{\text{sulfuric acid}}{2H_2SO_{4(aq)}} + \underset{\text{water}}{6H_2O_{(\ell)}} \rightarrow \underset{\text{alunite}}{KAl_3(OH)_6(SO_4)_{2(c)}}$$

$$+ \underset{\text{silica hydroxide}}{3Si(OH)_{4(aq)}} \tag{3}$$

Jarosite and alunite are both water soluble and may therefore precipitate in fractures or exposed rock surfaces, or they can be carried in solution some distance beyond the embankment.

This type of clay mineral weathering produces a deterioration of the rock particles in the embankment which may lead to unstable conditions.

Cement weathering. A fundamental key to the strength of a sedimentary rock is the effectiveness of its cement. The relatively impermeable nature of mudrocks has resulted in less precipitation of cement in shales than in sandstones. However, cement may still be a critical constituent, especially for rocks containing appreciable silt. The common cementing agents include calcite, gypsum, iron and aluminum oxides and hydroxides, pyrite, and silica.

The weathering of the cement and its subsequent removal pose three problems which may affect the embankment: (a) the admission of water to the clay mineral constituents that may result in expansive or dispersive behavior or clay mineral weathering; (b) general rock breakdown and loss of grain-to-grain contacts; and (c) formulation of new, nonclay mineral constituents which may bring about crystallization forces which cause the material to swell. Problems (a) and (b) are discussed below and problem (c) is treated in the section dealing with crystallization pressures.

The extent to which cement weathering occurs is a function of the type of cement present and the nature of the weathering environment within the embankment; the latter will be discussed first. An embankment placed as rock fill represents an oxidizing, aqueous system. Infiltrating rainwater carrying dissolved carbon dioxide will impart a slight acidity to the chemical system. Thus, only those mineral cements in equilibrium under these conditions will persist and remain unweathered. The conditions of importance are thus oxidation potential (Eh) and pH. The Eh and pH conditions for common aqueous environments are shown in Figure 31. Table 19 gives the common mineral cements and the Eh and pH conditions under which these minerals are most stable.

Figure 31 illustrates the approximate conditions under which the common cements are stable. Lines A and D are boundaries within which water is stable; line B is the general boundary to the right of which carbonate and gypsum are stable. The area to the left of line B represents stable conditions for silica, iron oxides, and iron hydroxides. Line C separates the sulfides (below) from the sulfates. The approximate Eh and pH conditions for rainwater, streamwater, groundwater, and bogs are labeled on the diagram. The reader is cautioned that the indicated stability fields are also affected by the concentrations of ionic species in solution and by the partial pressures of dissolved gases, both of which may change the fields from those shown.

The stability of mineral cements within the slightly acid, oxidizing environment of the embankment is given in Table 19. Quartz, silica, and the iron and aluminum oxides and hydroxides are stable, whereas the calcite, dolomite, and pyrite are unstable. Siderite is probably the most stable carbonate. Gypsum is ordinarily unstable as an original mineral but may occur as a stable new mineral after the reaction of sulfuric acid on calcite. Quartz, silica, and the oxides and hydroxides of aluminum are not appreciably dependent upon Eh and pH in most natural environments; however, quartz and silica become somewhat more soluble at pH = 8 or greater.

The chemical weathering processes which cause the alteration of the unstable mineral cements are solution or chelation,* oxidation, and hydration. Calcite, dolomite, and gypsum react by solution (or chelation).

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* Chelation differs somewhat from solution in that the former involves the complexing of metal ions such as Ca^{++}, MG^{++}, or Fe^{+++} by organic matter without effervescence.

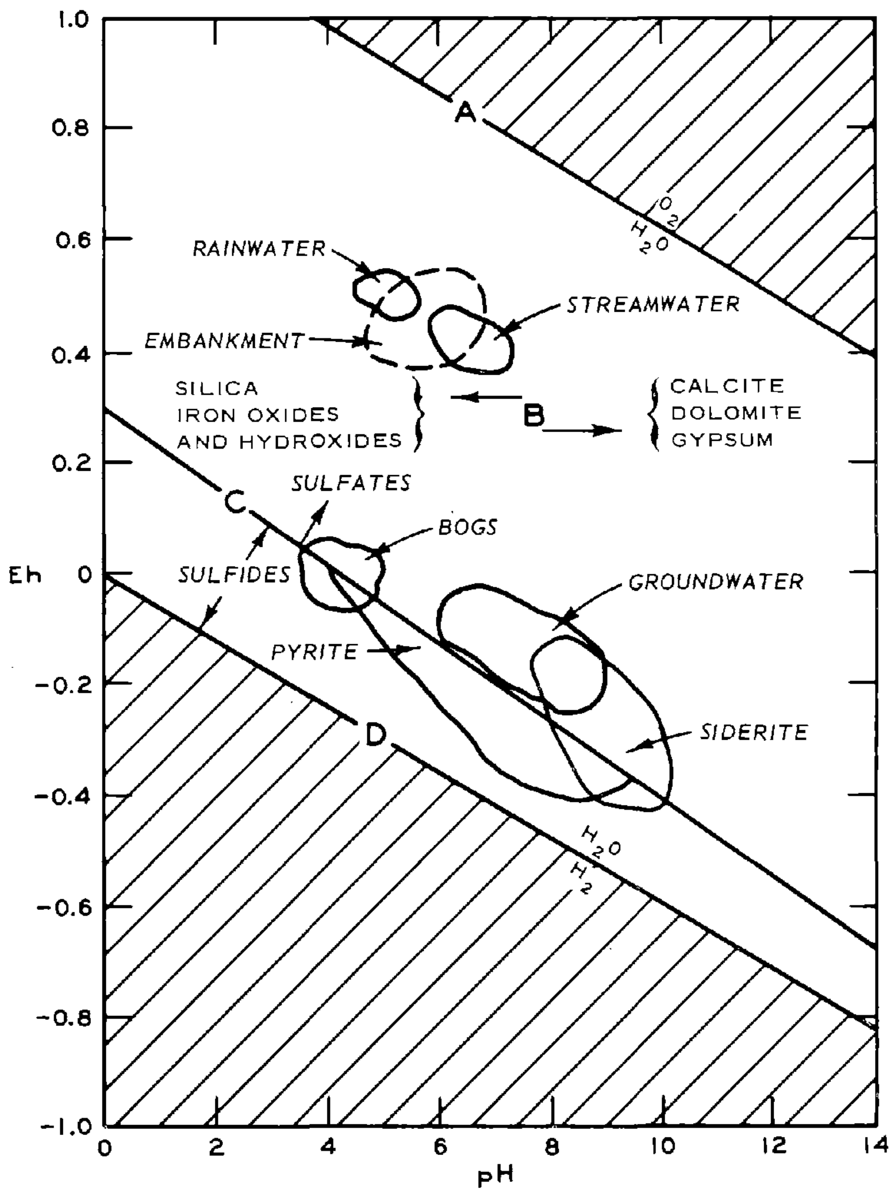


Figure 31. Eh-pH diagram illustrating approximate stability fields of mineral cements and the Eh-pH conditions of common aqueous environments, modified after Garrels and Christ (1965) and Krumbein and Garrels (1952).

Table 19. Mineral cements and their stable environments.

Mineral Cement	pH	Eh	Stable in Embankment
Calcite	$\geq$ 7.8	Variable	No
Dolomite	$\geq$ 7.8	Variable	No
Siderite	Variable	Variable	Variable
Gypsum	$\geq$ 7.8	Variable	Variable
Quartz	More or less independent of Eh and pH (see text)		Yes
Silica (amorphous)	More or less independent of Eh and pH (see text)		Yes
Pyrite	4 to 8	Reducing	No
Iron oxides & hydroxides	$\geq$ 7.8	Oxidizing	Yes
Aluminum oxides and hydroxides	More or less independent of pH and Eh (see text)		Yes

whereas pyrite and other sulfides are altered by oxidation. Hydration may occur simultaneously with solution and oxidation, and this process may involve both carbonates and sulfides.

In the absence of chelating compounds, the carbonates are reactive in the presence of weak acids which, as previously stated, may be present in the infiltrating rain and surface water due to dissolved carbon dioxide. Acid conditions may also be caused by deicing salts and due to the oxidation of sulfides. The carbonates which react in this acid environment are either completely removed in solution or are changed to another mineral species. The equations for these two types of reactions are given below.

$$\underset{\text{calcite}}{CaCO_{3(c)}} + \underset{\text{carbonic acid}}{H_2CO_{3(aq)}} \longrightarrow Ca^{++}_{(aq)} + 2(HCO_3)^-_{(aq)} \qquad (4)$$

In the presence of sulfuric acid calcite reacts thus:

$$\underset{\text{calcite}}{CaCO_{3(c)}} + \underset{\text{sulfuric acid}}{H_2SO_{4(aq)}} + \underset{\text{water}}{2H_2O_{(\ell)}} \longrightarrow \underset{\text{gypsum}}{CaSO_4\cdot 2H_2O_{(c)}} + \underset{\text{carbonic acid}}{H_2CO_{3(aq)}} \qquad (5)$$

Reaction 5 indicates that hydration also occurs simultaneously with solution.

The oxidation reactions by which the sulfides are weathered are somewhat more complex. The products of the reaction depend upon several factors including the completeness of the reaction and the ionic constituency of the water. Generally, however, the reaction of pyrite plus oxygen and water yields iron sulfate and sulfuric acid. The sulfate may become hydrated and may also contain sodium, calcium, and/or potassium ions within the structure. The first step of the oxidation reaction is given below.

$$\underset{\text{pyrite}}{2FeS_{2(c)}} + \underset{\text{water}}{2H_2O_{(\ell)}} + \underset{\text{oxygen}}{7O_{2(g)}} \longrightarrow \underset{\text{iron sulfate}}{2FeSO_{4(aq)}} + \underset{\text{sulfuric acid}}{2H_2SO_{4(aq)}} \qquad (6)$$

The sulfuric acid produced in Equation 6 above may contribute to solution of carbonate, as previously described. The iron sulfate may further react with water to form the hydrated sulfate mineral, melanterite, as shown in Equation 7.

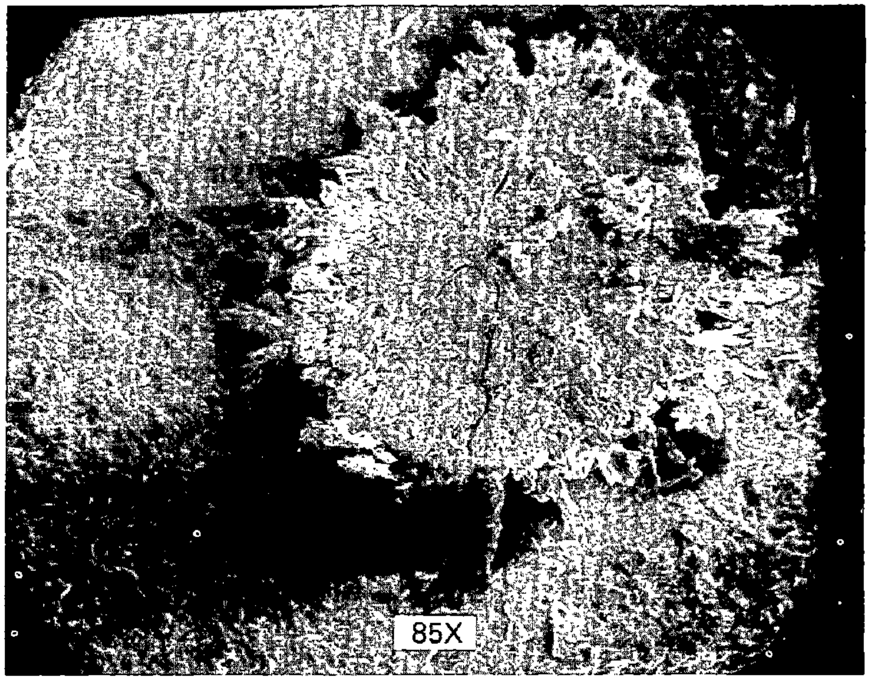
$$\underset{\text{pyrite}}{2FeS_{2(c)}} + \underset{\text{water}}{16H_2O_{(\ell)}} + \underset{\text{oxygen}}{7O_{2(g)}} \longrightarrow \underset{\text{melanterite}}{2FeSO_4 \cdot 7H_2(O)_{(c)}} + 2H_2SO_{4(aq)} \qquad (7)$$

Further oxidation and hydration may lead to the development of coquimbite, hematite, limonite, or amorphous gels.

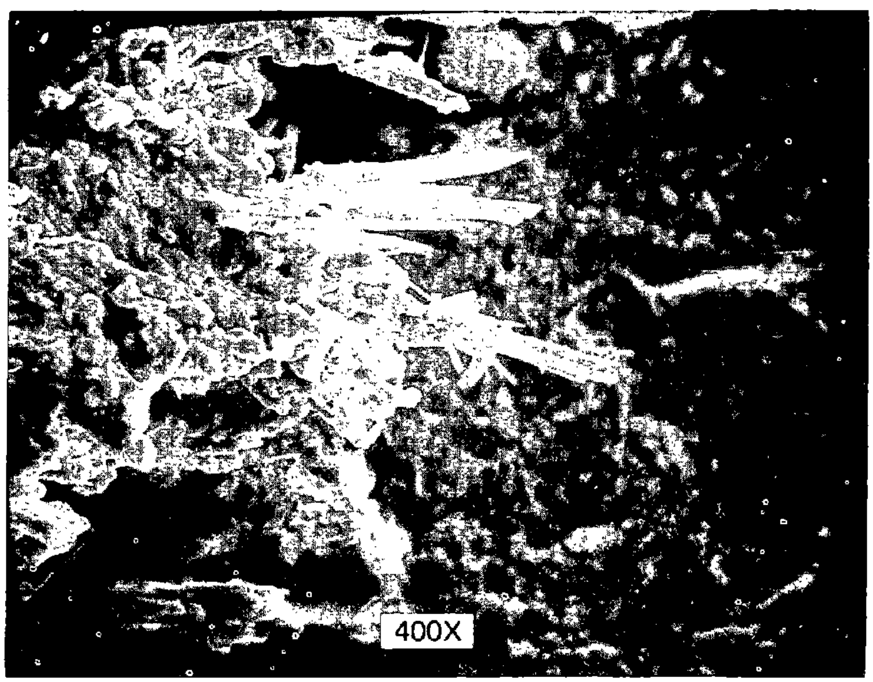
Crystallization pressures. The weathering processes previously described have been discussed in terms of chemical reconstitution and the resulting deterioration of the original minerals in the embankment material. Certain lines of evidence suggest that the deterioration of shales may also be attributed to pressures developed by the crystallization of new mineral components such as the formation of gypsum by the action of sulfuric acid on calcite. The explanation for these pressures is based upon the fact that the new minerals often require more volume than the original minerals. Usually, the larger space requirements are caused by the hydrous nature of the new components.

Figure 32 is a scanning electron microscope (SEM) photograph of a protrusion developed on a fracture plane (parting surface) in Pierre shale (Cretaceous age). The surface disruption has been produced by the crystallization of gypsum. The pressures developed during the crystallization process may have been responsible for the development of the fracture plane itself.

It is not known whether the gypsum was formed by the action of acid on calcite or by the precipitation of gypsum out of the pore water. The latter explanation would account for similar features seen on the sides of shale core samples after the samples had been allowed to dry for



a. Gypsum protrusion on fracture surface.



b. Close up of protrusion edge.

Figure 32. Fracture surface in Pierre shale exhibiting protrusion produced by the crystallization of gypsum.

several months. It would appear that the pore water in some shales may contain appreciable salts in solution such that changes in pressure or evaporation of pore water result in the crystallization of the dissolved salts. The formation of these new minerals apparently develops sufficient pressure to disrupt grain-to-grain contacts which results in a general deterioration of the rock.

Rock deterioration may not be the only result of these crystallization pressures. Millot (1970) described a bedrock situation at Nancy, France, in which the oxidation of pyrite led to the formation of gypsum after calcite. The gypsum was formed between shale layers, and the resulting heave or expansion cracked building foundations. No doubt, these pressures were amplified due to their occurrence in bedrock; however, it would appear that these processes could also affect the materials in an embankment and cause surface heave.

Millot also described the results of tests conducted on the weathering of granite by initially saturating the rock with a saline solution followed by moistening twice daily. These tests were conducted over a period of 3 months. At the end of the test run, the granites saturated with the saline solution had disintegrated 10 to 1000 times greater than a control specimen not treated with the saline solution.

The formation of new minerals involves two measurable parameters: change in volume and increase in internal pressure. The latter parameter may be determined empirically but with difficulty. The former, however, can be determined from the chemical reaction which produces the new mineral. The calculations which follow demonstrate the volume increases which may occur during the alteration.

Equation 7 represents the alteration of pyrite to melanterite. The only solid, crystalline phases present in this example are the pyrite and its alteration product, melanterite. The water and sulfuric acid are liquid phases and oxygen is a vapor phase.

$$2FeS_{2(c)} + 16H_2O_{(\ell)} + 7O_{2(g)} \longrightarrow 2FeSO_4 \cdot 7H_2O_{(c)} + 2H_2SO_{4(aq)} \qquad (7)$$

The formula weights of pyrite and melanterite are 120.1 and 278.0, respectively. Thus, 1 g of pyrite will produce 278.0/120.1 or 2.32 g of melanterite.

The 1 g of pyrite having a specific gravity of 5.1 will occupy 0.197 cc. The hydrous melanterite has a specific gravity of 1.85 and will occupy 1.254 cc. Thus, the crystalline solids require

$$\left(\frac{1.254 - 0.197}{0.197}\right)100$$

or 536 percent more space.

Table 20 lists the calculated volume changes for four weathering reactions which may produce new mineral phases. The largest changes in volume result from the calcite-gypsum and the pyrite-melanterite reactions. The alteration of illite produces significantly less volume change. These calculated values are based upon the assumption that the original mineral comprised 100 percent of the rock.

The volume changes listed in Table 20 should be considered potential in that the reaction will produce the new mineral accompanied by an increase in volume if the internal pressure regime of the rock is sufficiently low to allow the new mineral to form. The low internal pressure conditions will most likely occur in thin overburden or low embankment situations, but even then the expansion may only occur near or at the surface.

Unloading. Rocks below the earth's surface are constrained by overburden pressures and lateral pressures which depend upon the depth of cover, loading history, and nature of the material. Rocks confined by these pressures have reached a degree of equilbrium which is disturbed when the material is removed from the ground and when lateral and upper restraints are removed as in a cut.

The strength of shales is significantly influenced by the loading history. Most shales have been subjected to overburden pressures higher than those under which they exist today and are thus overconsolidated. These overconsolidated conditions are generally caused by the erosion of the overlying soil and rock in the recent or near-recent geologic past.

Table 20. Volume increases of the crystalline solid phases of selected chemical weathering reaction.

Weathering Reaction		Volume Increase of Crystalline Solids, Percent
Original Mineral	New Mineral	
Illite	Alunite	8
Illite	Jarosite	10
Calcite	Gypsum	60
Pyrite	Melanterite	536

Materials existing in a normally consolidated state and not subjected to overburden pressures higher than their present condition ordinarily would not be classed as rocks but would be considered as sediments.

It would appear that, with few exceptions, the shale materials used in embankment construction are overconsolidated. The amount of overconsolidations to which these various shales have been subjected is probably quite variable, but it seems likely that the older Paleozoic rocks are more highly overconsolidated than the Mesozoic or Tertiary rocks.

Overconsolidated, argillaceous <u>sediments</u> or weakly cemented rocks rebound when their vertical in situ stress is removed or decreased. Removal of the material from the ground would be one way that the material would have a decreased vertical stress and thus exhibit rebound. The rebound generally involves an increase in the void ratio of the material. The extent to which argillaceous <u>rocks</u> exhibit rebound is a function of the diagenetic and cementation effects which the rock has undergone.

It seems logical that well-cemented older rocks which have had their mineralogy changed by diagenesis would not exhibit nearly as much rebound as younger rocks.

Disequilibrium may occur due to the possible difference in magnitude between the vertical and horizontal stresses. For normally consolidated <u>clays</u>, the horizontal stress may range from 0.6 to 1.0 times the vertical; overconsolidated clays exhibit a horizontal stress which is 1.0 to 1.5 times the vertical. These stress differences, although difficult to predict or measure in the laboratory, may be an important consideration when shales are removed from the earth and placed in an embankment.

Shales possess numerous joints, fracture zones, and bedding separations which are effectively closed when the rocks are constrained within the earth. When the overburden is removed or when the rocks themselves are taken from the earth, the disequilibrium stresses allow these breaks in the rocks to open. The initial opening due to this stress relief may then allow moisture to enter the joints and bedding separations, thus contributing to rock deterioration by the processes of weathering, dispersion, etc.

Figure 33 illustrates X-radiographs (Krinitzsky, 1970; Knott et al.,

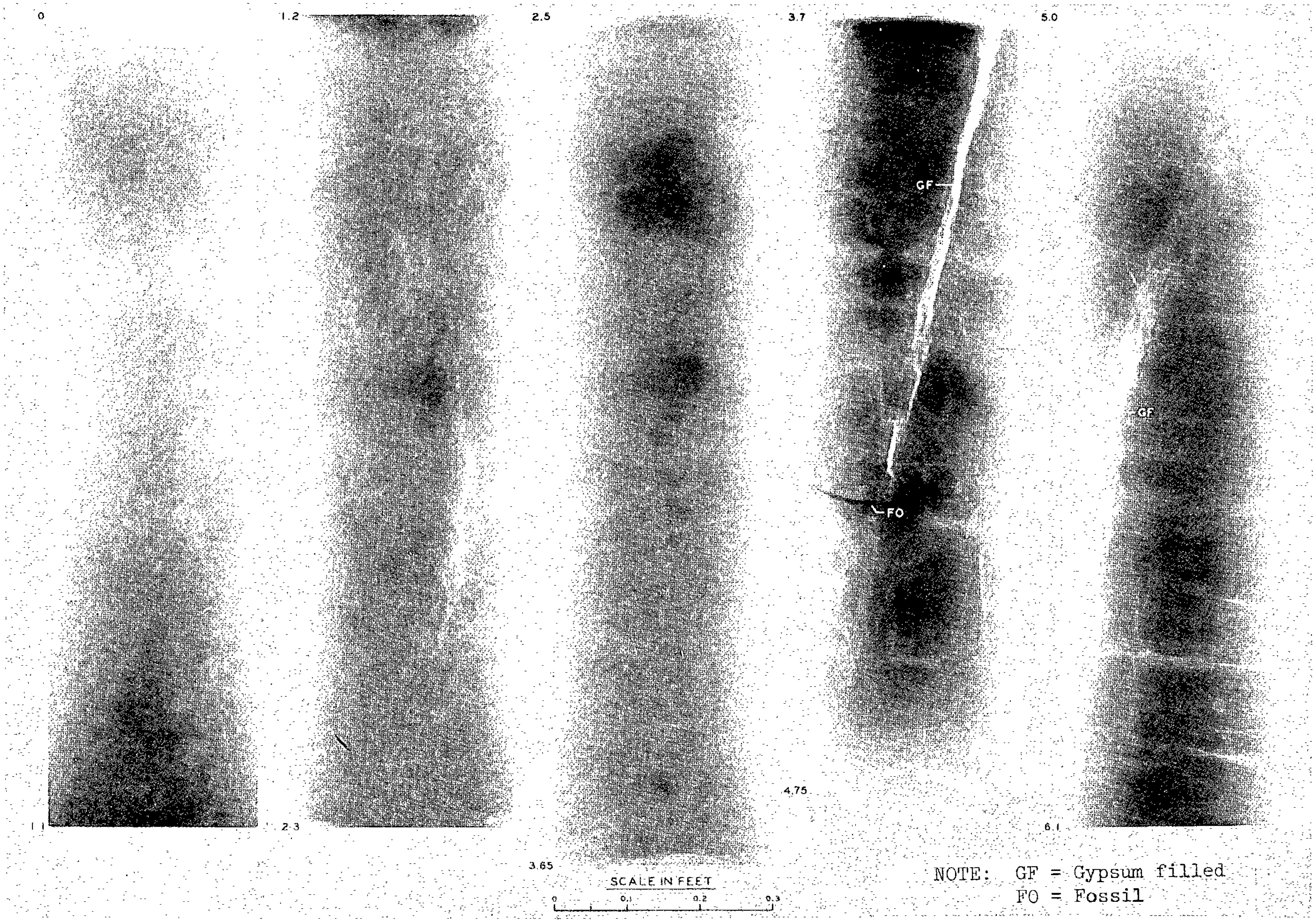


Figure 33. X-radiographs of Pierre shale from Colorado.

1973) of approximately 2 m of Pierre shale (Cretaceous age) enclosed in core barrels. The upper metre is more or less weathered and does not show the distinct bedding exhibited by the lower metre. The lower metre also exhibits numerous hairline cracks, some of which are bedding planes and others cut across the bedding. Cracks such as these are believed to be opened by the relief of constraining stresses. Two of the X-radiographs also show fractures which have been filled with gypsum. It is possible that the gypsum crystallization has contributed to the widening of the fracture and has occurred simultaneously to initiation of openings by stress relief (see Crystallization Pressures, page 150).

Rock strength. The effectiveness of cements and other forms of particle-to-particle bonds as well as intrinsic strengths of the constituent particles determines the short- and long-term material behavior. Short-term refers to behavior during ripping and compaction and long-term to behavior under in situ stresses after construction.

The natural rock strength is dependent upon the type of rock and the amount and type of discontinuities which may be present. The strength of siltstones would be a function of the amount and type of cement present, whereas clay shale strength is probably due more to other forms of bonding such as diagenetic bonds. The strength of both of these rocks would be influenced by discontinuities which also affect weathering and relate to stress relief during unloading. The intrinsic strength of constituent particles is probably more important for siltstones than for clay shales, since sandstones usually contain more angular or irregularly shaped particles of quartz or feldspar which may become broken under load. The intrinsic and overall strength of clay shales is at least partially a function of water content and loading history.

Miscellaneous factors. The following discussion concerns those factors which may contribute locally to the deterioration of embankment material but are of such nature that their contribution is usually limited. The factors considered deal with biological and deicing influences.

Biological effects. Macro- and microorganisms play an important role in the weathering processes which occur in natural weathering zones. Their influence is based on their cap ability to affect the chemical balance of the system and to mechanically alter the soil material. A common example of the effects of macroorganisms is plant roots which exhibit a hydrogen ion environment about their periphery. The hydrogen ions participate in the weathering process by effecting a cation exchange between the root and the minerals in the weathering system, thereby contributing to a chemical mineral alteration. Also, the root, if large enough, may aid in the development of cracks between mineral grains which contribute to the mechanical breakdown of the rock. Plants are not the only contributors to biological weathering; animals also affect the weathering environment by the chemical substances which they secrete or remove and by burrowing in the soil.

The familiar processes described above would usually not be applicable to embankment materials removed from below the zone of weathering and placed in a fill. Of course, this limitation would depend upon the depth of weathering in the construction area, the depth of cuts or borrow areas, and the vegetation on the embankment.

Microorganisms, particularly microflora, may participate in the chemical weathering of rocks by effecting changes in the chemical equilibrium of the system. These chemical processes may occur at depths below those ordinarily considered the zone of weathering thereby contributing to the deterioration of materials intended for use in rock fills. Apparently, the microflora is introduced at depth by the movement of groundwater downward through joints or faults in the rock.

Penner et al. (1970) have reported the heave in a pyritiferous, black, Ordovician shale in Ottawa, Canada, resulting from the alteration of pyrite and the production of gypsum and jarosite (see Cement Weathering, page 128). The mineral alteration and the resulting heave were attributed to oxidation by autotrophic bacteria. These organisms are capable of oxidizing iron and sulfur compounds and produce the same products as those attributed to inorganic processes which were described in the section on cement weathering. It may be difficult to determine in a given situation whether the alteration is due to organic or inorganic influences.

Deicing agents. The application of solid or liquid deicing agent to pavement surfaces may cause embankment material to deteriorate if these agents are allowed to enter the embankment. These agents may react with the material in the fill resulting in heave, piping, or other forms of deterioration.

The common deicing agents are sodium chloride ($NaCl$), calcium chloride ($CaCl_2$), urea $\left[CO(NH_2)_2\right]$, ethylene and propylene glycol, and other alcohols. Sodium chloride is probably the most commonly used agent due to its low cost. The effect of the agent on the fill material is a function of the nature of the agent, the mineralogy of the fill material, the application rate, and the number of applications.

The ionic concentration required to develop dispersion in a clay-water system may be difficult to predict but will depend upon the type of clay, its exchangeable cations, and the nature of the cations in the water. Generally, the degree of dispersion is increased as the ionic concentration is increased, up to a particular concentration. At this point, further increase in ionic concentration tends to flocculate the dispersed clay; this flocculation is sometimes called the common ion effect. This relationship is illustrated in Figure 34.

The effects produced by the introduction of deicing agents into the embankment involve clay mineral-water reactions; i.e., the deicing agent may affect or change the degree to which clay minerals orient the peripheral layers of water molecules or the extent to which the clay minerals imbibe water.

The discussions in previous sections have revealed the influence interlayer cations impose upon the properties of clay minerals, such as expansiveness, dispersiveness, and plasticity. Therefore, it appears that the introduction of sodium ions into the clay in the embankment material might effect a cation exchange which could produce undesirable effects in the material. On the other hand, calcium ions from calcium chloride could be beneficial and decrease the likelihood of dispersion or expansiveness. The effects produced by urea are less well known and probably depend upon the amount of dissociation that occurs. If ammonium ions (NH_4^+) are produced, some cation exchange may occur between ions on the clay and the NH_4^+ ions.

Polar, organic molecules, especially ethylene glycol and glycerol are capable of entering and occupying interlayer positions in montmorillonite. The presence of the organic molecules causes a swelling of the clay greater in magnitude than that produced by water. It would appear, therefore, that these agents may contribute to shale deterioration by expansion of the montmorillonite minerals (see Part V). Table 21 illustrates the effects which may be produced by the introduction of deicing agents into the embankment.

Table 22 illustrates the relationship between observable types of embankment distress and the deterioration factors. The deterioration factors are mechanisms which produce internal changes in the embankment material. These internal changes may then produce visible external indications of distress on the embankment. The type of distress most frequently associated with a particular deterioration factor has been listed in Table 22. Generally external distress will result from the interaction of several deterioration factors which may be operating simultaneously or distress may initially be caused by a particular factor which, in turn, leads to the development of another factor. For example, the removal of cements by weathering may lead to settlement or shear failure. This same weathering may also result in the development of fines, which will lead to dispersiveness or expansion.

Expansive clays. The mechanism of expansion will be manifest by horizontal or vertical heave on the upper surface and sides, respectively, of the embankment. Settlement and cracking also may occur on both top and sides during dry periods.

Dispersive clays. This factor may produce piping and other erosion features within and on the embankment. The similarities between expansiveness and dispersion may also result in the occurrence of distress features listed for expansive clays.

Clay weathering. This deterioration factor results in a breakdown of grain contacts between the clay minerals. The loss of grain-to-grain

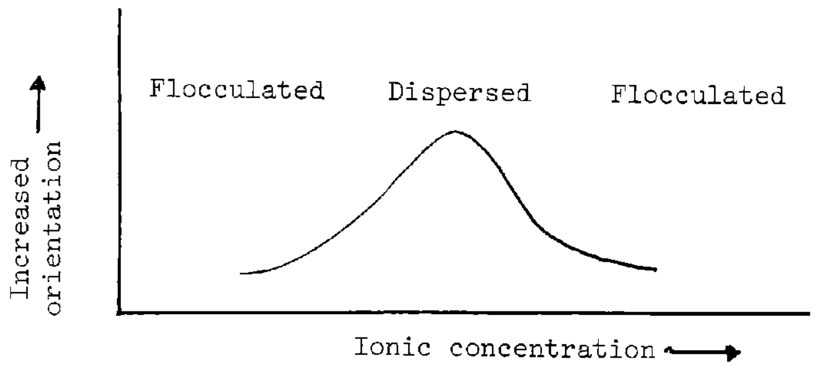


Figure 34. Relationship between degree of dispersion and ionic concentration.

Table 21. Effects of deicing agents on clay minerals in embankments.

Agent	Effect	Remarks
Sodium chloride ($NaCl$)	Dispersion	Cation exchange between Na^+ and ions on clay.
Calcium chloride ($CaCl_2$)	None	May be beneficial by exchanging Ca^{++} for Na^+ on clay.
Urea	Negligible	May be important if NH_4^+ ions are present
Ethylene and propylene glycol	Expansion	
Other alcohols	Negligible	

Table 22. Relationships between observable types of distress and deterioration factors.

Factors	Type of Distress				
	Heave	Piping	Settlement	Cracking	Shearing
Expansive clays	XX	--	X	XX	--
Dispersive clays	--	XX	--	--	--
Clay weathering	X	X	X	X	X
Cement weathering	--	--	X	--	X
Crystallization pressures	X	--	--	--	--
Unloading	--	--	X	--	X
Strength	--	--	X	--	X

contacts results in a softening and loosening of the material which may then be subjected to heaving, piping, settlement, cracking, or shearing.

Cement weathering. The removal of cements by weathering may result in a breakdown of grain contacts between nonclay mineral constituents. The breakdown and accompanying softening may then lead to settlement and shearing.

Crystallization pressures. This factor may contribute to embankment distress by disrupting grain-to-grain contacts and thereby producing heave or spreading of the embankment.

Unloading. This factor is another mechanism by which grain-to-grain contacts may be broken and as a result, settlement and shearing may occur.

Strength. The overall rock strength is generally related to the other, previously described mechanisms and affects the settlement and shearing resistance of the embankment.

VI. REVIEW OF LABORATORY EXAMINATION AND TESTING TECHNIQUES

The testing of shales intended for use in embankments should provide answers to three basic questions:

- a. Should the material be placed as rock, soil, or at all?
- b. If the material is placed as rock, what likely forms of deterioration will the fill experience?
- c. What soillike or rocklike properties does this material possess which will influence the design?

The first question involves the durability and hardness of the material and whether it can be conveniently ripped and placed as a soil. The durability and hardness must be related to the amount of compaction and the amount and type of cement or bonding possessed by the material. The second question concerns the degree of durability exhibited by the material and how this durability may be expected to change with time. The third question concerns such parameters as grain size, plasticity, moisture-density relationships, strength, and settlement or consolidation factors.

These three questions relate to the material's resistance to three basic modes of deterioration which may be categorized as chemical, physicochemical, and physical. The chemical mode includes breakdown of primary mineral components by chemical weathering; the physicochemical mode involves clay mineral hydration and swelling or dispersion; and the physical mode is related to rock strength.

Laboratory testing of clays and shales may be conveniently subdivided into the following three categories: mineralogical and petrological tests, soil mechanics tests, and durability tests. These three types of tests provide the designer with basic information on amount and nature of rock constituency (mineralogical and petrological tests); classification, plasticity, and strength (soil mechanics tests); and durability and soundness (durability tests). The following sections provide a brief description of current testing procedures and an evaluation of their applicability to embankment design.

Mineralogical and Petrological Tests

These tests provide information on the mineralogy, the microstructure, and the manner in which the mineral constituents are arranged in the rock (fabric). The techniques which provide this information are X-ray diffraction (XRD), the polarizing light microscope, X-radiography, and certain chemical tests.

XRD. XRD techniques can be used to identify the mineral constituency of the rock and may be quantified such that the relative proportions of the various mineral constituents can be determined. For most work, however, qualitative analysis is sufficient. The designer is concerned with two aspects of the mineralogy: the type of clay present in the clay-size fraction, and the nature of the bonding, cementing, or accessory minerals. These two aspects relate to potential short- and long-term problems in that the presence of swelling clays such as montmorillonite, including mixed-layer combinations, suggests deterioration by surface heave or dispersion, whereas the determination of mineral cements provides a clue to the long-range weathering susceptibility of the material.

Three XRD procedures provide the required data: (a) X-ray of randomly oriented powders of the bulk sample, (b) analysis of the oriented, sedimented clay-size fraction, and (c) if the X-ray diffraction analysis reveals d-spacings in the range of 12 to 15 Å, the sample should be solvated with ethylene glycol or a similar organic liquid* to identify montmorillonite. When solvated, the montmorillonite d-spacing increases to 17 Å. The X-ray analysis of the bulk sample identifies the nonclay mineral constituents such as calcite, gypsum, pyrite, and quartz. The analysis of the sedimented material identifies the clay minerals. XRD will not ordinarily detect the presence of amorphous material in the samples.

The diffractograms of the sedimented (including solvated) clay should be examined carefully to ensure the identification of swelling

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* Glycerol is also commonly used.

clay minerals. Particular attention should be addressed to the detection of swelling clays in mixed-layer combinations with other clay minerals.

Figure 35a illustrates the diffractogram of sedimented clay containing illite and chlorite; Figure 35b illustrates no change in d-spacing after solvation; Figure 35c illustrates a mixture of illite and montmorillonite; and Figure 35d exhibits a d-spacing shift of the montmorillonite peak to approximately 17 Å after solvation.

Figure 36 represents diffractograms of mixed-layer combinations of illite and montmorillonite. Randomly mixed-layer combinations composed primarily of illite exhibit a "shoulder" on the low 2θ side of the 10 Å peak (Figure 36a). Upon solvation, the shoulder becomes larger indicating that montmorillonite has become expanded (Figure 36b). A regular, 50-50 mixed-layer combination of illite and montmorillonite will exhibit a first-order d-spacing representing the sum of 10 and 15 Å (approximately 25 Å. A second-order d-spacing of 25 Å/2 (12.5 Å) may also be present (Figure 36c). Upon solvation the montmorillonite will expand to approximately 17 Å yielding a first-order d-spacing of 27 Å and a second-order d-spacing of 13.5 Å (Figure 36d).

Mineral identification by XRD provides the designer with a "yes" or "no" set of criteria in that it answers the question: Does this embankment material contain minerals susceptible to deterioration by expansion, dispersion, cement weathering, or possibly crystallization pressure? If the answer is "no," these factors are eliminated and clay mineral weathering and unloading should be suspected as possible factors.

Polarizing light microscopy. The polarizing light microscope may also be useful in gaining a better understanding of the mineralogy of the silt- and sand-size fractions from grain mounts and of the fabric from thin sections. The former permit examination of the individual grains from which grain shape and surface characteristics can be identified. The thin section is not very helpful for mineral identification but does detect the arrangement of the minerals and microstructures.

X-radiography. The most effective method of examining overall fabric in argillaceous rocks and sediments is X-radiography. These "shadow pictures" reveal details hidden from direct observation,

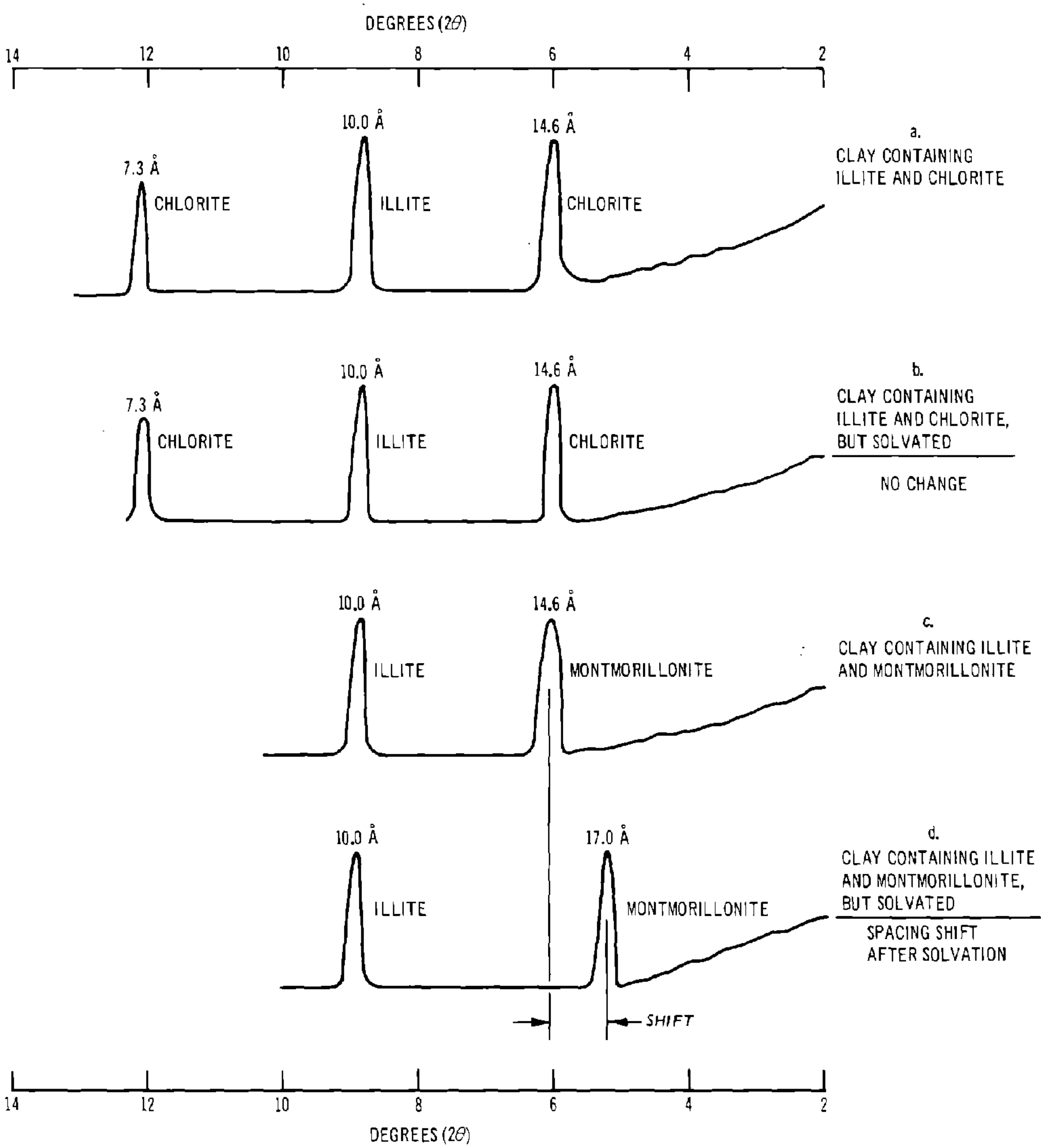

Figure 35. X-ray diffractograms of clay mixtures.

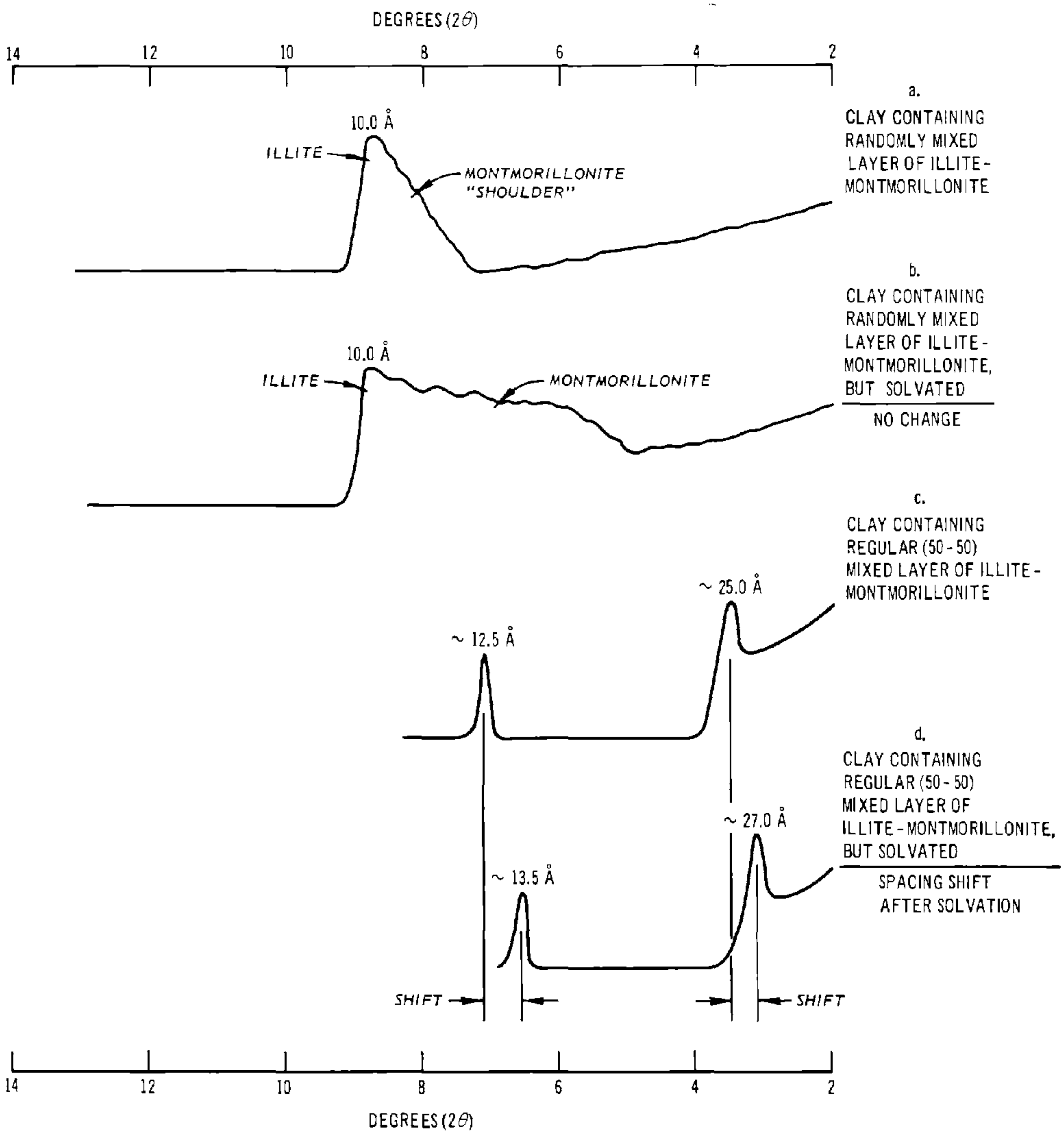

Figure 36. X-ray diffractograms of clays containing mixed-layer illite-montmorillonite.

particularly bedding, concretions, and hairline cracks, all of which may affect the strength and/or weatherability of the material.

Chemistry. Certain aspects of rock chemistry may provide useful information regarding material deterioration. Of particular importance with respect to dispersion is the identification of exchangeable cations on the clays. This identification may be accomplished by cation exchange capacity (CEC) tests (see Dispersive Clay Minerals, page 125, and Durability Tests below). Another aspect is the pH environment of the shale, either in situ or after slaking or ultrasonic tests. The in situ pH may be indicative of the nature of the natural weathering environment and thus may suggest the presence of decomposition products which also may occur within the embankment (see Cement Weathering, page 128). The pH after durability testing may be an indicator of the chemical deterioration occurring after physical breakdown of the sample and may also be similar to the chemical weathering environment in the embankment (see Part VIII).

Durability Tests

The tests described in this section are used in an attempt to measure the degree of rock induration by imparting a known amount of energy into the sample and observing the response of the sample to this energy. The energy is imparted to the sample in an attempt to simulate or at least approximate the more rigorous conditions under which the material will exist when placed in the embankment. Also, these tests present some information on the rippability of the natural material.

Those tests which appear to provide an indication of material suitability are: slaking, ultrasonic disaggregation, hardness, fissility, and sulfate soundness.

Slaking. The tests included in this category measure the response of the material to moisture either through an aqueous or a moisture-laden air environment. The basis for these tests is that weakly cemented or compacted argillaceous materials will when exposed to moisture imbibe water which will cause disaggregation. The disaggregation appears as a powdering, spalling, or flaking of the sample surface, or separations along bedding planes and may cause either a partial or complete breakdown of the material. The quantity of material slaked or removed from the samples may be determined by weighing, thus giving a numerical basis to the test.

Moriwaki (1974) studied the slaking behavior of artificially prepared samples of kaolinite, illite, and montmorillonite. Each clay mineral type was tested under a variety of environmental conditions, and each was prepared with absorbed calcium and absorbed sodium ions. Mixtures of these three clay minerals were also prepared and tested. Four different slaking modes were identified: swelling slaking (a softening and swelling throughout the material); dispersion slaking (a microscopic disintegration of the material accompanied by particle dispersion in the water); surface slaking (a microscopic disintegration proceeding from the outside and extending inward); and, body slaking (a miscroscopic disintegration proceeding from the inside toward the outer portion of the sample). Table 23 relates the general slaking mode to mineralogy.

Sherard's technique for the identification of dispersing clays has been discussed on page 125. This method provides a quantitative chemical measure of susceptibility to deterioration for some materials.

A simple straightforward method is to prepare a cube of sample approximately 1 in. on each side and place the cube in a beaker of distilled water. Observations over a period of 2 or 3 min will reveal whether or not the material is readily slakeable. Shale materials exhibiting any powdering or form of deterioration within this time frame would be expected to behave poorly when placed in the fill and exposed to moisture. It should be suspected that these materials possess one or more of the following characteristics: expansiveness, dispersiveness, large clay-size fraction, or weak cementation or bonding.

The presence of dispersive or expansive materials may also be detected by slaking samples in the presence of ethylene gylcol or similar polar organic liquid. The glycol is absorbed in interlayer positions on expansive clays and enhances their expansion and/or dispersion. This

Table 23. Slaking versus clay mineralogy (after Moriwaki, 1974).

	Slaking Type			
Mineralogy	Swelling	Dispersion	Surface	Body
Kaolinite		Na-Kaolinite		Ca-Kaolinite
Illite		Na-Illite		Ca-Illite
Montmorillonite	Na-Montmorillonite (1N NaCl)	Na-Montmorillonite	Ca-Montmorillonite	

procedure appears to be useful in excluding nondurable material during initial testing phases.

Different results from simple slaking tests will be obtained depending upon the water content of the sample. Materials tested above or at natural water content will not exhibit the deterioration found in oven- or air-dried samples. Thus, it may be necessary to consider the construction technique and the in-place environment when conducting the slaking test. A more rigorous form of this test involves alternate wetting (slaking) and drying as described under Corps of Engineers Experience and Practices; this method may approximate the effects of seasonal changes within the embankment.

Shale materials which do not slake appreciably when merely placed in water may require some form of energy input to ensure that they will behave in a satisfactory fashion in the fill. Franklin and Chandra (1972) describe a slake-durability test which involves the rotation of the sample in a wire-mesh basket which is partially immersed in water. The sample is oven-dried at 105°C for 2 to 6 hours then tested for 10 min at 20 rpm. The ratio of the weight retained after testing for two cycles to the weight of the original sample expressed as a percentage is defined (Equation 8) as the slake-durability index (I_D) and is a measure of the susceptibility of the material to deterioration in the presence of water.

$$\text{Slake-durability index} = \left(\frac{\text{Dry weight after two cycles}}{\text{Dry weight before testing}}\right) 100 \qquad (8)$$

Slake-durability tests were used by Deo et al. (1973) to classify shales in Indiana for use in embankments. The tests were conducted on ovendried and soaked samples, and the results compared with those from other tests including sodium sulfate soundness, fissility, and mineralogy. Fissility was found to be an indicator of deterioration since the more fissile material suffered more slaking than the more massive rocks.

Gamble (1971) also studied slake durability in terms of mineralogy and plasticity. He concluded that the correlations found between slaking characteristics and mineralogy and plasticity were sufficient to establish a classification scheme (Figure 28). Gamble believed that this slaking procedure is suitable for classifying construction materials.

Ultrasonic disaggregation. This technique has been used by Laguros (1972a and b) and Reidenouer et al. (1974) as a measure of the potential deterioration of argillaceous rocks. The procedure involves the application of high-frequency sonic energy to small samples of rock submerged in an aqueous medium. The sonic energy imparted to the sample results essentially in slaking, the extent of which is dependent upon the durability of the material as well as the test duration.

Laguros compared Atterberg limits and the amount of fines produced after 2 yr of outdoor atmospheric weathering with these same parameters exhibited by similar materials after 1 and 8 hr of ultrasonic disaggregation. The results of these tests and their comparison were that 1 hr of ultrasonic disaggregation produced an amount of material smaller than 5 micorometres which more closely approached the amount produced by 2 yr of outdoor weathering than did 8 hr of ultrasonic disaggregation. Neither the 1- nor the 8-hr test appeared to adequately predict the plasticity developed after 2 yr of artificial weathering.

Reidenouer et al. compared the effects of ultrasonic disaggregation on Pennsylvania shales and the relationships between these effects and mineralogy, soundness, and other geological and soil mechanics properties. They concluded that ultrasonic disaggregation is a reliable measure of durability; however, they felt that additional field experience would be necessary before the test results could be related to actual field behavior.

Rock hardness. The toughness, soundness, and long- or short-term resistance to deterioration may be related to the hardness of the shale, i.e. resistance to scratching or penetration. Resistance to scratching or penetration is related to the audible effects produced when the rock is struck with a hammer. The harder, tougher, metamorphic rocks such as slates ring when hit, whereas the clay shales and some siltstones have a duller, lower pitch sound. The mere act of breaking the rock in the field may provide some idea of rock hardness. Resistance to scratching and possibly penetration is dependent upon the amount of clay present in the rock, e.g., rocks rich in clay minerals are more likely to be easily scratched with the fingernail, whereas rocks composed primarily of silt-sized quartz are more resistant although, if they are poorly cemented or bonded, the individual quartz grains may become detached by scratching.

Aughenbaugh* has reported that hardness values determined from Shore scleroscope measurements have been useful in predicting deterioration of roof shales in coal mines. The durability of these rocks appeared to be related to water content and mine humidity. The hardness values were judged to correlate with the water content of the rocks.

Fissility. Deo et al. (1973) used fissility as a means of categorizing Paleozoic shales in terms of durability and strength. They quantified the dimensions of broken shale fragments and found that those fragments with the highest fissility exhibited the lowest values from slake-durability tests.

Sodium sulfate soundness. Deo et al. (1973) and Reidenouer et al. (1974) generally concluded that this soundness test procedure (including modifications) is too severe for all but the most durable rocks and, therefore, is not a suitable means for testing a wide variety of argillaceous rocks.

Soil Mechanics Tests

Those tests commonly associated with testing of soils and sediments have also been applied to shales intended for foundation and fill construction. The logic for this application has been that the relatively soft and soillike shales should behave in a fashion similar to that of soils when loaded. The tests include moisture-density, grain-size analyses, Atterberg limits, and various strength and compression tests. The

* N. B. Aughenbaugh (1975), personal communication, University of Missouri, Rolla.

significance of these tests may be dependent upon the method of construction in that grain-size analyses and plasticity have more meaning when the rock is to be broken down and compacted as a soil. For material used in a rock fill, these two tests relate more to compositional factors which may indicate future material degradation.

Grain-size analysis. Determination of the grain-size distribution of cemented or highly compacted argillaceous rocks is difficult, and the results may be unreliable and of minor importance in determining the weatherability of the material used in rock fills. The designer is interested in the amount of clay-size particles present in order to estimate the effects of swelling, dispersion, or clay mineral weathering; however, the cementation and agglomeration of the individual clay platelets may prevent this determination. The amount of effort required to completely disaggregate the rock is more important since this effort must be related to rock durability and soundness.

Atterberg limits. Disaggregation problems may not permit the determination of meaningful Atterberg limits for indurated material intended for rock fills. However, softer material may yield reliable data which may contribute to the design of fills constructed by breaking down the rock and compacting it as soil. Generally, high plasticity indexes and liquid limits suggest that the material is likely to behave poorly in a rock-fill embankment.

It seems logical that the Atterberg limits as well as grain-size distribution of the material after weathering in an embankment will change. Laguros (1972b) has shown that artificial weathering of Oklahoma shales has produced definite increases in both the Atterberg limits and the amount of clay-size material. Therefore, these tests conducted on fresh material may not provide information necessary to predict material deterioration.

Moisture-density tests. Materials to be placed as soil are usually subjected to moisture-density tests to determine required water contents and compaction in the field. However, the suitability of these tests for material intended as rock fill may depend upon the size of the material when broken down. It is apparent that materials which break down to particles having diameters larger than 4 or 6 in. is not amenable to moisture-density testing. However, for materials with smaller sized particles, compaction tests may provide reliable and necessary data for design and also give a qualitative estimation of further material breakdown during compaction.

Triaxial testing. Triaxial tests of undisturbed core samples and of material in the as-placed condition may provide the designer with strength data which could contribute to the embankment design. The triaxial tests of undisturbed cores yield some information on the overall strength of the rock and also on the feasibility of placing the material as rock as opposed to a soil. Tests of material intended to be placed as rock fills may be useful; however, these tests may be impractical due to size limitations imposed by existing triaxial apparatus. Shales which are to be placed as soils should have consolidated undrained $\overline{R}$ tests performed in order to adequately evaluate the as-placed strength of the material.

The usefulness of triaxial testing also depends on the extent to which the fractures and other structural features in the tested material model similar discontinuities present in the rock placed in the embankment.

Miscellaneous Tests

The following tests have been proposed for the identification and determination of physical properties of argillaceous material.

Methylene blue absorption (MBA). Nettleton (1974) investigated the bases and causes of rock durability as determined from slake-durability tests and related these data to MBA values. The MBA value in milliequivalents per 100 g of dry soil is defined as:

$$MBA = \frac{100}{W} TN \tag{9}$$

where

W = dry weight of soil, g

T = volume of titrant, cc

N = normality of dye, milliequivalents/cc

Nettleton concluded that high MBA values correlate well with high losses from slake-durability tests although he recommended that both tests be performed.

Dielectric dispersion. Basu and Arulanandan (1974) developed a procedure for predicting potential swell of argillaceous materials by measuring the dielectric dispersion. Dielectric dispersion is the maximum change in the material dielectric constant measured at two extreme alternating current frequencies. The dielectric constant does not change below or above these two frequencies. The dielectric dispersion was reported to be correlatable with specific clay mineral types and with swell potential for monlithified materials. Its effectiveness with rocklike materials is not known.

Discussion

The testing procedures discussed represent current techniques for the laboratory investigation of shale materials. The test procedures are summarized in Table 24 and are correlated with the deterioration factors they attempt to measure and opinions as to their suitability for testing argillaceous rocks as determined by this study.

Although several of the tests in Table 24 are recommended, it is apparent that most suffer from at least one and possibly several drawbacks, i.e. they are not quantitative, experience with them is limited, they are severe, and they are generally impractical. These detracting factors are indicated in Table 24 and discussed below.

The quantitative tests such as XRD only provide the designer with an insight as to the composition of the material. Such results, however, do help explain results of other tests and may also alert the designer to specific material behaviors such as expansiveness.

The experience gained in using the tests involves correlation of test results with field performance and, having made these correlations, with a wide variety of materials representing different petrologic

Table 24. Relationships between deterioration factors, laboratory tests, and recommendations.

Deterioration Factors Test	PC Expansive Clays	PC Dispersive Clays	C, PC Clay Weathering	C Cement Weathering	C, P Crystallization Pressures	P Unloading	P Strength
	Type of Deterioration						
X-ray diffraction	1,3,5	1,3,5	1,3,5	1,3,5	1,3,5	--	--
X-radiography	--	--	--	--	4,5	4,5	--
Polarizing light microscope	--	--	--	3,5	--	3,5	--
pH	--	--	1,3,6	1,3,6	1,3,6	--	--
Jar slaking	4,5	5	--	--	--	--	--
Slake durability	1,6,7	1,6,7	--	--	--	--	1,6,7
Exchangeable sodium percentage and CEC	1,3,6	1,3,6	--	--	--	--	--
Ultrasonic disaggregation	4,6,7	4,6,7	4,6,7	--	--	--	4,6,7
Shore hardness	--	--	--	--	--	--	4,6
Fissility	--	--	--	--	--	5,6	--
Sulfate soundness	--	--	2,7	--	--	--	--
Mechanical analysis	1,3,8	1,3,8	--	--	--	--	--
Atterberg limits	1,3,8	1,3,8	--	--	--	--	--
Moisture-density	--	--	--	--	--	--	1,3,8
Triaxial	--	--	--	--	--	--	1,3,8
MBA	4,6	4,6	--	--	--	--	--
Dielectric dispersion	4,6,8	4,6,8	--	--	--	--	--

Notes: Types of deterioration measured

PC = physicochemical
C = chemical
P = physical

Recommendations

1. Recommended test
2. Not recommended test
3. Routine test
4. Further study required

Limitations

5. Not quantitative
6. Limited experience
7. To severe
8. Impractical

compositions and geologic ages. This drawback is particularly apparent in slake-durability tests, as well as in others.

The extreme severity (or lack of it) poses problems in discrimination between tested materials. This drawback was shown to apply for sulfate soundness tests and may also apply for ultrasonic disaggregation and slake-durability tests.

It appears that no single test predicts the amount of material breakdown with time. Furthermore, it appears that no one test successfully covers all three forms of deterioration: chemical, physicochemical, and physical. Thus, several tests are required.

Selected Tests

The preceding test procedures as well as other standard and non-standard tests were considered in detail prior to initiation of the variability testing program in this study (Part VII). Tests were selected based upon the following factors: previous success, general acceptability, time requirements, costs, simplicity of procedure, and indication of shale variability.

The tests which satisfied these factors also provided a measure of the three modes of deterioration. The tests selected for use in the variability study (Part VII) and their applicability for general use are as follows. XRD: chemical, physicochemical, physical deterioration, and identification; jar slaking: physicochemical; slake durability: physicochemical and physical; fissility: physical; shore hardness: physical; pH (in conjunction with the slake-durability tests): chemical, physicochemical, and physical.

VII. PROBLEM SHALES AND THEIR VARIABILITY

A major part of the field effort was directed toward describing the stratigraphic setting and variability of intrinsic properties of shales. Emphasis was placed on problem shales because of the limited time available for study. The age range of shale was limited to the Paleozoic age for the same reason. Within problem shale formations, numerous physical properties and mineralogical and geologic parameters were identified and in most cases quantified for a group of 69 samples. The problem shales sampled include nine not listed in Table 4: these later additions resulted from identifications made during visits and field trips with State highway personnel.

The samples for study and number obtained within broad age groups are as follows:

Geologic Age	Location	Number
Pennsylvanian	Tennessee	4
Pennsylvanian	Western Kentucky	7
Pennsylvanian	Eastern Kentucky	3
Mississippian	Indiana	5
Mississippian	Western Kentucky	2
Mississippian	Eastern Kentucky	3
Devonian	Kentucky, Tennessee, Ohio	5
Devonian	Virginia	3
Silurian	Eastern Kentucky	3
Silurian	Western Kentucky	3
Ordovician	Kentucky, Indiana, Ohio	17
Ordovician	Virginia	1
Cambrian	Tennessee	13

Procedures for Variability Tests and Examinations

Sixty-nine samples were examined for variability in three general ways: visual inspection, mineralogy, and laboratory testing (Tables 25-27). Samples were described in a systematic manner to reveal any consistent differences in color, structure, fragment shape, and texture. Later, these samples were subjected to relatively simple indexing tests from among the many available (Part VI) that, in combination, approximate processes involved during shale breakdown in embankments (Table 26).

Table 25. Shale sample descriptions.

Shale Number	Natural Water Content, Percent	Color	Structure	Fragment Shape	Texture*	Formation
			Kentucky			
KY 1	8.8	Greenish-gray	Bedded	Platy	With silty beds	Osgood
KY 2	12.6	Greenish-gray	Bedded	Small pieces	Soft, sandy	Osgood
KY 3	18.7	Yellowish-gray	Indistinct	Irregular	Soft clay	Osgood
KY 4	4.1	Brownish-black	Thinly fissile	Platy	--	New Albany
KY 5	9.5	Greenish-gray	Indistinct	Pebbles	--	New Providence
KY 6	4.1	Medium gray	Laminated	Tabular	Siltstone	New Providence
KY 7	3.5	Olive gray	Faintly bedded	Thinly platy	Sandy	Tradewater
KY 8	13.6	Brownish-gray	Indistinct	Irregular	--	Tradewater
KY 9	12.9	Medium gray	Indistinct	Irregular	Soft, silty	Tradewater
KY 10	12.4	Medium yellowish-brown	Indistinct	Blocky	Silty	Tradewater
KY 11	6.5	Medium gray	Indistinct	Platy	--	Tradewater
KY 12	7.8	Olive black	Laminated	Thinly platy	Silty	Tradewater ?
KY 13	9.9	Medium dark gray	Indistinct	Tabular	--	Tradewater ?
KY 14	5.9	Dark greenish-gray	Indistinct	Platy	--	Crab Orchard
KY 15	10.9	Greenish-gray	Fissile	Tabular	Silty	Crab Orchard
KY 16	13.7	Greenish-gray	Faintly bedded	Blocky	--	Crab Orchard
KY 17	18.6	Greenish-gray	Indistinct	Blocky	Silty	Preacherville
KY 18	3.0	Brownish-black	Thinly fissile	Platy	--	New Albany
KY 19	6.0	Greenish-gray	Fissile	Platy	Siltstone	Nada
KY 20	5.1	Dark gray	Indistinct	Thinly platy	--	Breathitt
KY 21	5.3	Greenish-gray	Indistinct	Tabular	Silty	Nada ?
KY 22	5.5	Dark reddish-brown	Fissile	Thinly platy	Siltstone	Crider Clay
KY 23	8.0	Dark greenish-gray	Fissile	Tabular	--	Crider Clay ?
KY 24	4.4	Dark greenish-gray	Indistinct	Platy	Silty	New Providence
KY 25	8.4	Greenish-gray	Faintly bedded	Blocky	--	Kope
KY 26	12.1	Greenish-gray	Indistinct	Prismatic	--	Kope
KY 27	7.5	--	--	--	--	Kope
KY 28	11.4	Dark greenish-gray	Indistinct	Prismatic	--	Kope
KY 29	8.2	Greenish-gray	Faintly bedded	Blocky	--	Kope
KY 30	6.3	Dark greenish-gray	Faintly bedded	Blocky	--	Kope
			Tennessee			
TN 1	7.6	Yellowish-gray	Indistinct	Platy	--	Pumpkin Valley
TN 2	3.9	Greenish-gray	Indistinct	Tabular	Siltstone	Nolichucky
TN 3	10.8	Light olive gray	Indistinct	Tabular	Silty	Nolichucky
TN 4	11.9	Light olive gray	Indistinct	Tabular	--	Nolichucky
TN 5	7.8	Pale yellowish-brown	Indistinct	Platy	--	Nolichucky
TN 6	6.8	Light olive gray	Indistinct	Blocky	Silty	Nolichucky
TN 7	12.5	Yellowish-gray	Faintly bedded	Tabular	Silty	Nolichucky
TN 8	11.2	Pale yellowish-brown	Indistinct	Tabular	--	Nolichucky
TN 9	7.7	Grayish-orange pink	Indistinct	Prismatic	Silty	Nolichucky
TN 10	10.2	Pale yellowish brown	Indistinct	Platy	--	Nolichucky
TN 11	8.3	Light olive gray	Laminated	Tabular	Silty	Nolichucky
TN 12	--	Medium bluish-gray	Bedded	Platy	Sandy	Rome
TN 13	1.2	--	--	--	--	Rome
TN 14	4.8	Medium dark gray	Faintly bedded	Irregular	--	Redoak Mountain
TN 15	--	Olive black	Bedded	Tabular	Siltstone	Redoak Mountain
TN 16	--	Medium dark gray	Indistinct	Tabular	Siltstone	Graves Gap
TN 17	3.3	Medium dark gray	Indistinct	Irregular	--	Cross Mountain
TN 18	--	Olive black	Thinly fissile	Platy	--	Chattanooga
			Ohio			
OH 1	8.1	Dark greenish-gray	Indistinct	Prismatic	Silty	Fairview
OH 2	--	Dark greenish-gray	--	Irregular	--	Kope
OH 3	--	--	--	--	--	Kope
OH 4	--	Greenish-gray	Indistinct	Tabular	Silty	Elkhorn
OH 5	--	Grayish-red	Faintly bedded	Prismatic	--	Elkhorn
OH 6	--	Dark yellowish-brown	Indistinct	Blocks	--	Bedford
OH 7	--	--	Faintly fissile	Platy	--	Ohio
			Indiana			
IN 1	--	Greenish-gray	Faintly bedded	Tabular	Limestone	Whitewater
IN 2	--	Dark greenish-gray	Indistinct	Irregular	--	Dillsboro
IN 3	--	Medium dark gray	Laminated	Tabular	Sandy	Palestine
IN 4	--	Medium dark gray	Fissile	Irregular	--	Palestine
IN 5	--	Dark greenish-gray	Faintly bedded	Tabular	--	Kope
IN 6	--	Dark gray	Laminated	Irregular	Silty	Palestine
IN 7	--	Medium dark gray	Faintly bedded	Tabular	--	Palestine
IN 8	--	Medium dark gray	Faintly bedded	Tabular	Sandy	Palestine
IN 9	--	Dark greenish-gray	Indistinct	Irregular	--	Dillsboro
IN 10	--	Dark gray	Finely laminated	Platy	--	Palestine
			Virginia			
VA 1	--	Dark gray	Laminated	Tabular	Siltstone	Brallier
VA 2	--	Dark gray	Bedded	Tabular	--	Millboro
VA 3	--	Medium dark gray	Indistinct	Tabular	--	Millboro
VA 4	--	--	--	--	--	Edinburg

* Other than shaly.
? = Tentative formation identification.

Table 26. Shale test results.

Shale Number	Water Content Before Test, Percent 1	2	3	Avg	Jar Slake Index Damp	Dry	Slake Durability Index, percent 1	2	3	Avg	Scleroscope Index Parallel Damp	Dry	Perpendicular Damp	Dry	Test Water, pH* Cycle 1	Cycle 2	Avg
									Kentucky								
KY 1	6.9	8.4	5.9	7.1	6	2	30	5	10	15	--	--	15	26	8.6	--	--
KY 2	21.1	19.6	--	20.3	3	1	11	6	--	9	--	--	--	--	9.0	9.4	9.2
KY 3	18.4	18.0	18.4	18.3	4	2	2	2	29	11	--	--	--	--	9.6	9.2	9.4
KY 4	4.1	4.7	3.2	4.0	6	6	99	99	99	99	25	29	40	54	6.8	4.5	5.6
KY 5	6.2	7.0	7.7	7.0	6	4	82	73	78	78	14	20	18	27	7.7	7.7	7.7
KY 6	3.6	3.5	3.0	3.4	5	6	92	93	95	93	24	27	--	--	9.3	9.0	9.2
KY 7	5.4	5.2	5.8	5.5	6	6	89	90	89	89	--	--	14	22	6.8	7.3	7.0
KY 8	17.0	8.3	8.2	11.2	6	3	15	11	6	11	--	--	--	--	6.1	6.5	6.3
KY 9	7.2	7.7	--	7.5	5	1	0	0	--	0	--	--	10	21	3.0	--	--
KY 10	11.8	12.3	10.9	11.7	6	4	62	48	47	52	12	18	--	20	6.7	6.3	6.5
KY 11	6.0	6.1	6.7	6.3	5	4	25	21	20	22	--	--	18	21	6.1	6.1	6.1
KY 12	6.4	--	--	--	6	-	84	--	--	84	--	--	--	--	7.8	7.2	7.5
KY 13	9.0	9.3	8.8	9.0	6	4	51	55	54	53	13	14	17	31	7.0	7.3	7.2
KY 14	3.1	2.7	4.1	3.3	6	5	70	84	42	65	--	--	14	--	8.1	8.2	8.2
KY 15	8.3	8.5	8.2	8.3	6	2	5	5	2	4	12	15	18	23	7.3	7.3	7.3
KY 16	15.3	15.9	13.2	14.8	6	1	0	0	0	0	11	14	--	--	7.6	7.1	7.4
KY 17	6.6	7.1	7.1	6.9	6	1	1	1	1	1	14	17	18	26	8.6	--	--
KY 18	3.0	4.6	4.0	3.9	6	6	99	99	98	99	--	--	35	37	4.9	5.3	5.1
KY 19	4.4	4.7	4.5	4.5	6	6	95	93	96	95	--	--	22	43	7.4	7.5	7.4
KY 20	4.7	5.0	4.8	4.8	6	4	92	91	90	91	--	--	20	29	7.0	7.4	7.2
KY 21	5.4	5.1	4.3	4.3	6	5	78	88	94	87	22	--	22	26	7.8	7.8	7.8
KY 22	5.2	5.5	5.3	5.3	6	4	75	78	64	72	--	--	20	25	7.2	7.6	7.4
KY 23	8.0	9.4	8.4	8.6	6	1	9	2	6	6	13	--	17	32	9.2	7.0	8.1
KY 24	4.7	4.6	4.3	4.5	6	4	76	72	73	74	--	--	20	32	8.1	8.4	8.2
KY 25	8.8	8.0	8.5	8.4	6	4	63	49	62	58	11	17	16	26	8.5	--	--
KY 26	8.4	8.3	8.5	8.4	5	5	59	60	37	52	10	14	16	27	8.6	8.4	8.5
KY 27	6.7	6.5	6.7	6.6	4	3	68	73	61	67	14	20	20	27	9.0	9.0	9.0
KY 28	10.5	10.6	11.1	10.7	6	1	28	26	22	25	13	15	17	25	8.8	8.6	8.7
KY 29	8.9	8.8	8.4	8.7	6	2	26	22	25	24	11	19	16	29	8.2	8.8	8.5
KY 30	5.6	5.9	5.8	5.8	6	4	17	17	19	18	17	22	22	25	7.4	8.7	8.0
									Tennessee								
TN 1	8.3**	5.2	7.1	6.2	6	6	41**	75	73	74	--	--	16	26	6.6	6.3	6.4
TN 2	3.2	3.0	3.1	3.1	6	5	93	92	93	93	25	32	--	--	6.9	6.8	6.8
TN 3	10.3	7.6	8.7	8.9	5	3	63	74	70	69	--	--	24	30	6.4	6.2	6.3
TN 4	10.3	10.3	10.2	10.3	6	3	55	56	54	55	15	24	16	33	6.3	5.6	6.0
TN 5	7.0	6.8	9.0	7.6	5	2	75	76	65	71	12	18	12	26	6.0	7.2	6.6
TN 6	3.9	4.0	5.4	4.4	6	5	84	89	86	86	17	25	--	--	6.5	6.6	6.6
TN 7	12.6	12.4	11.2	12.1	6	4	73	74	70	72	--	--	12	30	6.1	6.3	6.2
TN 8	9.7	9.7	11.3	10.2	5	5	83	84	85	84	12	17	15	30	5.8	5.8	5.8
TN 9	6.8	6.8	7.6	7.1	5	2	80	83	79	81	15	--	18	26	5.6	6.1	5.8
TN 10	8.6	8.7	9.7	9.0	6	3	49	43	34	42	16	22	24	40	6.1	6.2	6.2
TN 11	10.3	12.0	11.6	11.3	6	3	77	87	86	83	14	19	15	22	6.8	6.6	6.7
TN 12	1.7	2.1	1.6	1.8	6	5	80	78	86	81	--	--	13	13	6.8	6.9	6.8
TN 13	0.8	0.8	1.3	1.0	6	2	87	83	70	80	20	21	--	--	6.5	6.3	6.4
TN 14	5.8	6.0	5.3	5.7	6	5	46	49	33	43	15	21	16	20	4.2	6.3	5.2
TN 15	3.7	3.6	3.6	3.6	6	5	93	93	92	93	17	23	19	27	6.5	6.3	6.4
TN 16	3.4	3.3	3.7	3.5	6	5	89	91	88	89	18	25	25	28	7.7	6.6	7.2
TN 17	1.8	2.0	1.4	1.7	6	6	97	97	97	97	25	35	30	44	6.2	6.6	6.4
TN 18	0.6	0.6	0.4	0.5	6	6	99	99	99	99	30	38	40	45	5.6	6.1	5.8
									Ohio								
OH 1	4.4	4.5	5.0	4.6	6	4	73	75	62	70	14	26	18	29	7.8	8.6	8.2
OH 2	7.1	7.1	8.0	7.4	6	4	3	2	24	10	12	17	18	28	8.6	8.7	8.7
OH 3	8.3	6.8	6.7	7.3	-	3	39	37	32	36	15	21	16	28	--	8.7	--
OH 4	3.4	3.3	3.6	3.4	4	3	52	59	69	60	--	--	--	--	6.5	--	--
OH 5	8.0	8.3	8.3	8.2	5	2	15	16	21	17	14	18	23	38	8.7	--	--
OH 6	9.2	8.4	8.7	8.8	-	3	50	71	63	61	13	18	17	25	--	8.2	--
OH 7	1.5	1.0	2.3	1.6	6	6	99	99	99	99	24	31	28	50	6.3	--	--
									Indiana								
IN 1	1.5	1.1	2.2	1.6	6	6	96	98	95	96	--	--	--	--	8.5	8.1	8.3
IN 2	8.6	8.5	7.5	8.2	5	4	25	34	47	35	13	14	16	30	8.7	8.5	8.6
IN 3	2.1	2.1	2.0	2.1	5	6	87	87	93	89	--	--	27	34	6.6	6.6	6.6
IN 4	3.7	3.7	3.3	3.6	6	3	88	88	88	88	15	25	14	38	7.0	7.3	7.2
IN 5	7.2	8.9	11.8	9.3	6	4	46	36	31	38	13	17	14	25	--	--	8.2†
IN 6	4.7	5.0	4.8	4.8	6	5	76	82	77	78	20	24	16	27	6.7	7.2	7.0
IN 7	4.9	4.8	4.9	4.9	6	3	79	74	74	76	19	22	23	30	6.0	6.1	6.0
IN 8	4.8	4.9	4.8	4.8	4	3	81	79	76	79	18	20	20	31	6.8	6.3	6.6
IN 9	5.9	6.2	12.6	8.2	6	2	24	18	6	16	17	20	16	20	8.9	8.3	8.6
IN 10	5.3	5.8	5.6	5.6	6	2	20	17	21	19	15	23	15	33	6.3	6.7	6.5
									Virginia								
VA 1	1.0	0.8	1.5	1.1	6	6	98	98	98	98	33	40	39	51	4.5	4.6	4.6
VA 2	0.6	0.4	0.9	0.6	6	6	99	99	99	99	25	40	37	40	6.0	6.2	6.1
VA 3	1.3	1.3	1.5	1.4	6	5	97	98	97	97	--	35	44	--	5.1	5.3	5.2
VA 4	0.2	0.2	0.6	0.3	6	6	98	99	97	98	--	--	--	--	8.2	8.4	8.3

* pH of distilled water used in test is 6.4.
** Material 1 more weathered than accompanying 2 and 3.
† Cycle not identified on one pH test.

Table 27. Shale mineralogy and semiquantitative* composition.

Shale	Clay Minerals				Nonclay Minerals							
Number	Clay-Mica	Chlorite	Kaolinite	Montmorillonite	Quartz	K-Feldspar	Plagioclase	Calcite	Dolomite	Siderite	Pyrite	Hematite
						Kentucky						
KY 1	C	Tr	R	--	C	--	R	--	M	--	--	--
KY 2	C	--	R	--	C	--	R	--	M	--	--	--
KY 3	C	Tr	R	M†	C	--	R	--	M	--	--	--
KY 4						NOT EXAMINED						
KY 5	C	R	Tr**	--	A	--	M	--	--	Tr	--	--
KY 6	C	R	Tr	--	A	--	C	C	--	Tr	--	--
KY 7	C	--	C	--	A	--	M	--	--	M	--	--
KY 8	C	M	M	--	C	--	M	--	--	R	--	--
KY 9	M	--	M	R†	A	--	R	--	--	R	--	--
KY 10	C	M	M**	--	A	--	M	--	--	R	--	--
KY 11	C	M	M	--	A	--	M	--	--	R	R	--
KY 12	C	M	M**	--	A	R	R	--	--	R	--	--
KY 13	C	M	M**	--	C	--	M	--	--	M	--	--
KY 14	C	R	--	--	C	R	--	--	C	--	R	--
KY 15	C	R	--**	--	C	--	R	--	M	R	--	--
KY 16	C	R	--**	--	C	--	R	--	R	--	--	--
KY 17	C	M	--	--	C	--	R	--	C	R	--	--
KY 18	C	Tr	M	--	A	--	M	M	--	R	M	--
KY 19	C	R	--	--	A	R	M	--	--	--	--	--
KY 20	C	M	M**	R	A	--	M	--	--	R	--	--
KY 21	C	M	--**	--	A	R	R	R	--	--	--	--
KY 22	C	--	M	R†	A	M	R	M	--	R	--	--
KY 23	C	--	M**	--	A	M	M	--	--	R	M	--
KY 24	C	M	M	--	A	--	M	--	M	--	--	--
KY 25	C	M	--**	--	C	--	M	M	M	--	R	--
KY 26	C	M	--	--	C	--	R	C	--	--	R	--
KY 27	C	M	--**	--	C	--	M	C	M	--	--	--
KY 28	C	M	--**	--	C	--	M	C	R	R	R	--
KY 29	C	M	--	--	C	--	M	M	R	R	--	--
KY 30	C	M	--**	--	C	--	M	M	--	--	R	--
						Tennessee						
TN 1	C	--	--	--	--	A	--	--	--	--	--	--
TN 2	C	R	--**	--	M	C	--	--	--	--	--	--
TN 3	C	R	--**	--	C	M	--	--	--	--	--	--
TN 4	C	R	--**	--	C	M	--	M	--	--	--	--
TN 5	C	R	--	--	C	M	--	--	--	--	--	--
TN 6	C	R	--**	--	C	M	M	--	--	--	--	--
TN 7	C	R	--**	--	C	M	--	--	--	--	--	--
TN 8	C	--	--	--	C	R	--	Tr††	--	--	--	Tr††
TN 9	C	--	--	--	C	R	R	Tr††	--	--	--	--
TN 10	C	R	--**	--	C	R	--	--	--	--	--	--
TN 11	C	R	--	--	C	R	--	--	--	--	--	--
TN 12	C	R	--**	--	C	C	--	--	--	--	--	--
TN 13						NOT EXAMINED						
TN 14	C	M	M	--	A	--	M	--	--	R	--	--
TN 15	C	M	M**	--	A	--	M	--	--	M	--	--
TN 16	C	M	M**	--	C	--	C	--	--	M	--	--
TN 17	C	M	M	--	A	--	M	--	--	M	--	--
TN 18	C	--	M	--	A	--	C	--	--	R	M	--
						Ohio						
OH 1	C	M	--**	--	C	--	M	M	M	--	--	--
OH 2	C	M	--**	--	C	--	R	R	M	--	R	--
OH 3	C	M	--**	--	C	--	M	C	M	--	R	--
OH 4	C	M	--	--	M	--	R	--	A	--	R	--
OH 5	C	M	--	--	C	R	--	--	M	--	--	M
OH 6	C	M	M**	--	A	--	M	--	--	--	--	--
OH 7	C	Tr	M	--	A	--	M	--	--	R	M	--
						Indiana						
IN 1	M	Tr	--	--	M	R	--	A	M	--	--	--
IN 2	C	M	--**	--	C	--	R	M	M	--	R	--
IN 3	C	R	M	--	A	--	C	--	--	M	--	--
IN 4	C	R	M**	--	A	--	C	--	--	M	--	--
IN 5	C	M	--**	--	C	R	M	M	--	R	R	--
IN 6	C	M	--	--	A	--	M	--	--	R	R	--
IN 7	C	R	--**	--	A	--	M	--	--	M	--	--
IN 8	C	R	M**	--	C	--	M	--	--	M	--	--
IN 9	C	M	--**	--	C	--	M	R	M	--	--	--
IN 10	M	R	C	--	A	M	--	--	R	R	Tr	--
						Virginia						
VA 1	C	M	--**	--	A	--	M	--	--	R	R	--
VA 2	C	M	--	--	A	--	M	R	R	R	R	--
VA 3	C	R	--**	--	A	--	M	R	R	R	R	--
VA 4	M	M	--	--	M	--	R	A	--	--	--	--

* Relative amounts determined by X-ray diffraction peak intensities: (A) abundant, greater than 50 percent; (C) common, about 25-50 percent, (M) minor, about 10-25 percent, (R) rare, about 5-10 percent, (Tr) trace, less than 5 percent, (--) not present.
** Composition projected from associated samples; no acid treatment for distinguishing kaolinite from chlorite.
† Mixed layer clay (montmorillonite and clay-mica).
†† Tentative mineral identification.

The samples were also carefully examined by semiquantitative XRD analysis to provide a semiquantitative mineralogical composition for each.

Sample description. Portions of about half of the samples were sealed in jars in the field and subsequently tested for water content in the laboratory. Water that normally sweats out after removal from the ground was retained in the jars, so these water contents are representative of field conditions. A complete set of water contents was determined later during the process of slake-durability indexing (Table 26). These water contents do not appear to be substantially different from field water contents despite the sample sweating that had occurred in plastic sample bags between the time of sampling in the field and the laboratory determination.

Each sample was examined visually in the laboratory and described as follows in Table 25:

a. Color was determined on a representative hand specimen according to the Munsell color system, and the nomenclature used in Table 25 is according to that system.

b. The term "structure" refers to either bedding or fissility. The particular terms chosen for this description are intended to indicate the degree of intact fabric structuring.

c. Under "fragment shape," the following self-explanatory terms were used for description: "thinly platy," "platy," "tabular," "prismatic," "blocky," and "irregular."

d. Texture was described as "clayey," "silty," or "sandy" with emphasis on departures from a "shaly" texture. More than half of the samples were shale or mudstone; the remaining samples were silty or sandy, though still basically shale. When discrete thin silty or sandy laminae were evenly dispersed, the material was considered simply as silty or sandy shale rather than a mixture of siltstone, sandstone, and shale laminae. Seven of the rocks were siltstone rather than shale and are identified as such on the table. One sample was an argillaceous limestone.

e. The formation is also identified in the table. In a few cases, this identification was tentative at best. The nonproblem shales were from the New Albany, Palestine, Whitewater, Ohio, Elkhorn, and Brallier formations. According to information from highway departments, the remaining formations contain known problem shales: Osgood, New Providence, Tradewater, Crab Orchard, Preacherville, Nada, Lee, Crider clay, Newman, Kope, Dillsboro, Fairview, Millboro, Chattanooga, Nolichucky, Rome, Edinburg, and Pennsylvanian formations of Eastern Tennessee (including Redoak Mountain, Graves Gap, and Cross Mountain formations).

Indexing and associated tests. The slake-durability test and two supplemental index tests chosen for their simplicity were selected to obtain indexes representing the pertinent physical behavior of shale. Since dry-wet cycling is a conspicuous deleterious environmental factor (Part VI) in behavior of soft rocks, the slake-durability test appeared to be well suited to the study. A simple jar slake test was conducted for comparison with slake-durability results. A second simple test, sclerscope hardness was used to provide an index (discussed in Part VI and Table 24) which might relate to strength and elasticity. Additional supplemental determinations of water content before slake-durability testing and of the acidity of the test water after testing were made.

The basic portion of the testing program was the slake-durability test described by Franklin and Chandra (1972). This test measures the resistance of rock to weakening and disintegration as a result of two cycles of drying and agitation in a water bath. The test is accepted as a standard by the International Society of Rock Mechanics. The apparatus for conducting the test consists essentially of:

a. A solid-ended drum cage of 2-mm standard mesh, 100 mm long, and 140 mm in diameter.

b. A trough to be partially filled with water and to contain the drum cage in a horizontal position.

c. A motor drive capable of rotating the drum 20 rpm.

In the standard procedure, a representative group of 10 rounded lumps of the rock, each weighing 40 to 60 g and together totaling 450 to 550 g, is placed in the drum and dried to a constant weight at 105°C. In the present study in which the same pair of drums were used for running an overlapping sequence of tests, all drying and weighing were done in trays outside of the drum. The sample was then placed in the drum and the drum mounted in the partially filled trough and coupled to the motor. The trough was filled to 20 mm below the drum axis with distilled water and the drum rotated at 20 rpm for 10 min.

The drum was then removed from the trough and unloaded. The sample was oven-dried and weighed. The same procedure was repeated in the second cycle and the slake-durability index, I_D , was calculated using Equation 8 (Part VI). Water content of the material was obtained from the moisture loss determined during the first drying for slake-durability testing. The material remaining after testing (Figures 37-48) gives some suggestion of the type of breakdown that may take place in embankments.

Jar samples were taken from the water bath shortly after the end of agitation for both cycle one and cycle two of slake-durability testing and sent to the chemical laboratory for determination of the pH of the test water. The water samples were obtained 1 to 10 min after completion of test agitation. The delay in sampling allowed coarse and gritty materials to settle out completely, but the water was still cloudy from suspended fine material. After a day of settling, this suspended material formed a thin layer of clay at the bottom of the jar.

A hardness index (termed "scleroscope index" in Table 26) was determined using a Shore scleroscope, which is designed to measure an index of hardness of metals somewhat similar to and correlatable with Rockwell hardness. The Shore scleroscope (Model D) is a nondestructive, hardness measuring device which indicates relative values of hardness by the height of rebound of a small diamond-pointed piston dropped vertically within a tube onto the test surface from a set distance of approximately 1 in. The height of rebound is indicated on a scale of 0 to 120 divisions. The sample is held fixed during testing by the tube being lowered until it bears against the sample resting in turn on the flat metal base.

For the scleroscope index tests on the shale samples, a specimen weighing approximately 50 g was used. The most reliable readings were obtained from pieces cut with parallel surfaces. Each impact damaged the surface so that a new test area was required. The set of data shown in Table 26 is applicable for tests parallel and perpendicular to bedding or fissility and at moist and ovendry conditions. Ideally, a number of impacts should be averaged for best scleroscope indexes;

Figure 37. Material remaining after slake-durability testing, KY 1-KY 6.

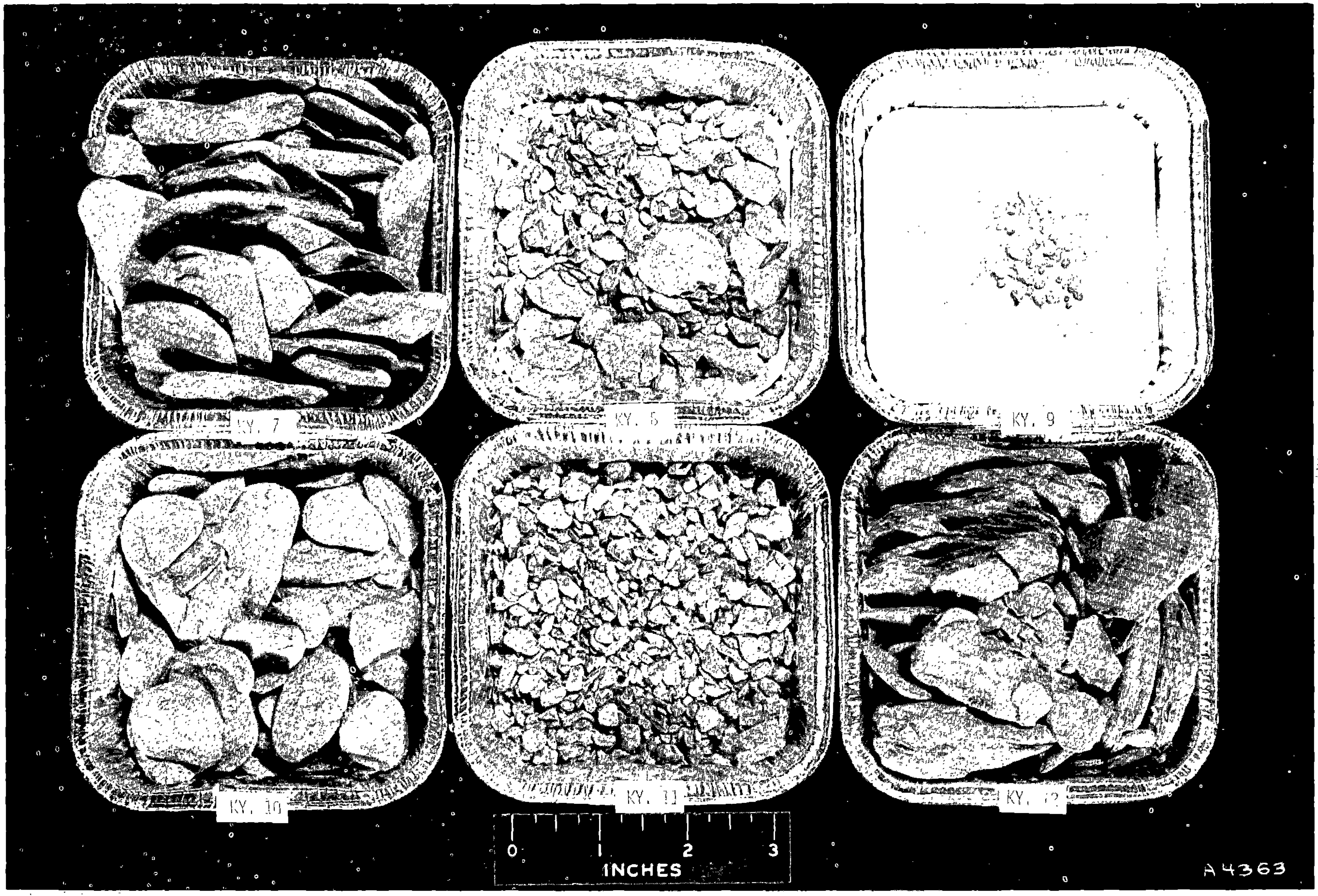

Figure 38. Material remaining after slake-durability testing, KY 7-KY 12.

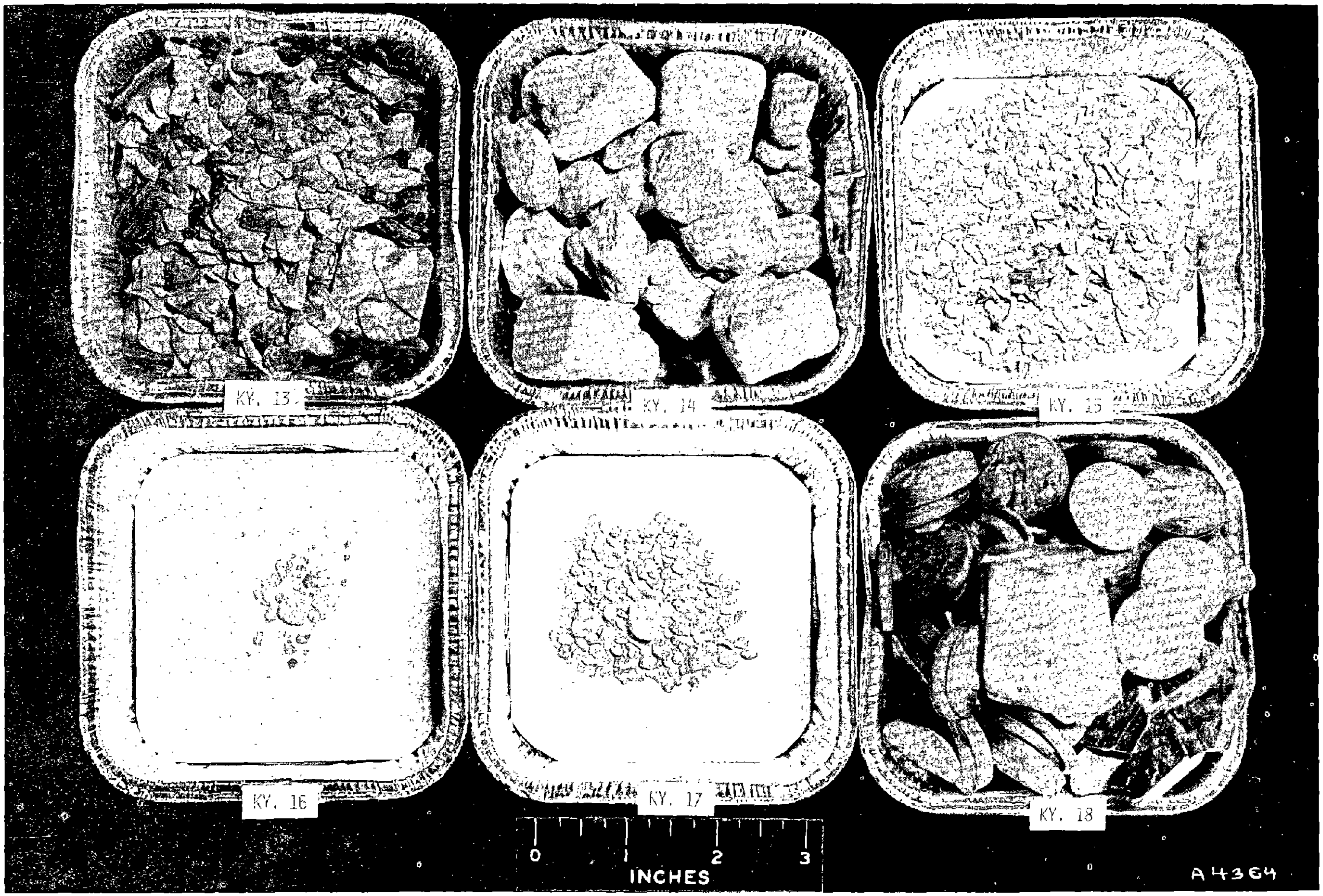

Figure 39. Material remaining after slake-durability testing, KY 13-KY 18.

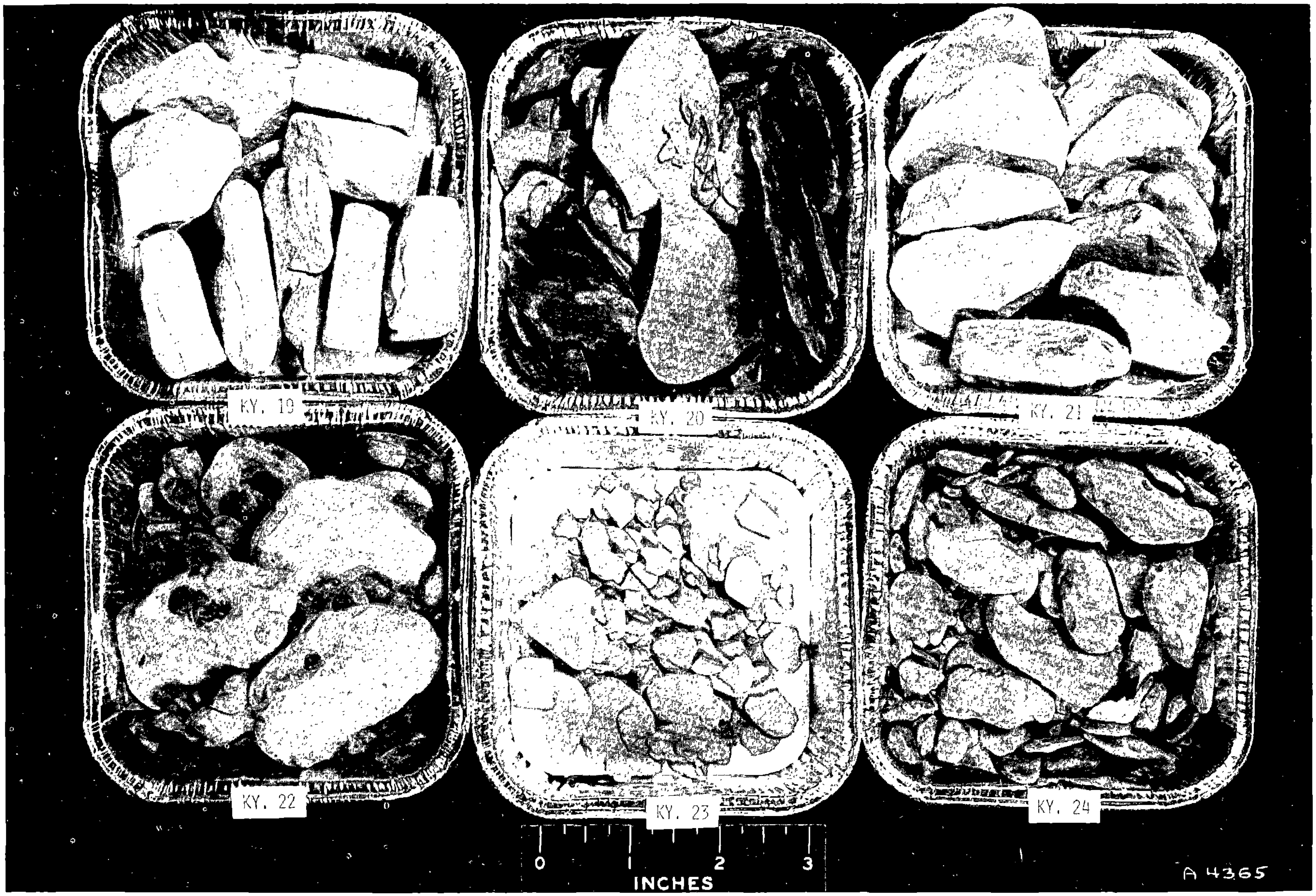

Figure 40. Material remaining after slake-durability testing, KY 19-KY 24.

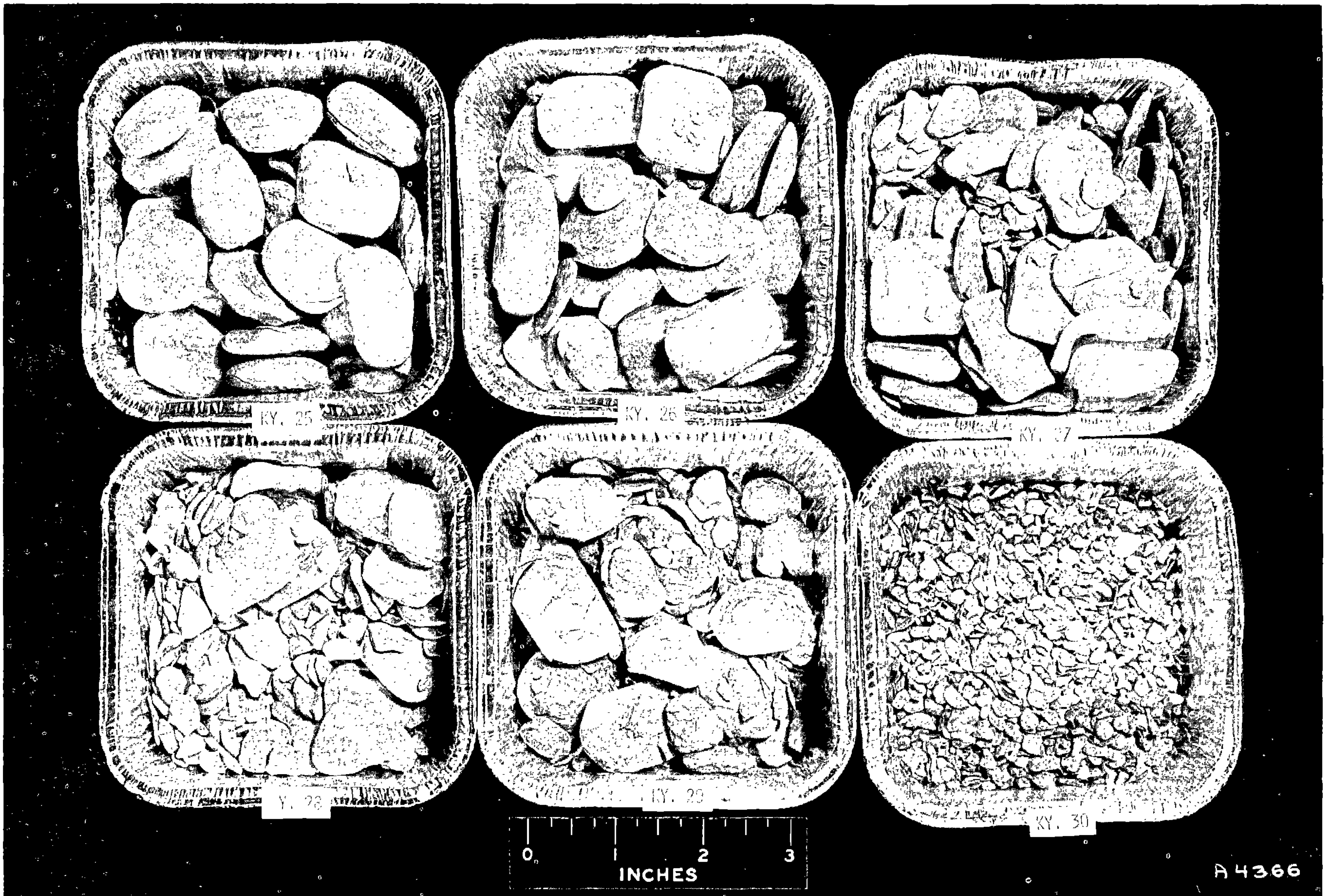

Figure 41. Material remaining after slake-durability testing, KY 25-KY 30.

Figure 42. Material remaining after slake-durability testing, TN 1-TN 6.

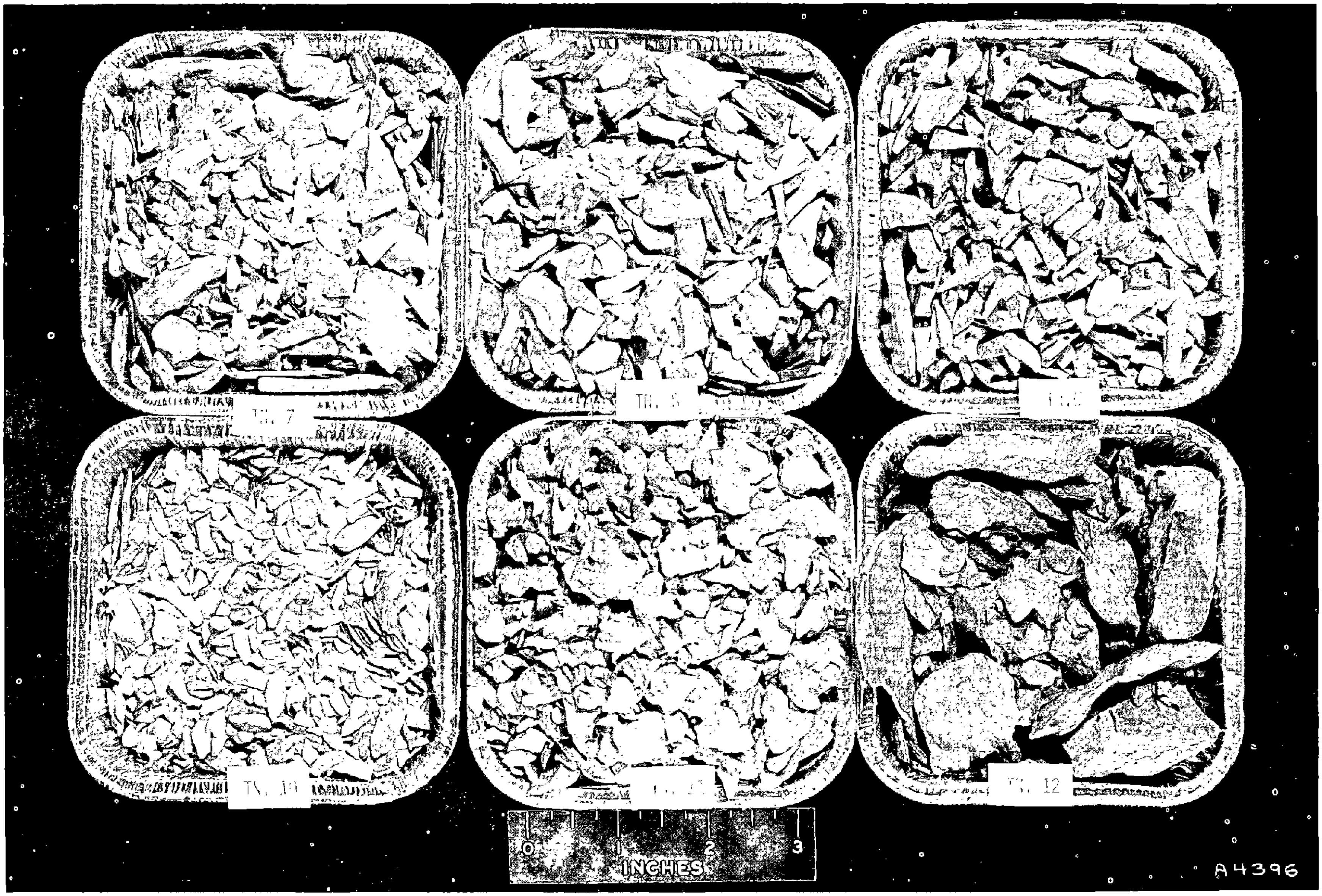

Figure 43. Material remaining after slake-durability testing, TN 7-TN 12.

Figure 44. Material remaining after slake-durability testing, TN 13-TN 18.

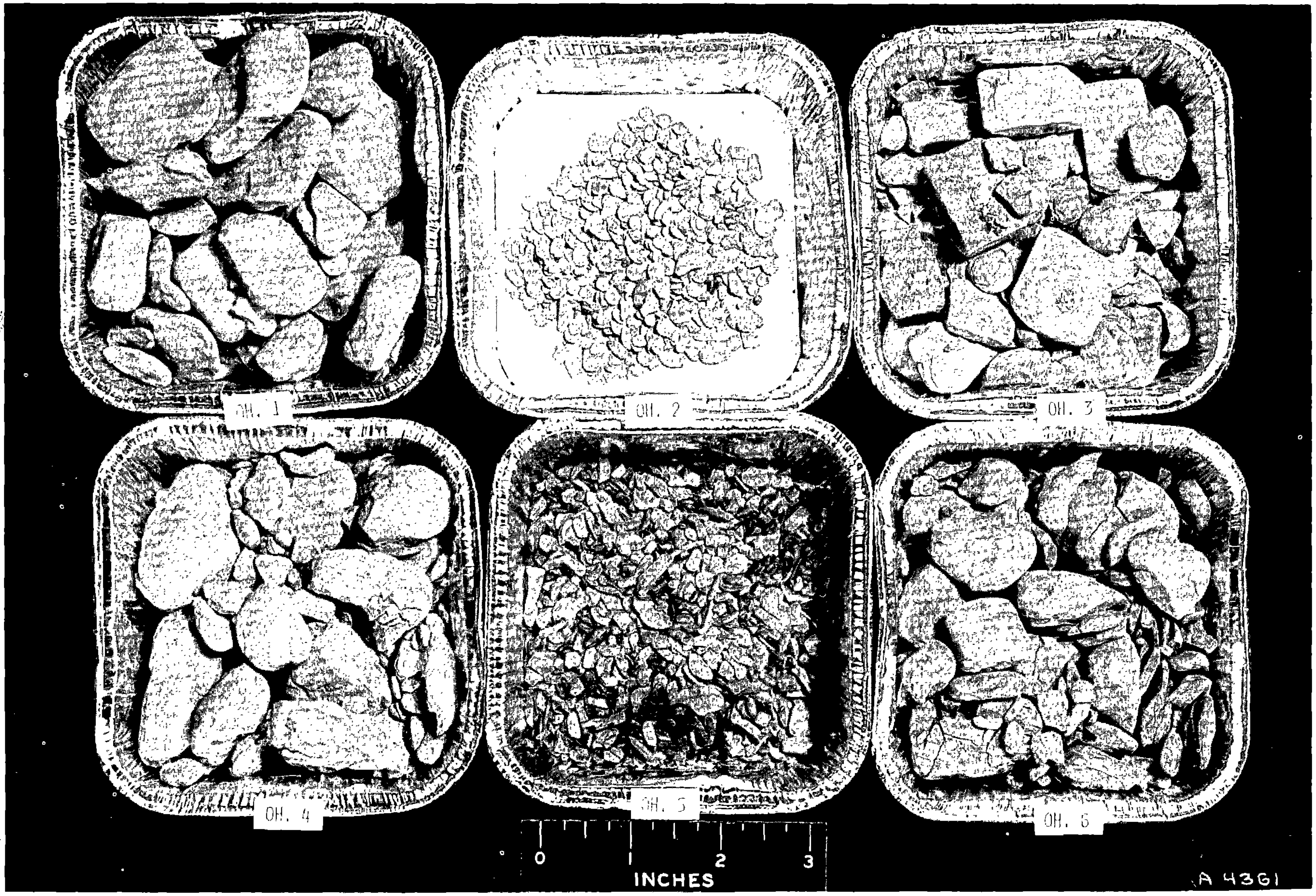

Figure 45. Material remaining after slake-durability testing, OH 1-OH 6.

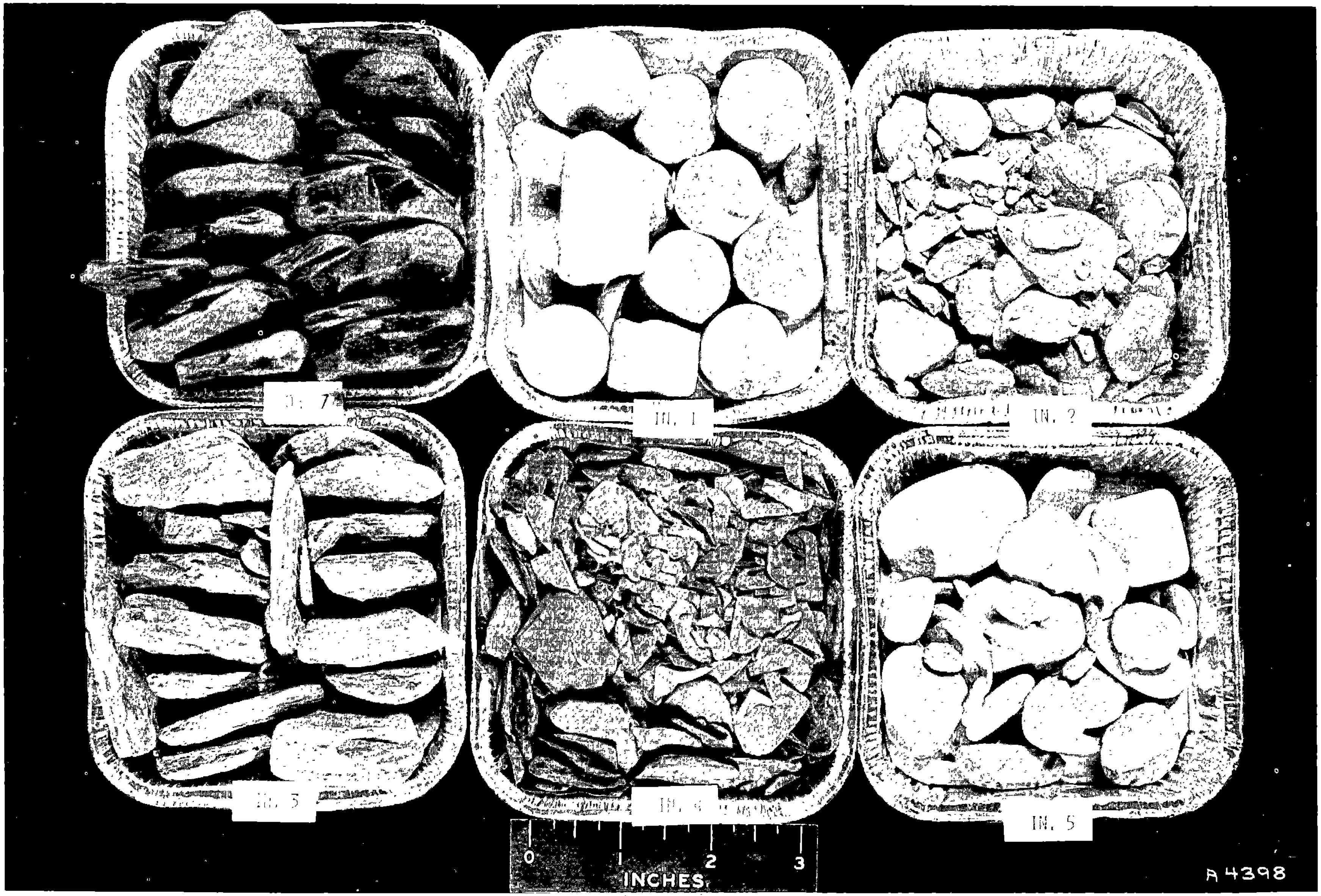

Figure 46. Material remaining after slake-durability testing, OH 7, IN 1-IN 5.

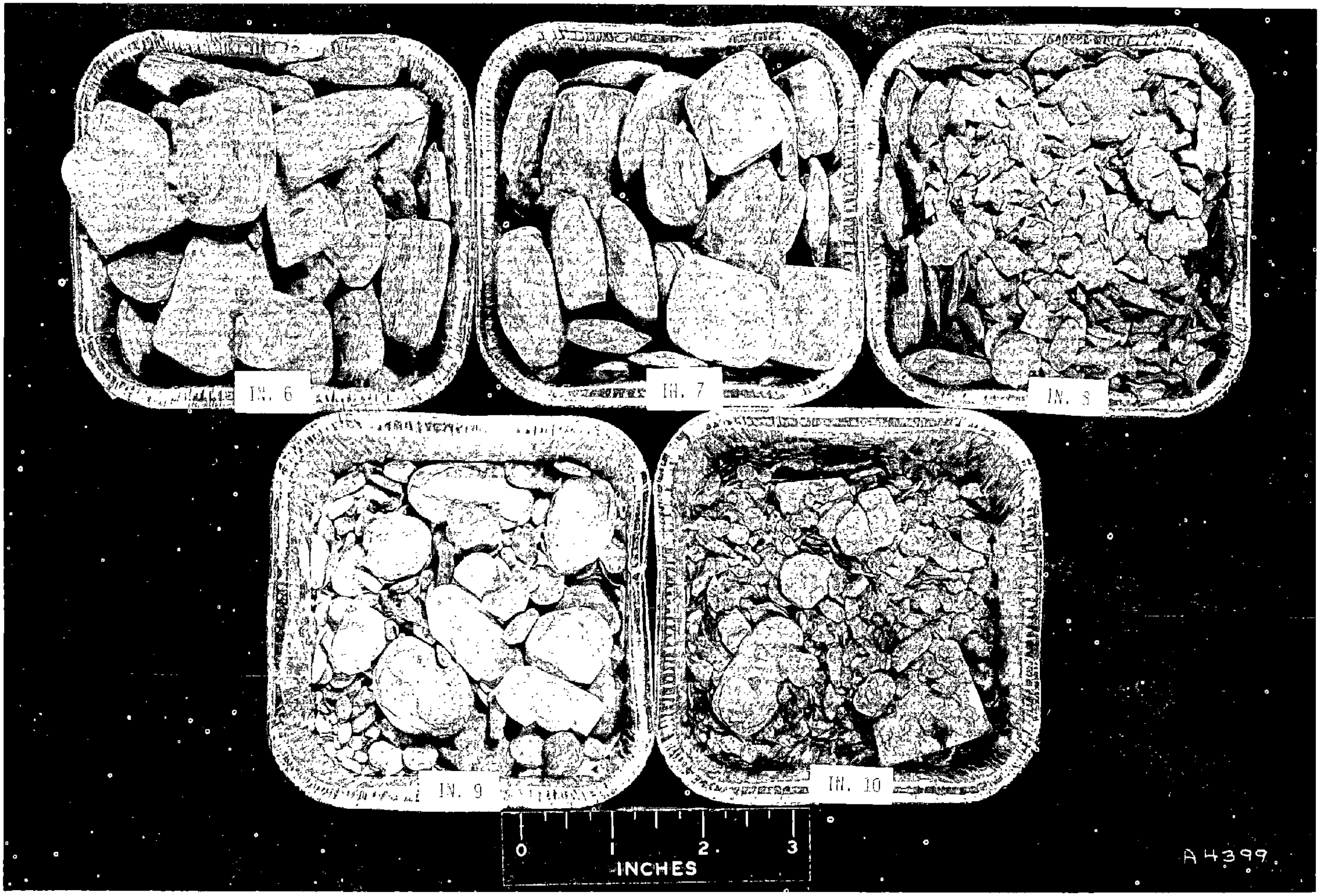

Figure 47. Material remaining after slake-durability testing, IN 6-IN 10.

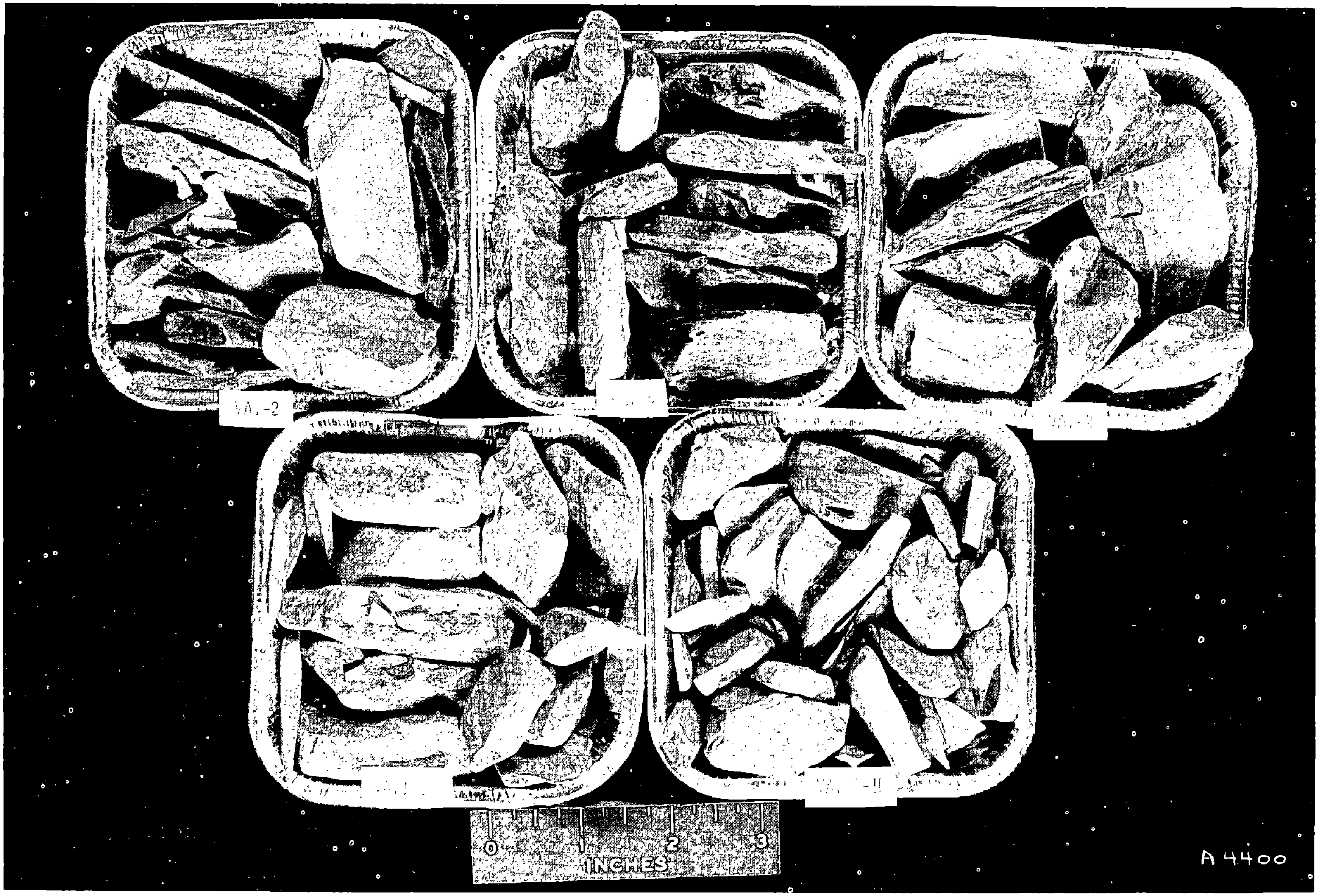

Figure 48. Material remaining after slake-durability testing, VA 1-VA 4.

however, in these tests, condition and configuration of the test blocks were often poor, and the values listed in the table are based on what was judged to be a reasonable degree of reproducibility of test results.

A fifth test conducted on the shale samples was a jar slake test for direct comparison with slake-durability results. For this study, the test consisted of placing an irregular fragment of the material weighing approximately 20 g in a dish of distilled water and describing the resulting behavior in the following categories.

Category	Behavior
1	Degrades to a pile of flakes or mud
2	Breaks rapidly and/or forms many chips
3	Breaks slowly and/or forms few chips
4	Breaks rapidly and/or develops several fractures
5	Breaks slowly and/or develops few fractures
6	No change

The test was conducted on both damp and ovendried material, but the damp material was generally insensitive as indicated by the results shown in Table 26. Reactions usually occurred the first 10 min. Therefore, careful observations were made during the first half hour. It seemed that further breakdown was very subordinate, and the only other observation necessary was of the final condition after 24 hr. The only jar slake behaviors observed were fracturing and flaking. A few samples seemed to have experienced a subtle softening, but no attempt was made to identify this behavior systematically.

Mineralogical examination. Representative portions of shale samples were subjected to analysis of their mineralogy. XRD was used to produce a semiquatitative evaluation of the mineral content. Each sample was crushed and quartered. Ten grams were ground to pass a No. 325 sieve. The powder was placed in a 3-in. aluminum holder, backpacked to minimize preferred orientation, and then X-rayed.

Another portion of the raw sample, amounting to 25 g, was put into 500 ml of distilled water and agitated in a blender for 5 min. The slurry was put into a 1000 ml beaker, an additional 500 ml of distilled water was added, and settling behavior was observed. If settling was visible, the top 1-1/2 to 1-3/4 in. of suspension was siphoned into an 800 ml beaker after 4 hr and the beaker was filled with distilled water. Two 3- by 1-in. glass slides were suspended in the beaker overnight and then withdrawn. The sedimented slides were air-dried and X-rayed.

If settling was not visible, 500 ml of slurry was discarded, and the remaining 500 ml of slurry was diluted again by adding 500 ml of distilled water. If the settling was still too slow, 2 to 20 drops of a dispersing agent were added. The sedimented slides were prepared as described above and X-rayed.

Samples with a 14 Å spacing were treated with glucerol, allowed to stand overnight, and then X-rayed again. Where the 14 Å spacing was caused by montmorillonite, absorption of the glycerol caused the spacing to increase to about 17 Å. One sample was examined for possible halloysite because of its especially moist condition; no halloysite was found.

The remaining examination of clay minerals, in particular that for kaolinite and chlorite, was accomplished selectively within groups of related samples. Representative samples were treated with hydrochloric acid diluted with distilled water in the ratio 1 part acid to 4 parts water. The slurry was allowed to react for 6 hr at 80°C. The residue was then washed, put on a 1- by 1-in. glass slide, air-dried, and then X-rayed. This procedure removed the chlorite which, when present in substantial quantity, masks the kaolinite in an X-ray pattern.

The minerals searched for, at least selectively, are listed in Table 27 in two groups, clay minerals and nonclay minerals. The nonclay minerals were determined from XRD of the bulk sample. The clay minerals were determined from the sedimented slides, with or without treatment. Clay-mica describes a group of minerals with similar crystal structures; the predominant mineral of this group in shale is usually illite.

The distinction of kaolinite and chlorite is based on the following procedure. Chlorite can be positively identified by the presence of its 4.7 Å spacing. A prominent and diagnostic spacing for kaolinite at about 7 Å is shared with chlorite. Treatment with acid breaks down the chlorite structure and leaves only the diagnostic kaolinite peak.

The only other way kaolinite was identifiable was in the absence of chlorite in the original sample, in which case there is no superimposition of two mineral peaks at 7 Å. About half of the samples were not acid-treated, and the identification of kaolinite was by association and therefore not confirmed. The samples were studied in related groups and apparently representative samples were selected from each group for acid treatment. The abundance of kaolinite determined in these representative samples was then projected to the other samples in the group. These projected compositions of kaolinite are indicated by asterisks in the table.

Results of Tests and Examinations

The results of the laboratory work conducted on the total collection of problem and nonproblem shales are tabulated in Tables 25-27. In this section, characteristics of shale formations and ranges within a given characteristic are reviewed. Samples are tabulated by State in chronological order of collection so that no meaningful trends will be found in proceeding downward in each column. Similarly, the average values or characteristics given below are not intended to do more than generalize the entire heterogeneous collection.

Description of samples. Natural water content as sampled in the field (Table 25) varied between values of approximately 1 and 19 percent; 45 determinations averaged 8.46 percent (standard deviation $S = 3.90$) but these included 9 values of greater than 12 percent which quite possibly were affected by surface weathering. The remaining 36 determinations averaged 7.05 percent ($S = 2.69$). Color varied widely in the shales sampled although there was a preponderence of greenish-gray in some formations. The degree of bedding or fissility also varied from relatively structureless to highly laminated material. This internal structure was manifested to some degree in the shape of fragments that resulted during natural or test degradation, i.e., tabular and platy fragments reflecting internal anisotrophy.

The texture or fabric grain size of the rock was typically shaly but often with a silty component. The texture reflects to some degree an intentional selectivity exercised in the field sampling program of shales.

The formations that were sampled were all Paleozoic in age (Table 25). This fact should be kept in mind in drawing conclusions that may eventually be extended to younger shale formations.

<u>Physical test results</u>. Table 26 indicates the results of physical tests on shale and associated determinations of water content and test water pH. The water content was determined before each of the three slake-durability tests on each sample, as indicated in the table. An average value of water content is also given. The range of average water contents was from 0.3 to 20.3 percent, but again certain samples were abnormally high in water as though they had been softened by surface weathering processes. Omitting four values of greater than 12 percent, the average water content is reduced to 5.78 percent (S = 2.99). A more meaningful arrangement of water content averages is shown in the groupings of samples according to formation. Such groupings are discussed in the next section.

Index values for the three individual slake-durability tests are given in Table 26 along with average values. These indexes ranged between 0 and 99 percent. The mean of average index values for all 69 samples is 60.94 percent (S = 32.60). The differences in indexes among the three tests on essentially the same material indicate that the testing did not always realize the reproducibility to within 5 percent claimed by Franklin and Chandra (1972). Reproducibility was quite high among the more resistant rocks, but was expected since these rocks did not degrade appreciably. Among the worst samples in terms of reproducibility were IN 9, OH 2 and 6, and KY 1, 3, and 14 (Table 26).

The results of pH determination on slake-durability test water after each of two cycles are indicated in Table 26 as averages. The pH averages ranged from 3.0 to 9.6. The mean of all 69 average values is 7.16 (S = 1.26). Again, this wide dispersion of values reflects the wide variation of shale types sampled in the program. When the pH's are grouped according to particular formations, the dispersion is reduced considerably and is not very great within a particular formation. It is interesting to note, however, that problem shales can produce either acidic or alkaline water in the slake-durability test.

A considerable amount of judgment was necessary to arrive at representative scleroscope index values. Irregular surfaces, sample sizes, and cracking had major effects on the index and were difficult to resolve. It was possible to obtain an average based on several closely agreeing impacts in about half of the tests.

Mineralogical analysis results. The results of the mineralogical analysis are presented in Table 27. Gross characteristics of the collection as a whole include the abundance of quartz in approximately half the samples. In fact it is apparent, considering that the values are semiquantitative, that quartz often exceeded the total of clay minerals. Among clay minerals, clay-mica (presumably illite) almost always dominated. Chlorite was usually second in abundance followed by kaolinite. Only four samples contained significant montmorillonite, and in three of these cases the montmorillonite was in mixed-layer relationship with clay-mica.

Among the three carbonates identified (calcite, dolomite, and siderite), the first two are most characteristic of marine shale and in part, represent fragmental fossil debris. Siderite is the dominant carbonate in samples of Mississippian and Pennsylvanian formations, particularly in coal-bearing strata. Siderite may largely constitute a cementing material, and therefore it was considered subsequently in regard to possible strengthening and increased slake-durability. Pyrite was commonly found in marine shale.

Comparisons with Stratigraphy and Location of Problems

The extensive data presented in the tables are best considered in terms of variability when compared within the confines of individual formations. The sampling program produced sample groups of modest size from three relatively confined stratigraphic sequences: the upper Ordovician around Cincinnati, Ohio; the middle Cambrian near Knoxville, Tenn.; and the middle Pennsylvanian in western Kentucky. Numerous smaller groups were obtained from thinner sequences of problem shales. Figures 49-55 show the sampled strata and general lithology along with sample characteristics that seem to correlate with stratigraphic position and/or problem localization. Abbreviations used in these figures are listed below.

<u>a</u>. Under "structure": No = no structure, Bd = bedded, and Lm = laminated.

<u>b</u>. Under "texture": Cl = clayey, Si = silty, and Sa = sandy.

The abbreviations under the mineral content columns are given in a note on Table 27.

<u>Kope formation</u>. The upper Ordovician formations near Cincinnati consist of several hundred feet of marine limestone and shale in thinly alternating beds (Gray, 1972). The sequence occurs over a wide area and has caused embankment problems of considerable magnitude in the past (Part III). The base of the sequence of interest is a limestone known as the Point Pleasant or Cynthiana (Figure 49). The top of the sequence is marked by Brassfield limestone at the base of the Silurian strata. Stratigraphic names that have been or are being used for the formations within this sequence are indicated on the figure, along with the approximate positions of 15 samples used in this study.

The more interesting characteristics of the samples are shown graphically on the side for relative comparison in relation to stratigraphic position. The top three samples in the set are of lesser importance than those below because of their anomalous lithology and remote position with respect to problems. Samples OH 4 and 5 were taken from a unit known as the Elkhorn beds composed of alternating red and green shale and siltstone at the very top of the Ordovician strata in central Ohio. Sample IN 1 is a limestone which at its locality in Indiana is situated a few feet below the Silurian strata.

Within the more pertinent portion of the sequence occupying the lower two-thirds, the slake-durability index varies from low values to intermediate values of about 70 percent. Personnel of the Kentucky Department of Transportation offered the opinion that, if there is a

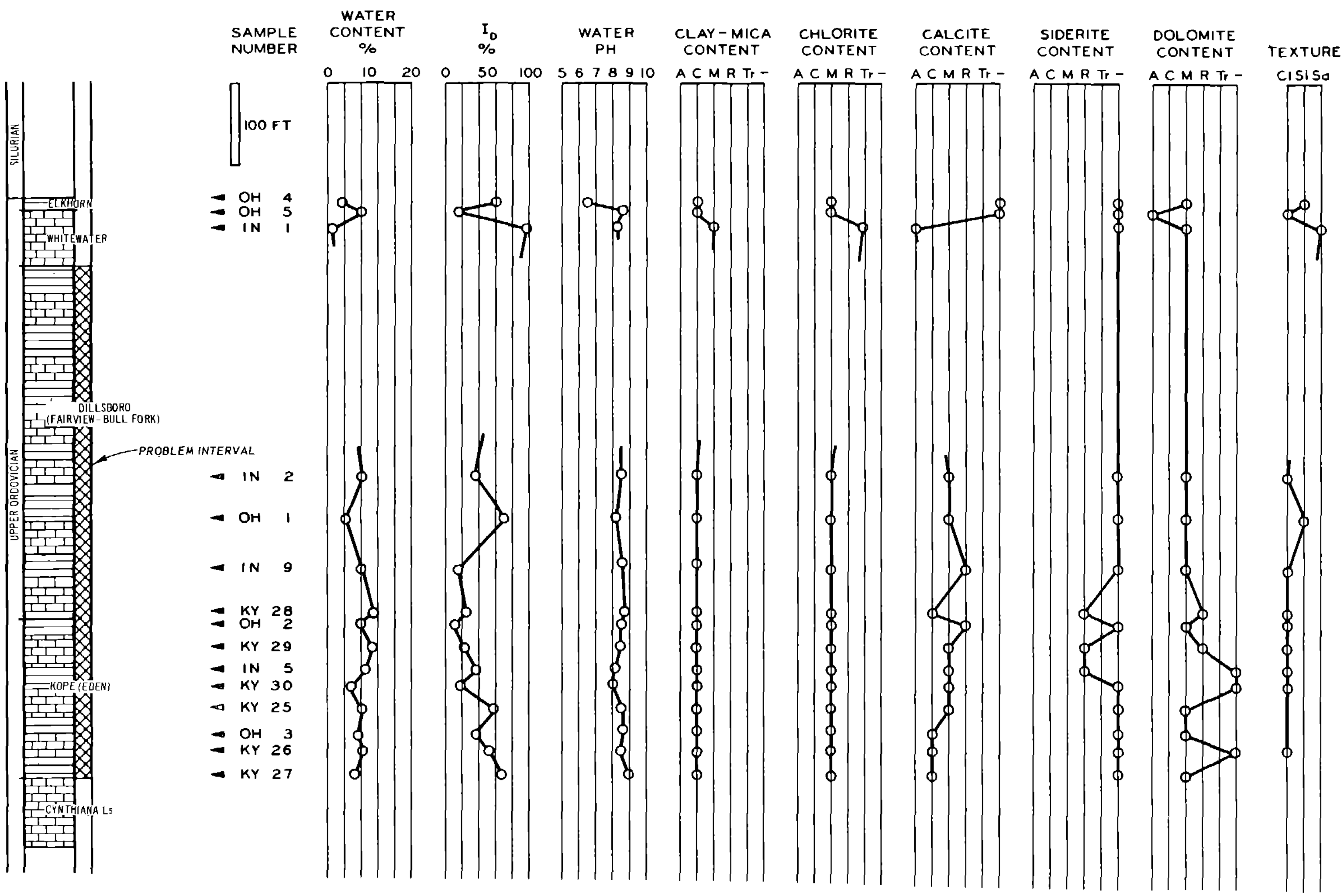

Figure 49. Test results and characteristics of the Dillsboro and Kope formations.

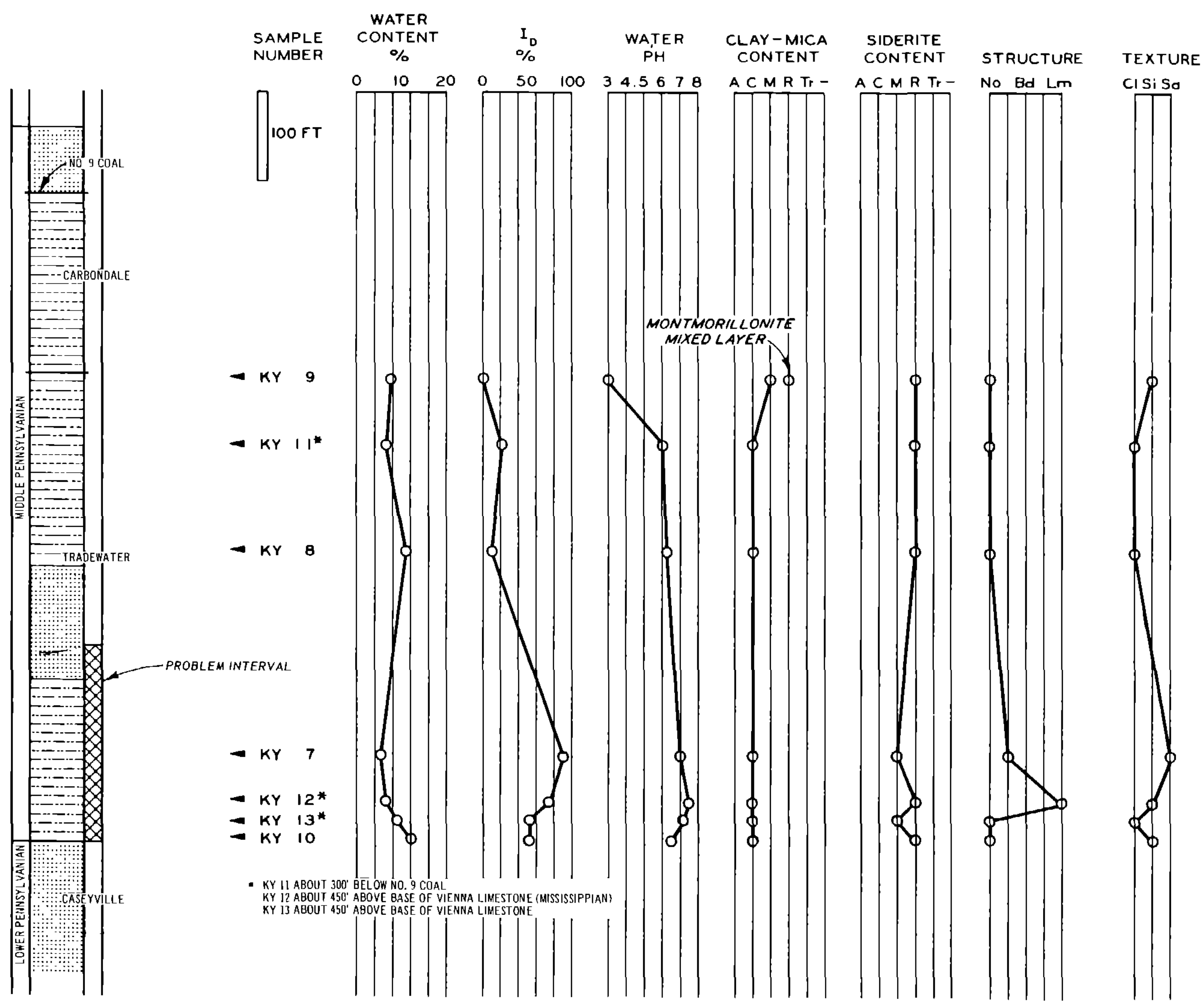

Figure 50. Test results and characteristics of the Tradewater formation.

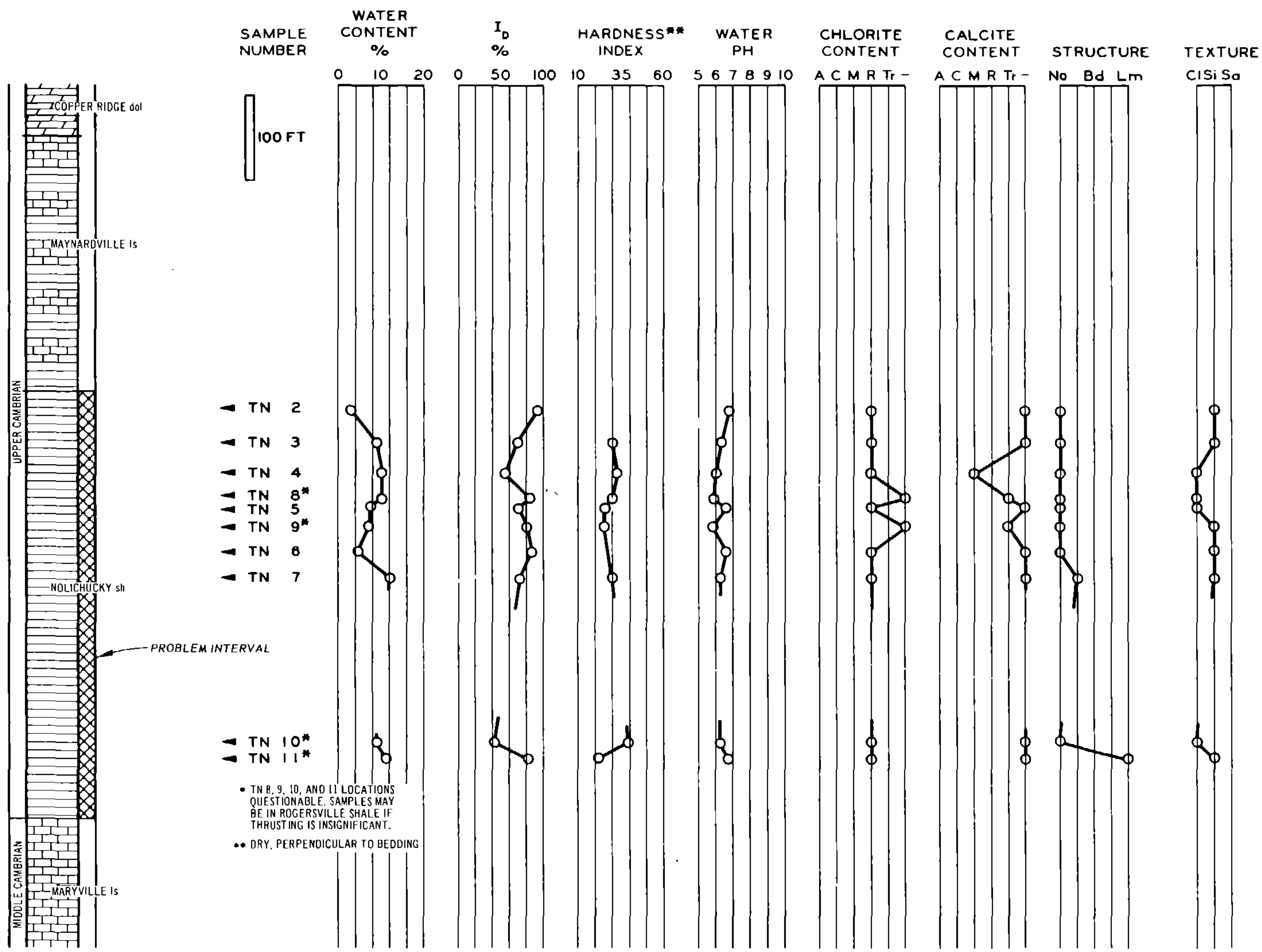

Figure 51. Test results and characteristics of the Nolichucky shale.

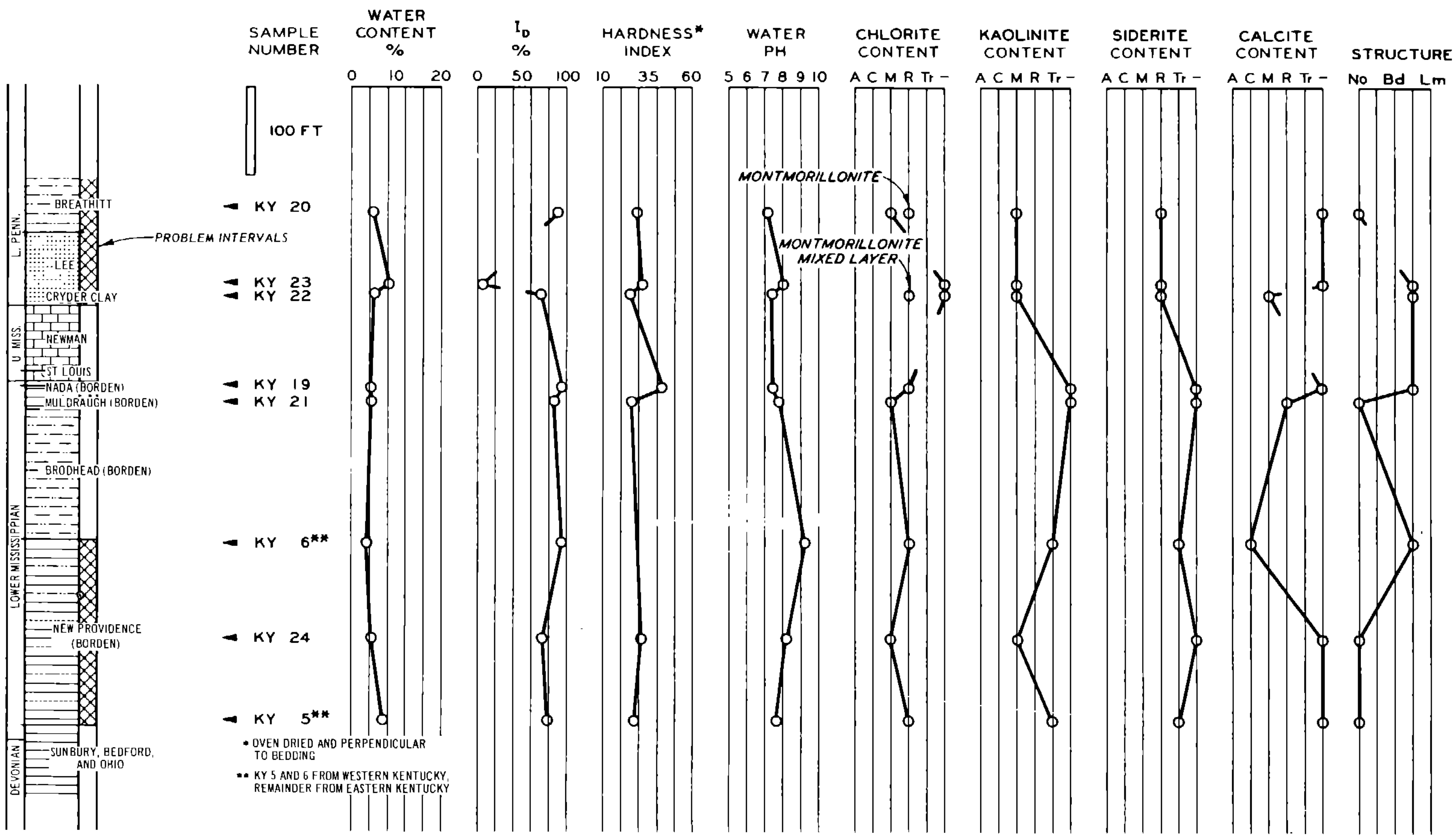

Figure 52. Test results and characteristics of the New Providence, Nada, Lee, and Breathitt formations.

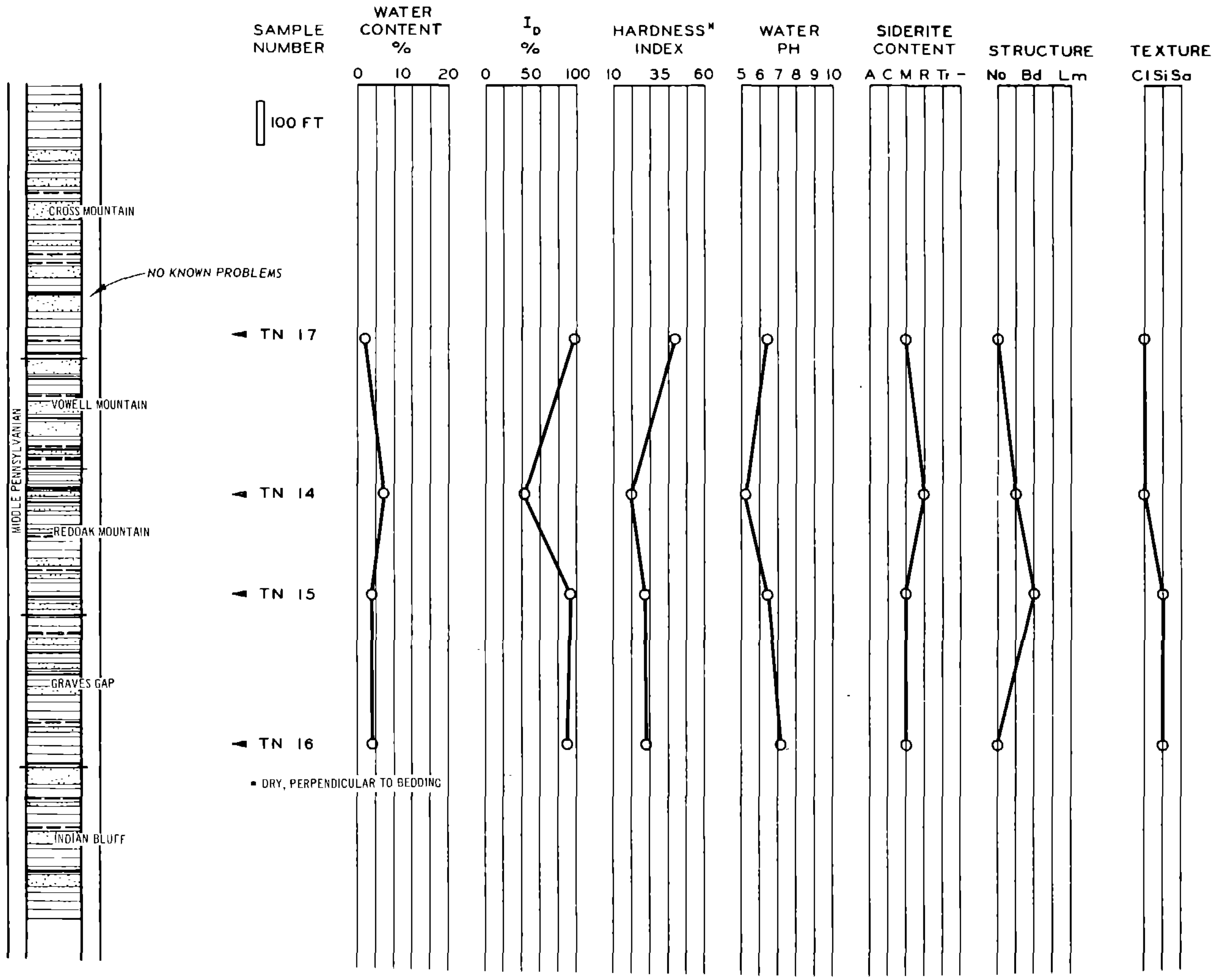

Figure 53. Test results and characteristics of Pennsylvanian formations of eastern Tennessee.

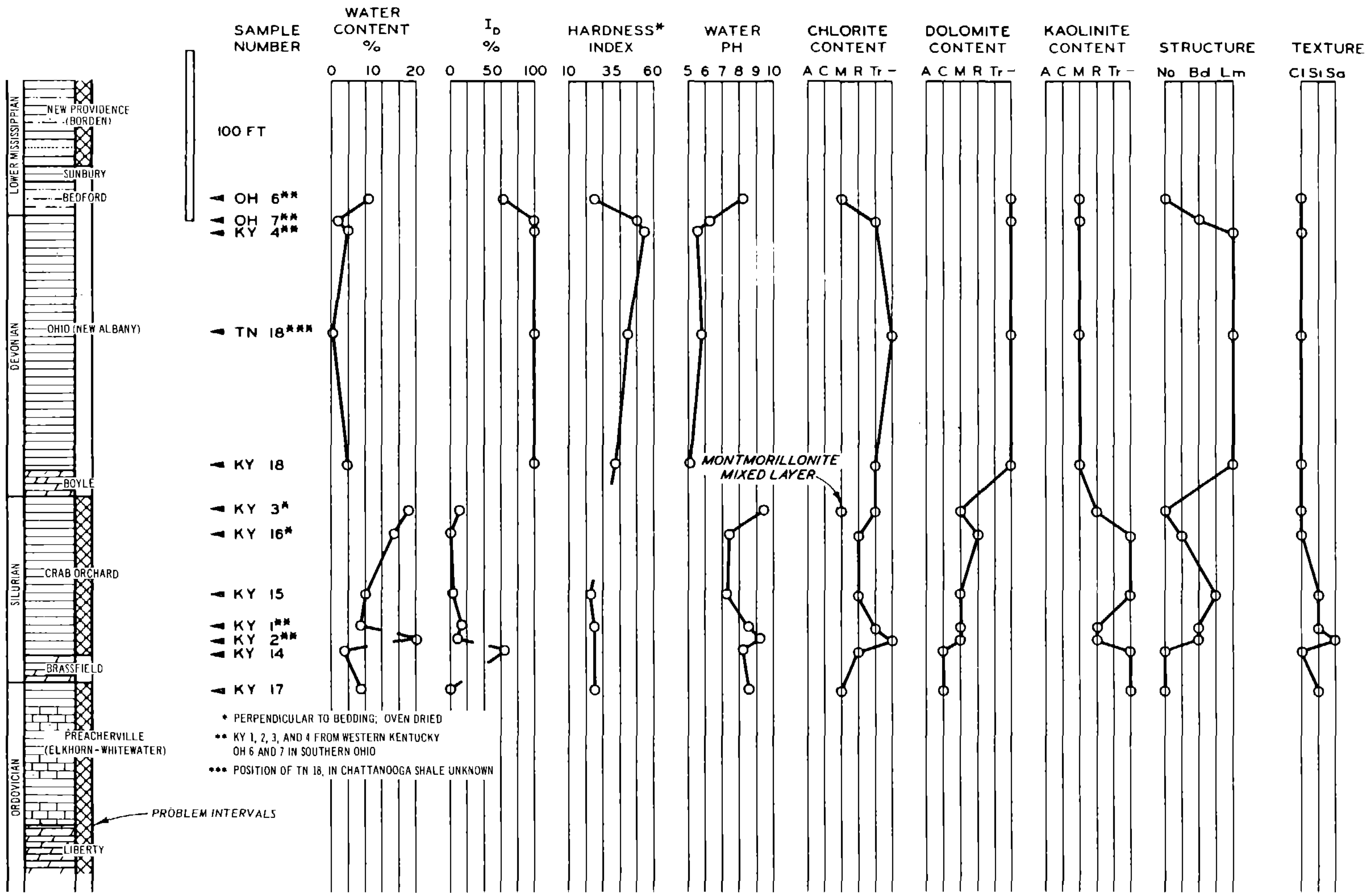

Figure 54. Test results and characteristics of the Ohio (New Albany) and Crab Orchard (Osgood).

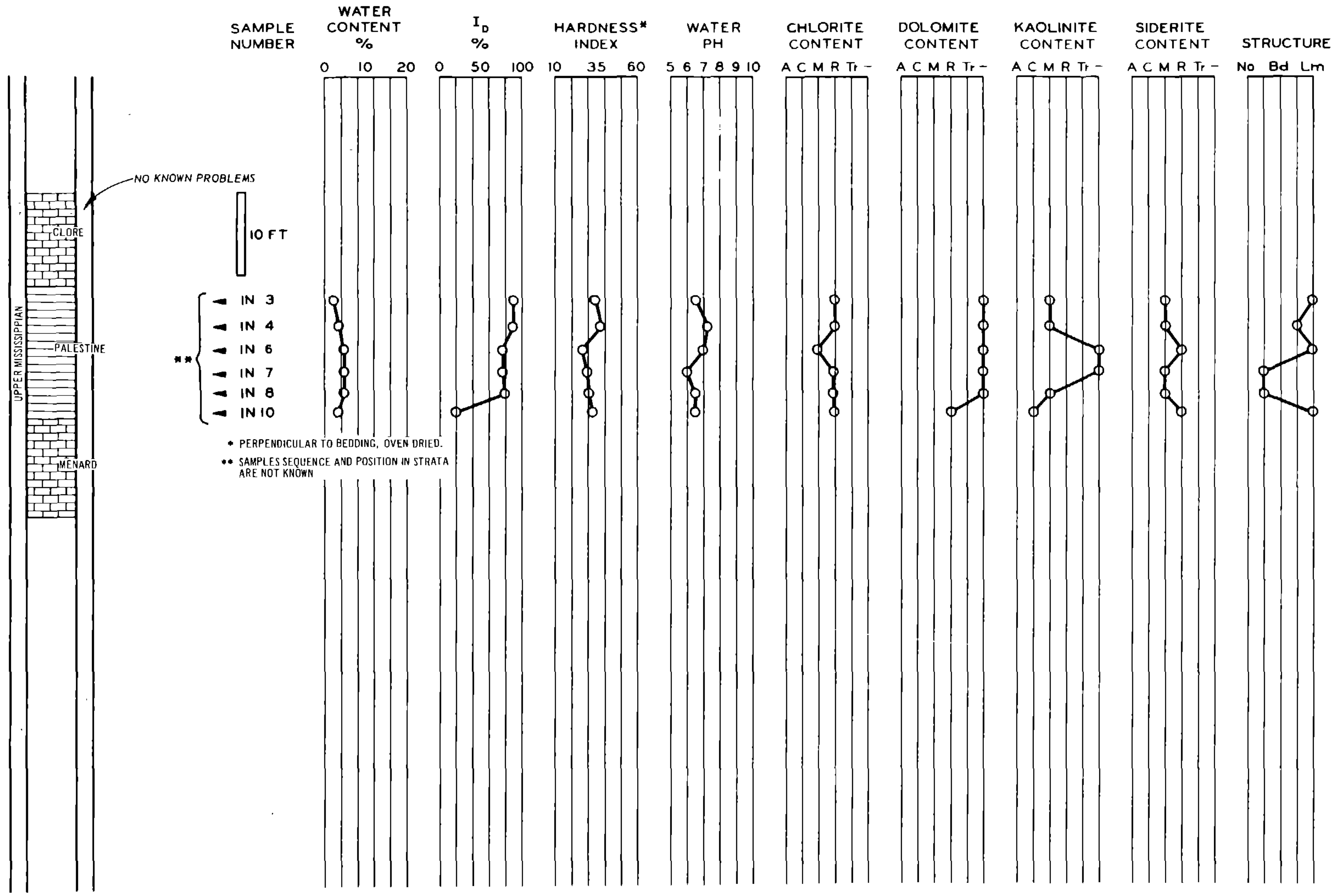

Figure 55. Test results and characteristics of the Palestine formation.

weaker portion of these strata, it probably is the upper Kope formation. The only Ordovician problem of considerable magnitude in Indiana involved shale embankments constructed of material taken near the Kope-Dillsboro contact. Low slake-durability test results appear to be most characteristically obtained for this problem interval.

An interesting correlative of the distribution of the slake-durability index is calcite content. Calcite is a minor mineralogical constituent for the most part in the low index zone but seems to increase in the lower 50 ft of a formation where slake-durability also shows a tendency to increase. The other constituents of the rock appear to have no relationship to slake-durability and localization of problems, with the possible exception of water. Water content seems to be somewhat higher in the top of the Kope formation than in the bottom. An abundance of calcium carbonate within or adjacent to the shale layers accounts for the higher pH between 8 and 9. The pH does not appear to correlate with slake durability.

Tradewater formation. The lower and middle Pennsylvanian strata in western Kentucky consists of the Caseyville, Tradewater, and Carbondale formations. In some areas, there is a clear distinction among these formations in the above order progressing upward (Figure 50), but elsewhere, as a consequence of intricate facies changes in this type of strata, the units are apparently contemporaneous with one another. The Caseyville formation at the bottom of the sequence is usually distinguishable by cross-bedded sandstone in the upper 200 ft. The distinction between Tradewater and Carbondale formation is less obvious since both units consist of sandstone, siltstone, shale, limestone, underclay, and coal; however, the Tradewater is regarded as a major problem formation.

Tradewater sandstone is light to medium gray and is micaceous. The siltstone is light to medium gray and interbedded with thin-bedded sandstone. Shale is apparently quite subordinate to siltstone but does occur consistently in association with coal beds.

Seven samples of Tradewater formation were obtained in the field (Figure 50). The stratigraphic position of the samples within the lower and middle Pennsylvanian sequence is indicated in the figure along with selected characteristics. Since the field sampling was not extensive nor positions well-established, the results in the figure cannot be taken as more than a first approximation. Sample KY 9 contained coaly material and probably was located near one of several coal beds (probably No. 6 coal) found in these strata.

Slake-durability results indicate an interesting distinction between upper and lower groups. The upper samples have indexes of 9 to 20 percent, whereas the lower sample indexes are all above 50 percent. However, the three lower samples came specifically from cuts that provided material used in embankments that had failed; therefore, the lower portion of the Tradewater formation is apparently more troublesome, despite slake-durability results suggesting more deterioration in the upper group.

The group of Tradewater samples shows a prominent correlation between slake durability and test water pH. This correlation is shown graphically in Figure 50 by the approximately parallel trends of these two parameters from sample to sample. The other mineralogical and physical characteristics shown in the figure tend to be either constant or erratic through the strata and therefore suggest no reasons for a consistent difference in the behavior of portions of this formation.

Nolichucky shale. The Nolichucky shale was one of several Cambrian units specified by the Tennessee Department of Transportaion for which problems had been experienced within compacted embankments. Two of the other specified units were the Rogersville and Rome shales lying several hundred feet lower in the stratigraphic section. Sampling in the field was conducted along fresh cuts made for a relocated section of Tennessee Highway 92 south of Jefferson City, Tenn.

Ten samples were taken from cut surfaces along the route. Stratigraphic positions were deduced on the basis of known geological structure. Four of the samples were taken apart from the others, and their positions in the figure depend on the effects of two thrust faults believed to cut the sampling traverse. If the thrusting is insignificant, samples TN 8, 9, 10, and 11 are out of position and should be located in the Rogersville shale. Actually, the Rogersville shale is very similar to the Nolichucky shale, and they are grouped with other units in the Conesauga group where details for subdivision are lacking.

The Nolichucky shale consists of well-bedded, yellowish to olive-green calcareous shale. Beds of limestone to a few inches in thickness commonly occur at irregular intervals (Bridge, 1956). In the sampling program, a variation in texture was noted, as indicated in Figure 51.

The slake-durability index of this material was relatively high, usually well over 50. Nolichucky shale behaved distinctly from all other shale by bulking conspicuously upon breakdown in slake-durability testing. The material (Figures 42 and 43, samples TN 2-10) characteristically degraded into flakes about 1/2 in. broad, not enough to pass through the screen of the slake-durability apparatus. No well-defined trend in slake-durability index is apparent through the strata. Test water pH shows a correlation with slake-durability index, and the pretest water content seems to be inversely related to the index. Hardness, structure, texture, and chlorite and calcite content are shown in the figure but apparently have no consistent variation from top to bottom. Because of the uniformity in appearance and test results, the Nolichucky is inferred to be uniform in its behavior in embankments.

New Providence formation. The New Providence formation occurs in eastern and western Kentucky and has been responsible for embankment problems in both areas. The New Providence formation is approximately 200 ft thick and constitutes the lower half of the Mississippian Borden formation in stratigraphic nomenclature proposed most recently (Hoge and Chaplin, 1972). Figure 52 shows the arrangement and position of this formation in the stratigraphic column near Morehead, Ky., but a similar sequence occurs in western Kentucky between Devonian black shale (New Albany) and limestone of the Meramecian series (Peterson, 1966).

The New Providence consists of shale and siltstone that is dark greenish-gray, to bluish-gray. The siltstone may have thin bedding, and ironstone lenses and concretions are common. The New Providence grades upward into the Brodhead formation by an increase in the siltstone percentage; above that lies the Muldraugh formation, a third member in the newly defined Borden formation. At the top is a thin shale in eastern Kentucky known as the Nada member. The Muldraugh formation contains an appreciable percentage of limestone in complicated facies arrangements with siltstone.

Five samples were collected from New Providence and associated formations. Two of these were taken in western Kentucky and three in the east, but they are combined in the figure because of the similarity in nature of the formation in the two areas. The pertinent test results and characteristics are shown for each sample. Three samples of superjacent rock in the lower Pennsylvanian are shown in the same illustration. The kaolinite and siderite contents of the eight samples appear to show the distinction between the lower Pennsylvanian and lower Mississippian shales. The lower Pennsylvanian shales are richer in kaolinite and siderite as a group. During the collection of samples, failed embankments of New Providence and related shales were pointed out but more troublesome portions of the shale strata could not be pinpointed. Apparently all shale from the Breathitt formation to the top of the Devonian black shale has potential for causing trouble. It is perhaps not surprising, therefore, that the slake-durability and hardness tests show uniformity of results, and among other characteristics, there is no consistent trend progressing through the strata.

<u>Lower Pennsylvanian formation of eastern Kentucky</u>. Three samples of shale from lower Pennsylvanian strata of eastern Kentucky were collected in the field. The shales were identified as problem shales by state highway representatives. The Breathitt and interfingering Lee formations are composed of shale, siltstone, and sandstone. A prominent shale member of the Lee formation occurs at the bottom and includes a bed of flint clay, the Crider clay. Sample positions are shown in Figure 52 along with those of samples of underlying Mississippian rocks.

The three Pennsylvanian samples show inconsistent characteristics except in the content of kaolinite and siderite. Kaolinite is apparently characteristically present in minor amounts and siderite in rare amounts, but both minerals occur in greater amounts than in the underlying Mississippian strata. The outstanding characteristic of these lower Pennsylvanian strata appears to be a measurable content of montmorillonite, either alone or in mixed-layer relation with clay-mica.

Redoak Mountain and associated formations. A thick sequence of middle Pennsylvanian formations from eastern Tennessee was sampled. According to State highway representatives, these coal-bearing shale and sandstone units had presented problems in past embankments and were viewed with concern for future construction. Samples taken at random over a stratigraphic thickness of 1000 ft can be regarded as typical of the entire 4000 ft of middle Pennsylvanian strata (Johnson and Luther, 1972).

The characteristics and test results in Figure 53 provided useful information not only about this sequence but for comparison with Pennsylvania strata in Kentucky (Figures 50 and 52). The slake-durability index was high in three of the four samples. Correspondingly, the water content was low. Samples TN 14 and 17 were freshly broken rock from coal strip mines and gave representative water contents. One of the factors that may account for the high slake-durability index is the comparative abundance of siderite versus the exclusion of calcite and dolomite. The siderite probably constitutes a cementing agent holding the material together and resisting slaking.

Crab Orchard formation. The Silurian shaly unit lying immediately above the Brassfield dolomite is called the Crab Orchard formation in eastern Kentucky and the Osgood formation in western Kentucky. Both units are regarded as troublesome by the Kentucky Department of Transportation and were considered together in this study. Three samples were obtained from each part of the State, and they are shown in their relative positions in Figure 54. This figure is based on the stratigraphic column near Owingsville in eastern Kentucky (Hoge and Chaplin, 1972); the comparable Osgood section in the west is less than half as thick.

As described by Peterson (1969), the Osgood in western Kentucky consists of shale and dolomite. The shale is greenish-gray and pale red and is nonfissile dolomitic shale and mudstone and weakly fissile clay shale. The unit grades into the underlying Brassfield dolomite by interbedding of shale and dolomite. Serious embankment problems have occurred in both Osgood and the Crab Orchard formations near the sampling sites (Table 28). The entire group of samples therefore can be regarded as problem shale. Their nature appears to be reflected in the slake-durability test results for which five of six samples average less than 20 in slake-durability index. Figure 54 shows this trend as a stark contrast to results from the overlying Ohio shale. The slake test water was generally alkaline, but there was considerable variability. Dolomite is a particularly interesting material component. A rather consistent decrease in dolomite content occurs upward through the Crab Orchard-Osgood formations. This trend apparently is related to the presence of dolomite in the shale and the transitional contact with the underlying Brassfield dolomite. One of the Osgood samples contained montmorillonite in mixed-layer relation with clay-mica.

Ohio shale. The Ohio or New Albany shale is relatively thin but very persistent and widespread black shale. The geological age is upper Devonian. The thickness ranges from about 50 to 150 ft in the Ohio, Kentucky, Tennessee region. The shale is grayish-black, very carbonaceous, brittle, and fissile. The four samples collected and tested came from three States but are shown together in Figure 54 in relation to the Crab Orchard formation.

In contrast with most of the other shale units, this widely distributed black shale has relatively uniform characteristics. The water content is 4 percent or less, slake-durability index is consistently near 100 percent, water pH is slightly acidic, and scleroscope hardness is high.

The mineralogy of the unit is also consistent based on sampling in this study. Quartz is consistently abundant, and the dominant clay mineral is clay-mica (illinite). Chlorite is subordinate to kaolinite. A substantial amount of pyrite was present in the four samples examined mineralogically.

Palestine formation. Six samples for the Palestine formation (Mississippian) in southern Indiana were supplied for study by the Indiana State Highway Commission. The samples had been obtained in work for construction on I-64 through Perry County. The Palestine formation is not considered a problem shale but under different construction procedures could conceivably cause problems. The samples were taken along a section of about 4 miles and the stratigraphic position within the thin Palestine formation was not established. In Figure 55, they could be shown only as a group, providing no information on variation through the formation.

Figure 55 shows that the water content and slake-durability index, with one exception, are fairly uniform among the Palestine formation samples. Similarly, the pH of the test water and chlorite content are uniform. The material contained an appreciable amount of siderite, but only one sample contained dolomite. Interestingly, this one sample with a small percentage of dolomite gave anomalously low slake-durability results.

Millboro formation. The characteristics and behavior of samples of the Millboro formation (Devonian) from Clifton Forge, Va., are given in Tables 25 through 27 and related to problem behavior in Part VIII.

Discussion of Observations and Testing

The testing program indicated that, despite the sampling of numerous categories of shale formations with at least several samples, more comprehensive sampling and testing are warranted. The results of the testing of Ordovician strata near Cincinnati suggest that patterns of variation within formations will appear if enough samples are studied.

One of the most obvious needs for further study involves shale younger than Pennsylvanian. Otherwise, they should not be included in any projections from this study.

Typical shale formations and their characteristics. The following categories of shale formations were sampled and studied: interbedded limestone and shale; sandstone, siltstone, and shale formation without coal; sandstone, siltstone, and shale formations with coal; carbonaceous black shale; and thick uniform shale. Most of these category examples came from structurally undeformed areas that except for age effects have undergone approximately the same history since deposition. The selection of examples for testing was designed to exclude structurally or historically complex materials.

Among the various categories of shale, certain generalizations can be made. Each category has its own combination of properties and characteristics that tends to set it apart from adjacent rock units and even other shale. On the other hand, certain variations seem to exist within each shale formation. These are manifested in physical test results or as subtle changes in mineralogy. The lower half of the upper Ordovician sequence, i.e., the Kope and Dillsboro formation, exhibits a general trend of increasing slake durability and increasing calcite content in shale downward from the lower Dillsboro formation (Figure 49). This apparent correlation implies that calcite is acting as a cementing agent in the shale and increasing its durability.

One of the most interesting tests during this study was that for pH of the test water after slake-durability testing. This test may provide very inexpensive information of the chemistry of pore water and perhaps an indication of the ion exchange taking place in the clay minerals.

The scleroscope hardness can advantageously accompany the slake-durability test as an indication of relative material hardness related perhaps to strength and modulus. A disadvantage of this test is the rather expensive cutting procedure for preparing a suitable surface for testing. The test is highly susceptible to specimen configuration and condition, a factor which may limit its usefulness for routine testing.

Jar slake testing. Throughout the test program, a simple jar slake test was conducted along with slake-durability tests. The results of these two tests are shown in Table 26 and Figure 56 for comparison. An approximate parallelism is apparent between the results for the jar slake test of dry material and the slake-durability test. The jar slake test has the advantage of being inexpensive, and many tests can be conducted in the same time as a few slake-durability tests. This study has suggested that a simple jar slake test may be more useful than slake-durability testing in routine highway construction where systematic testing of many samples, e.g., on core pieces, will provide

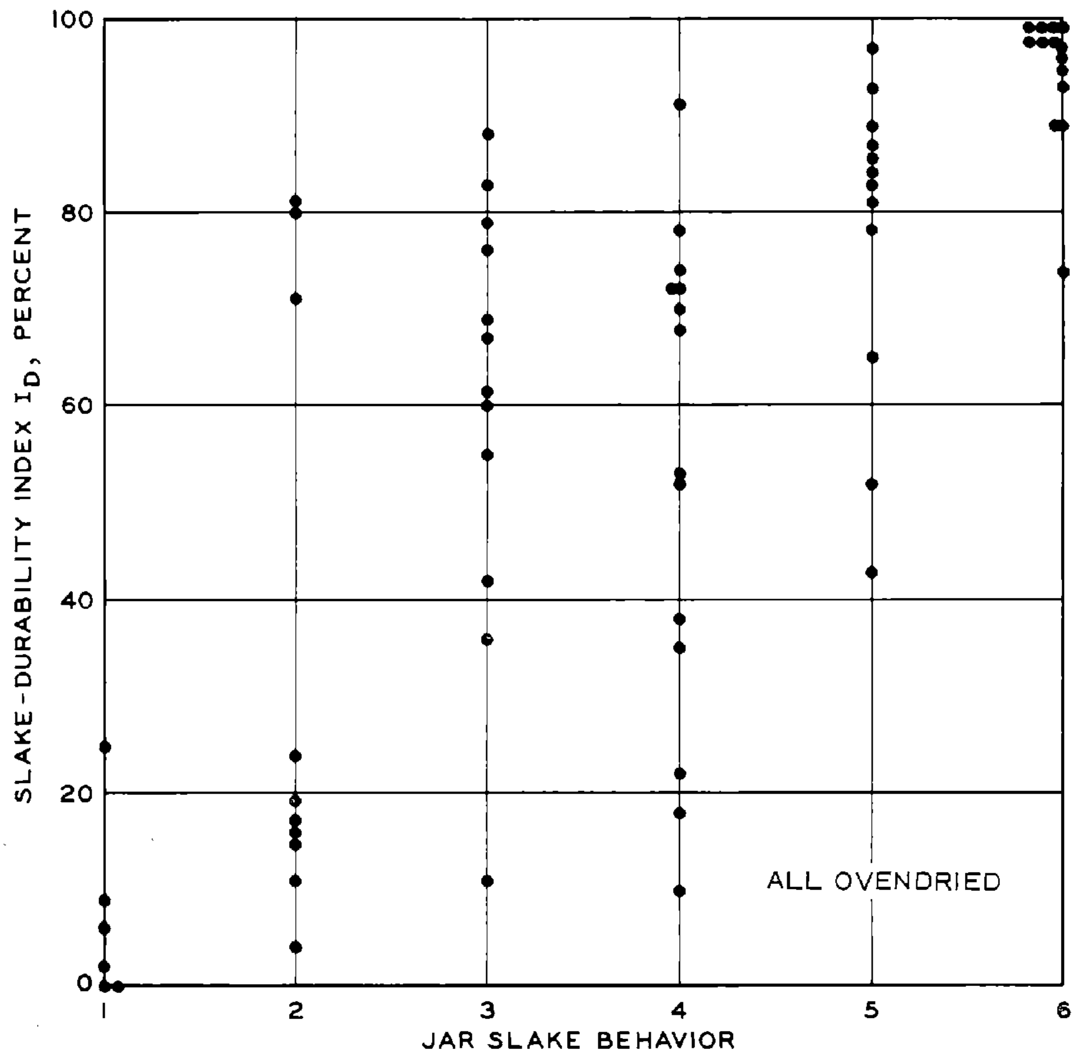

Figure 56. Correlation of indexes of jar slake and slake-durability tests.

continuous data for the geological strata being excavated and used in embankments.

Certain mineralogy is characteristic of certain shale types. Chlorite is consistently found in shales from a marine environment. Kaolinite is characteristic of formations formed mostly in nonmarine environment. To a lesser degree, siderite shows this same nonmarine association.

Montmorillonite was absent from most of the samples tested, and this fact tends to confirm previous observations that it is rare in Paleozoic formations (Part V). A mixed-layer combination of montmorillonite and clay-mica was found in three samples and in one other montmorillonite was present uncombined. Since many of the problem shales contain no montmorillonite, this particular clay mineral is not considered a major factor in embankment deterioration in Paleozoic and older rocks such as those sampled.

Useful parameters. Several of the characteristics used to study shale appeared to be more useful than others. The slake-durability test is useful because it provides index numbers for comparing one material with others previously tested and having known field behavior. The manner of breakdown in the test (Figures 37-48) may indicate embankment breakdown possible in the field. The Indiana State Highway Commission is currently using slake-durability tests to classify shaly materials for use in highway embankments. Certain limits are prescribed at which the designation of the material changes from rock to soil (Wood and Deo, 1975).

Simpler parameters that appear to be useful and to have potential for further clarifying shale problems include water content determined supplementary to the slake-durability test. Natural water content can be determined also, but this determination does not appear to be necessary since values are about the same as those determined prior to slake-durability testing. The water content appears to correlate in most cases with the slake-durability index.

VIII. DETERIORATION PROPERTIES OF SHALES AT KNOWN LOCATIONS OF PROBLEM EMBANKMENTS

A limited assessment of shale deterioration as a cause of problems in shale embankments was attempted as follows.

a. A review of collected information was made to identify problem embankments involving possible shale deterioration and for which the following information was known or could be deduced.

   (1) Specific location.

   (2) Embankment construction.

   (3) Embankment material and source (i.e., geologic unit).

   (4) Problem and extent of foundation involvement.

b. The embankment location and geologic source of embankment materials were used to identify applicable shale test results presented in the variability study (Part VII).

c. The deterioration properties for those shales associated with a problem embankment were then compared to determine if meaningful trends existed.

The problem embankments identified are listed in Table 28 and are limited to locations in Indiana, Ohio, Kentucky, Tennessee, Virginia, and West Virginia. Embankments where foundation conditions were known to be the principal cause of the problem were excluded (three in Kentucky). However, it was not possible to establish the degree of foundation influence for several embankments and these were included in the table with an appropriate remark.

The majority of those embankments listed in Table 28 were probably placed as rock fill except for the I-75 and I-275 interchange fills in Kentucky. These fills were compacted in 1-ft lifts using a sheepsfoot roller.

The applicable shale samples indicated in Table 28 by sample number (from Table 25) were obtained at the embankment location from the same material used in the embankment. The short time and scarcity of new excavations needed in obtaining unweathered shale samples as well as the objective of the variability study prevented sampling at several of the problem embankments listed.

Table 28. Problem shale embankments identified for use in assessing shale deterioration.

State	Location	Original Fill Material	Local Geological Unit	Applicable Shale Samples, (Sample No. on Table 25)	Type of Problem and (Number of Embankments)
Indiana	I-74, 5-mile section in southeastern Indiana, 25 miles northwest of Cincinnati	Shale and limestone	Eden group, Kope formation	IN 5	Slope failure (3), settlement (5 or more)
Ohio	I-75, Warren County at Penn. Railroad north of SR63	Shale and limestone	Maysville group	--	Settlement (1)
	I-74, Hamilton County at Wessellman Road	Shale and limestone	Maysville group	--	Settlement (1)
	SR 22, Fairfield County, west of Rushville at Rush Creek	Sandstone and shale	Logan formation	--	Settlement (1)
	I-70, Licking County, 18 miles west of Zanesville	Sandstone and shale	Massillon member, Pottsville formation	--	Settlement (1)
	SR 57, Lorain County, southeast of Elyria at Black River	Shale	Bedford formation	--	Settlement (1)
Kentucky	US 119, Harlan County, sta 1263, east of Tremont	Sandy clay and shale	Breathitt group, Hance formation ?	--	Slope failure (1)
	KY 1930, Kenton County, sta 0 to 6, near Decoursey	Shale and limestone	Eden group, Kope formation	--	Slope failure (1) foundation involved ?
	I-75 - I-275 interchange fills, Kenton County near Covington	Weathered shale and limestone	Fairview formation	--	Slope instability (1) (no distinct failure plane)
	I-75, Boone-Kenton Counties, mileposts 179.7, 179.9, 181.6, 182.1, and 188.5	Shale and limestone	Bull Fork formation	--	Slope failure (5)
	Western Kentucky Parkway, Grayson County, milepost 92, near Caneville	Clay shale, siltstone, and sandstone	Tradewater formation	KY 12 KY 13	Slope failure (1)
	I-64, Bath County, milepost 118, near Owingsville	Shale and dolomite	Crab Orchard formation	KY 14 KY 15 KY 16	Slope failure (1), partly in foundation
	I-64, Carter County, milepost 152.7, near Vanceburg-Olive Hill	Sandy clay and shale	Breathitt group	KY 20 KY 23	Slope failure (1), partly in foundation
	Blue Grass Parkway, Nelson County, milepost 21	Limestone and shale	Louisville and Sellersburg limestones underlain by Osgood shale formation	KY 1 KY 2 KY 3	Slope failure (1), foundation involved ?
Tennessee	I-75, Campbell County, Caryville to Jellico	Shale and sandstone	Hance, Lee, and Pennington formations	--	Slope failure (approximately 8 embankments)
West Virginia	US 460 east of Princeton	Shale and sandstone	Hinton formation	--	Settlement (4)
Virginia	I-64, Clifton Forge	Shale	Brallier and Millboro formations*	VA 1 VA 2 VA 3	Settlement (?) and deformation of embankments

Comparison of Results

The five problem embankments and test results on unweathered shale samples associated with these embankments are listed in Table 29. Several important points can be made from this limited comparison.

a. The natural water content and slake-durability index suggest that deterioration would be expected for most of the shale samples associated with the embankments in Indiana and Kentucky. The low jar slake indexes of 1 and 2 correspond to very low slake-durability indexes and indicate that deterioration could be determined quickly without the slake-durability tests. Jar slake values of 4 would require slake-durability tests as a second step. Similarly, the scleroscope index could not be used alone as an indicator of susceptibility. These last two tests require further refinement as indicated in Part VII.

b. The relatively high average slake-durability index (I_D) values of 84 and 91 percent for two of the shale samples for embankments in Kentucky (KY 12 and KY 20) suggest the importance of the type of breakdown during the slake-durability test. Both samples had an abundance of quartz (usually associated with stronger shales) and broke apart into numerous tabular pieces, as shown in Figures 38 and 40, although there was little loss of material. Large blocks of this shale placed as rock fill could break apart (e.g., under stress at block contacts). The presence of large voids would allow broken pieces to move into void spaces and produce a general loosening leading to settlement. Weathered shale or overburden soil, incorporated in the fill during construction, could be washed downward (by infiltrating water) into the smaller voids and eventually form a weak soil matrix around the broken shale pieces and lead to eventual slope instability.

c. All three shale samples from Clifton Forge, Va., had low water contents, very high I_D values (97 to 98 percent), and an abundance of quartz. One sample, VA 1 (Brallier formation), is not considered a problem shale in Virginia. The samples did not break apart significantly during the slake-durability test. However, the low pH indicated high acidity conducive to chemical weathering as discussed in the following section.

Example of Chemical Weathering

The material used in the rock fill for the Clifton Forge bridge approach embankments consisted of hard, laminated, tan to black shale of the Millboro formation (Devonian age). Results of tests on samples of

Table 29. Comparison of shale test results for selected problem embankments.

State	Location	Local Geologic Unit	Shale Sample No.	Natural Water Content Before Test, percent	Average Slake-Durability Index, I_D percent	Jar Slake Index, Ovendried Samples	Scleroscope (Hardness) Index, Perpendicular Damp	Scleroscope (Hardness) Index, Perpendicular Dry	Residue Water pH	Relative Quartz Content*
Indiana	I-74, southeastern Indiana	Eden group, Kope formation	IN 5	9.3	38	4	14	25	8.2	C
Kentucky	Western Parkway, Grayson County	Tradewater formation	KY 12	--	84	--	--	--	7.5	A
			KY 13	9.0	53	4	17	33	7.2	C
	I-64, Bath County	Crab Orchard formation	KY 14	3.3	65	5	14	--	8.2	C
			KY 15	8.3	4	2	18	23	7.3	C
			KY 16	14.8	0	1	--	--	7.4	C
	I-64, Carter County	Breathitt group	KY 20	4.8	91	4	20	29	7.2	A
			KY 23	8.6	6	1	17	32	8.1	A
	Blue Grass Parkway, Nelson County	Osgood formation	KY 1	7.1	15	2	15	26	--	C
			KY 2	20.3	9	1	--	--	9.2	C
			KY 3	18.3	11	2	--	--	9.4	C
Virginia	I-64, Clifton Forge	Brallier**	VA 1	1.1	98	6	39	51	4.6	A
		Millboro	VA 2	0.6	99	5	37	40	6.1	A
		Millboro	VA 3	1.4	97	6	44	--	5.2	A

* A = abundant, greater than 50 percent; C = common, about 25 to 50 percent.
** Nonproblem shale.

this shale, collected from the cut section adjacent to the embankment fill, have been presented in Table 29. The slake-durability indexes and the Shore hardness values suggest that this shale is quite hard and presumably durable. Acid environmental conditions are indicated by the sample pH determined after the slake-durability test.

The Virginia Department of Highways and Transportation collected split spoon samples from borings on the embankment. The boring plan is shown in Figure 57. Portions of the sampled material were sent to the Virginia Highway and Transportation Research Council (VHTRC)* and to WES for examination.

The blow counts given on the boring log showed that the material was relatively soft at some depths. Blow counts ranged from a low of 5 to a high of 50. The water contents were also variable and ranged from a low of 3.0 to a high of 28.9 percent.

Visual inspection of split spoon samples received at WES indicated that two types of material were present in each sample. The two types were (a) relatively fresh, hard, gray to tan, relatively well-laminated shale frequently exhibiting silt laminae and (b) highly weathered moist shale exhibiting the characteristics of clayey, silty soil. Both types of materials appeared more-or-less intermixed in each sample although one type would usually predominate.

Examination of the relatively fresh rock by binocular microscope revealed the presence of the predominate quartz, some silt-size mica (10 percent or less), and some pyrite (10 percent or less). Planes and fractures were coated with a yellowish mineral believed to be jarosite (substantiated by XRD) and occasionally contained silt-size crystals believed to be gypsum.

XRD analyses were conducted by VHTRC and WES** on the clay-size fraction and bulk sample, respectively, from selected borehole samples. These analyses revealed that the clay mineral constituents in decreasing abundance were illite, kaolinite and/or dehydrated halloysite, and

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* David F. Nobel (1975), personal communication.
** Allen D. Buck (1975), personal communication.

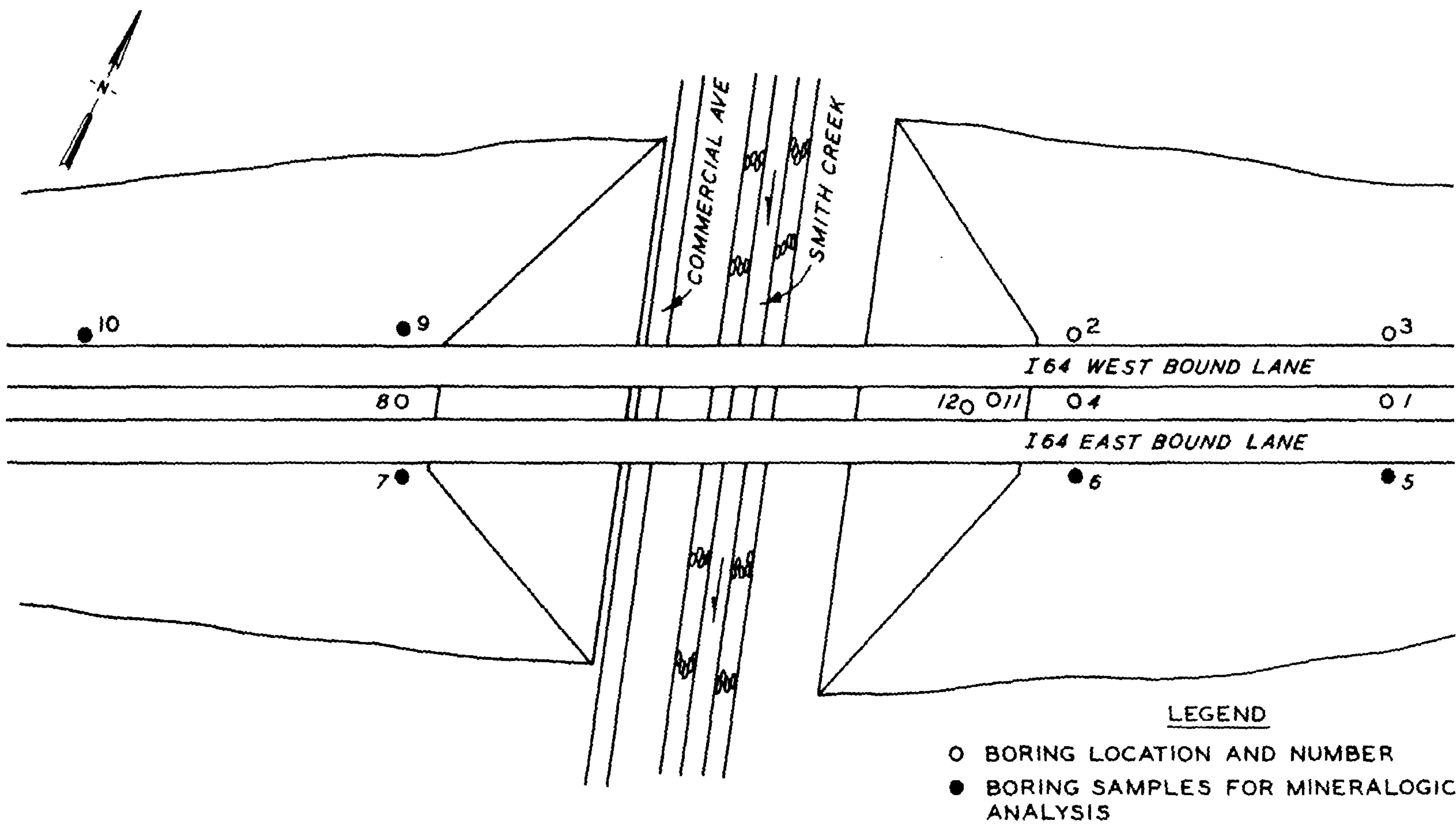
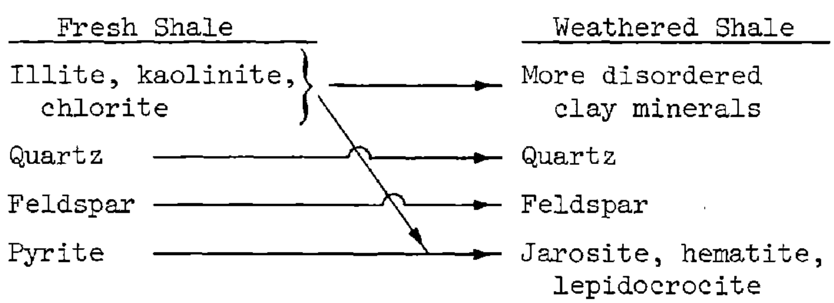


Figure 57. Schematic plan of boring locations, I 64, Clifton Forge, Va.

chlorite. The nonclay mineral component was predominately quartz accompanied by smaller and variable amounts of feldspar, pyrite, hematite, jarosite, and lepidocrocite. Carbonates were not detected in any sample.

XRD analyses at WES were conducted on both the fresh and weathered material. Analyses of the weathered samples indicated a significant decrease in pyrite and increase in jarosite, hematite, and lepidocrocite. The X-ray diffractograms of the weathered samples suggested that the clay minerals were more disordered structurally and that more amorphous material probably was present.

The alteration of the embankment material is shown in the flow diagram below.



The processes which lead to the softening and deterioration of the shale are believed to be mainly chemical in nature and were initiated by the weathering of the pyrite and the production of sulfuric acid which in turn attacked the clay minerals (see Equations 2 and 6, pages 128 and 134). The acidization of the clays resulted in an increase in their structural disorder and an increase in their hydration state and plasticity.

The foregoing analyses have shown the necessity of a complete test program for materials used for large embankments. It is noteworthy that the slake-durability index was not predictive in this case. However, the pH test might have been the warning that acid conditions were present and therefore that more attention was needed on the mineralogical composition of the embankment material.

IX. CONCLUSIONS AND RECOMMENDATIONS

Conclusions

All of the States in this study are having or have had problems with embankments constructed with shale except for Oregon, Pennsylvania, and North Carolina. The known causes of the unstable conditions of the embankments were identified by the State highway representatives to be the following:

- *a*. Excessive lift thicknesses.
- *b*. Inadequate compaction (specifications and control).
- *c*. Deterioration (physical and/or chemical).
- *d*. Expansive characteristics.
- *e*. Excessive steepness of side slopes.
- *f*. Infiltration of water.
- *g*. Lack of sidehill benching.
- *h*. Inadequate drainage.
- *i*. Random mixing of shale with harder rock types (limestone, sandstone, etc.).
- *j*. Failure to consider all geological conditions.
- *k*. Lack of established tests and criteria to reliably predict shale behavior after emplacement.

These causes are not necessarily listed in order of importance and may not have been experienced by all of the highway departments. One or any combination of these factors may contribute to unstable conditions in compacted shale embankments. One or more of these factors may contribute to or initiate the occurrence of another factor or factors. Other unidentified causes could be contributors to embankment problems.

The above causes except *c*, *d*, and *k* have rather obvious solutions, such as using flatter slopes, thinner lifts, increased compaction, and more extensive drainage. However, a requirement for the general use of all these measures may not be warranted and would be expensive, both to the design and construction of a project. Consequently, criteria are needed to determine when specific measures are actually required.

The capability of predicting the performance of shale used as an embankment material is lacking in almost every instance. This lack is due in part to the variability of the physical character of shales in their natural environment and the lack of proven test methods to classify shales in terms of durability when used as a construction material.

The shale occurrence maps developed in the study show a widespread distribution in all of the States concerned except four. In California and Oregon, most of the shale occurs along the coastal areas, while in North Carolina, shale occurs in a limited area in the Piedmont. Shale in Virginia is restricted to the Appalachian region in the western part of the State.

The amount of shale that occurs in a State is not necessarily proportional to the number of problems encountered in constructing shale embankments. For example, shales in West Virginia occur throughout the State in various quantities; however, the problem areas are located in the southern and northwest part of the State. Shale is widespread in Oklahoma; however, problems are restricted to the eastern part of the State within the higher rainfall belt. The States east of the Mississippi River have more severe problems with shale embankments than those to the west. The Cretaceous formations in the western States have swelling characteristics which are presenting the major problems. In contrast, the eastern States did not report any swelling problems associated with the shales.

All of the 16 States concerned have recognized the problems in using shale in embankments, and 12 States indicated problem shale formations ranging from one to two in three of the States to nine in one State.

Nine States compact shale only as a soil and seven States place shale either as soil or as rock, according to current highway practices for design investigation and construction of shale embankments. Three of the States that use shale as rock fill have special provisions or criteria for determining when a shale can be used as rock fill. Indiana uses the results of laboratory durability tests (see Figure 27) define rock-like shale and limits the lift thickness to 24 in. West Virginia classes shale as soft or hard depending on whether it can be broken down with the approved compaction equipment and also limits the lift thickness to 24 in. Kentucky has reduced the allowable lift thickness to 12 in. desirable and 18 in. maximum for shale materials considered a problem from past experience. California specifies the maximum lift thickness for all rocky materials on the basis of percent of rock having a size greater than 6 in. and requires 90 percent relative compaction. In general, shales placed as soil and adequately compacted have not presented embankment problems.

Current design investigation and construction practices for those States constructing shale embankments are not uniform. Obviously weak practices by some States include relying on the project engineer's judgment for density control and not specifying the criteria to be used. Acceptance or rejection criteria are not sufficiently quantitative or standardized among the States. Field exploration practices vary significantly between States, and criteria used have not generally provided the necessary data for proper construction. Segregating shale from hard rock (limestone and sandstone) is often not economically feasible and is not generally practiced. Using a mixture of shale and hard rock as rock fill can contribute to unstable conditions through differential weathering after placement.

Identifying exact locations of problem shale embankment was not accomplished in all States. One or two locations of shale embankments currently exhibiting distress were indicated in four States, and three States indicated several locations. The source of the shale used in a distressed embankment with regard to its position in the stratigraphic column was not identified except in a few cases. Both types of data somewhat restricted the completeness of this part of the study. Records of construction were indicated to be insufficient in detail to be useful in determining specific construction conditions and procedures, sources, and sequences of materials used in embankments, and types of compaction equipment and procedures. Thus, important information needed for determining the influence of possible factors contributing to shale embankment distress and failures was not obtained.

The major sources of experience with shales in embankments other than for highways were the CE and, to a limited extent, the USBR. The successful use of shale materials generally from spillway excavations in some 26 CE and 3 USBR earth and rock-fill dams provides pertinent information on shale durability, use of test fills, compaction procedures, and engineering properties of compacted shale, including considerable data on shear strengths.

With only a few exceptions, shale materials for CE projects are processed and/or broken down and compacted using heavy rollers such as tamping rollers for lift thickness of 8 in. and a 50-ton rubber-tired roller for lift thickness up to about 18 in.

The infrequent use of even durable shales or shale-rock mixtures as compacted rock fill (compacted by a 10-ton vibratory roller) indicates the importance of compacting shale materials as soil to densities equal to or higher than 95 percent of standard effort (AASHTO T 99) maximum density. No settlement or stability problems have been experienced with compacted shales in dams in the States of interest.

Considerable data on unconsolidated, undrained (Q) shear strength of compacted shales indicate a significant decrease in strength with increases in water content. The consolidated, undrained (R) shear strength of shales saturated prior to shear is significantly lower than that of shales in an unsaturated condition. However, the consolidated, drained ($\overline{R}$ or S) shear strengths of compacted shales are high and about equal to the R strengths for unsaturated shales. Thus, saturation leads to high pore water pressures during increases in loading and a large reduction in strengths based on total stresses. The strength data presented are considered useful for design guidance.

The probable intrinsic causes of shale deterioration are a combination of chemical, physicochemical, and physical processes which result in weakening and softening of the material. These processes affect the clay and nonclay mineral constituents by changing the hydration character of the clay minerals and by weathering the nonclay minerals. Shales containing expansive clays such as montmorillonite as a discrete mineral species or in mixed-layer form are potentially troublesome. Among the nonclay mineral constituents, the presence of pyrite, jarosite, and secondary gypsum suggests susceptibility to weakening by weathering within the embankment. Most of the potential deterioration processes are caused by and occur in the aqueous environment present in the embankment.

The tests believed necessary to predict potentially unsound shales include XRD, slake-durability tests, dispersion tests, and standard soil mechanics tests are applicable for the materials in question. The slake-durability index appears to be particularly useful but more study is needed with this method.

The laboratory testing conducted as part of the study of shale variability indicated, in achieving the objective of covering several categories of shale formations with at least several samples, that more comprehensive sampling and testing are warranted. Results of the testing of Ordovician rocks near Cincinnati suggest that variations within the formation will appear if enough samples are studied. Further study of rocks younger than Pennsylvanian age is one obvious need; present results cannot be used in any projections of the variability study to younger rocks.

The time available limited this part of the study to problem shale formations in Indiana, Ohio, Kentucky, Tennessee, and Virginia where the problems with shale embankments were more prevalent than in other States.

For each of the several categories of shale formations sampled and studied, a unique combination of properties and characteristics was evident and set the formation apart from adjacent rock units and even other shale. Conversely, certain variations within each shale formation were apparent from physical test results and subtle changes in mineralogy. Certain mineralogy is characteristic of certain shale types, e.g., chlorite in shales from a marine environment; kaolinite and siderite in shales from a nonmarine environment. Shale structure and texture appeared to have little bearing on the results of tests for durability or on the localization of embankment problems.

Results of the laboratory study indicated that the slake-durability test using standard procedures provides a useful index number related to shale durability. The greatest potential appears to be for use in a routine manner in which the test can provide an index for comparison of one material of interest with others previously tested and with known field behavior.

Several simpler tests appear to be useful. The natural water content of as-received samples (sealed in plastic bags) correlated in most cases with slake durability. The pH of the slake water after tests may provide an inexpensive bit of information on the pore water chemistry and perhaps some indication of ion exchange occurring in clay minerals.

A simple jar slake test showed an approximate parallelism with the slake-durability test. This simple test may be more useful than the slake-durability test for routine highway construction. Systematic testing can be done quickly on many samples such as on core pieces to provide continuous data for the geological strata being excavated and used in an embankment.

The scleroscope test and resulting hardness index is a useful supplement to the slake-durability test. However, the cutting procedure required to prepare a suitable surface for testing is relatively time-consuming and thus expensive. The test results are highly susceptible to specimen configuration and conditions and may limit its usefulness for routine testing.

The results of a limited evaluation of shale deterioration properties of unweathered samples from the location of five problem embankments indicated that physical shale deterioration was a primary factor in most cases. However, the type of breakdown in the slake-durability test, especially when high I_D values are obtained, is important and indicates that the I_D value may not be meaningful for hard shales that break apart only along bedding or other discontinuities. The I_D value using distilled water for the test is not an indicator of possible chemical weathering except in conjunction with pH measurements of the residue water and XRD tests as indicated from a study of shales at one location in Virginia. The slake-durability test also may not be applicable for other shales which soften but do not degrade. The use of the simple jar slake test as the primary test in such cases needs to be investigated. The applicability of the ultrasonic disaggregation test needs to be evaluated as an alternate method when the slake-durability test appears inadequate and for possible use in predicting rate of deterioration with time.

Recommendations

It is recommended that Phase III be initiated and that priority be given to the following items:

<u>a</u>. Extend the study to assess shale deterioration of other younger formations not studied, especially in the western States. This is considered important in establishing the validity of simpler index tests and the limits of applicability of the slake-durability test. Some shales may soften and become plastic but remain intact. In such a case, no loss of material would occur and a highly misleading slake-durability index would result.

<u>b</u>. Develop an economical laboratory test to evaluate the decrease in shear strength with time after compaction.

<u>c</u>. Develop a suitable and economical laboratory type test that also can be used in the field to evaluate compressibility of representative gradations, loads, and groundwater conditions expected in shale embankments.

A. APPROACH FOR PHASES II AND III

Phase II: Strength and Compressibility Evaluation and Remedial Treatment of Existing Embankments

Current methods and procedures used to determine in situ conditions, in situ strength, and compressibility and stability of existing embankments will be identified and evaluated, and the state of knowledge on material properties will be summarized. This portion of the study effort will be performed concurrently with the collection of information and data (Phase I).

Representative embankments of shale mixture typifying problem conditions not associated with foundation distress identified during Phase I contacts with State and Federal agencies will be selected for detailed studies to develop probable causes of embankment distress. If required information is found, the properties of the materials as they were excavated and changes occurring during compaction will be compared with material properties existing in the embankment.

The results will be developed to provide preliminary information on causes of distress in existing embankments of compacted shale mixtures and methods for evaluating existing strength and compressibility properties and expected changes with time.

Based on the type of detrimental conditions identified in the field, feasible remedial measures identified under Phase I, Task A, will be evaluated to determine field conditions for which a given treatment will be effective. Remedial measures considered will include:

- a. Drainage systems.
- b. Lime or cement slurry injection and grouting.
- c. Chemical injection.
- d. Electrokinetic chemical infusion.
- e. Electroosmotic stabilization.

This phase of the study will provide preliminary results and suggested remedial procedures required to develop the complete methodology.

Phase III: Development of Design Criteria and Construction of Control Techniques

Many of the tasks to accomplish the desired research will be studied under Phase III. For that reason tasks are discussed individually.

Task A, sampling program. Field observations, field tests, and a sampling program, based on a review of important intrinsic factors and needed type and quantity of samples identified under Phase I, will be formulated for trial use with current state highway department sampling programs for shale embankments in problem areas. A research civil engineer from the WES research team experienced in field sampling and testing will work in the field to provide technical guidance on modifications required to obtain suitable samples. If warranted, WES will perform limited supplemental sampling to establish practical methods and sample size limitations for cores, blocks, and fragmented in situ materials required for testing of shale mixtures. Guidelines on allowable sample disturbance and weathering will be verified. Required samples will be obtained for laboratory studies at WES as discussed in subsequent tasks. From the field study and results of Phases I and II, a recommended sampling program will be developed.

Task B, index tests. The applicability of index tests identified under Phase I will be determined for use in predicting the behavior of shale mixtures considering important intrinsic factors and results of field studies under Phase II. Necessary modifications or new index tests will be determined, using appropriate tests or examination techniques identified under Phase I, Task B, as being related to strength and compressibility behavior.

Task C, laboratory tests. Information from the literature, results of field data and observations, and past experiences with laboratory testing of compacted shale mixtures to duplicate prototype conditions will be used to prepare a needed test program for compaction and shear strength. Samples of shale mixtures used in embankments exhibiting both stable and unstable behavior obtained while conducting studies under Phases II and III, Task A, will be used. The WES Soils Research

Laboratory compaction and triaxial shear testing equipment that accommodates 18- and 15-in.-diam samples, respectively, will be used. Preliminary tests will be made to develop methods for adding desired water and determining curing conditions representative of prototype conditions. Tests will then be conducted to determine the effects of scaled gradations with 6-in.-diam specimens on consolidation and strength properties when compared with companion tests on specimens containing near prototype gradations and maximum grain size. Specimen preparation by kneeding compaction and impact compaction will be compared to determine degradation and the procedure most suitable to duplicate field compaction methods.

Task D, test strips, evaluation of short-term characteristics. Based on review of past experience with test embankments and the current state of the art gathered under Phase I, a program will be developed for constructing test strips of shale mixtures, control testing, and evaluating the results. Procedures will be recommended for field tests to determine compressibility and shear strength applicable to end-of-construction conditions. Technical guidance will be provided for construction, testing, and evaluating test strips in connection with current highway projects. Technical personnel will be provided to conduct the field tests, and the results will be evaluated. Recommended requirements for test programs will be developed using test strips and methods for conducting the tests and evaluating the results.

Task E, test strips, evaluation of long-term characteristics. Possible methods of testing compacted mixtures in the laboratory or in the field (such as large-scale shear and compression tests with cyclic wetting and drying) will be considered. However, long-term strength and compressibility characteristics depend in part on seasonal cyclic wetting and drying, softening of the initially hard particles, stress changes, deformations, and intrinsic property changes from physicochemical phenomena. These changes could not be adequately represented in test embankments or in the laboratory. A more feasible approach involves instrumentation of aged embankments of shale mixtures exhibiting distress. Piezometers to measure pore pressures, and slope indicators or plane-cased holes with sounding pipes on cables to locate shear planes, could furnish in situ data for backcalculating an averaged long-term shear strength from stability analyses. This approach has been successfully used by WES in recent studies of clay shale slopes along the Panama Canal. Field shear tests at other aged embankments exhibiting no instability or deformation could be used to obtain limited data on long-term shear strength of competent embankments of shale mixtures. Limited data on compressibility characteristics could be obtained from large-scale plate loading tests in test pits in existing aged embankments.

Task F, field compaction and control. The results of previous tasks, collected field information, and in-house experience with embankment compaction and control techniques for earth and rock-fill dams will be assimilated and evaluated. Specific recommendations will be developed with detailed guidance for varying field conditions. Use of heavy compaction equipment or shale breaker rollers to break down large particles instead of separating out large particles on a grizzly would be considered in cases in which extrapolation of laboratory data is not feasible. Methods of relating field in-place density test results with comparable compaction test results on scaled gradations of material will be studied. Guidance for proper methods to use under specific field conditions will be emphasized and suggested guide specifications will be developed.

Task G, pretreatment techniques. Information gathered from the field, other agencies, the literature, and in-house experience on compacted shale masses will be evaluated to identify and evaluate the effectiveness of pretreatment techniques based on past experience. Treatments such as soaking borrow areas, adding chemical or lime during compaction, batch mixing, and effectiveness of various types of compaction equipment will be included in the study. Recommendations will be made and tests suggested concerning the selection of suitable pretreatment techniques and compaction equipment for different types of shales.

Task H, design and analysis considerations. Results of all previous tasks and accumulated in-house experience will be reviewed, evaluated, and condensed to provide detailed technical guidance for design, analysis, and selection of necessary features.

B. CHECKLIST FOR INFORMATION ON DESIGN AND CONSTRUCTION OF COMPACTED SHALE EMBANKMENTS

Definitions

a. Shale - the term shale for the purpose of this study includes shale, clay shale, claystone, siltstone, and mudstone.

b. This study is restricted to compacted embankments only and excludes cut slopes and foundation conditions. Embankments greater than 3 ft high are to be considered.

Personnel Contacted

Names, position or title, address, and telephone number.

General

a. Map of Federal and State highway systems.

b. Organization chart of highway departments.

c. Personnel responsible for design, construction, and maintenance.

d. Copy of contract specifications for embankments (or standard specifications for highways).

e. Sources used by State to obtain guidance for geology, design studies, construction, and remedial treatments.

f. Availability of published reports pertinent to embankments.

g. List of types of information furnished.

Experience

a. Stability problems in embankments:

   (1) If no problems exist, are problem shale materials used successfully because of special construction processing techniques?

   (2) Location and areal extent of shale or similar sedimentary rock formations from which borrow material has caused embankment stability or deformation problems; location and areal extent of nonproblem shale or similar sedimentary formations based on local experience.

(3) Specific locations of embankment problems and borrow sources and availability of samples.

(4) Type(s) of embankment problems (excessive settlement, slope failure, surface sloughing, etc.).

(5) Techniques used to monitor embankments.

(6) Methods used to evaluate and determine source of problem (types of field investigations, field tests, and instrumentation; types of laboratory tests and special tests).

(7) Factors causing problem (softening, deterioration of embankment materials, chemical change, seepage, inadequate processing of borrow materials, inadequate compaction, and combination of factors).

b. Preconstruction or design investigations:

(1) Geological investigations (type and how detailed).

(2) Field sampling and testing (type, frequency, type of information obtained).

(3) Laboratory tests (type for engineering properties, chemical analyses, mineralogical data, etc.).

(4) Classification systems or criteria for determining suitability of borrow or excavated material for use in embankment fill (Atterberg limits, gradation, slaking, etc.) as either rock fill or as soil fill (after some special processing during construction).

(5) Criteria for compaction, settlement, shear strength, and moisture or seepage control.

(6) Evaluation of end of construction and long-term stability, deformation, and compressibility. Are any embankment slope stability analyses performed?

c. Construction practices:

(1) Processing or pretreatment of borrow or excavated materials used in embankment fills (type equipment and procedures).

(2) Field compaction specifications (type of equipment, loose lift thickness, number of passes, etc.).

(3) Control during construction (gradation, in-place density, moisture content, etc.).

(4) Embankment slopes, base and crown widths, height, zonation, seepage measures, surface water control on fill, and prevention of saturation after placement.

d. Maintenance and remedial treatment:

(1) Types of remedial treatment used (replacement, drainage, chemical stabilization) and procedures for evaluating and selecting best procedures to use.

(2) Subsequent behavior of selected remedial treatments (which types work best for different situations).

Special Investigations or Research Efforts

a. Test fills or embankment sections: location, age, water content, density, present conditions, type of material, construction history, type of borrow source, types of samples, field tests, instrumentation (pore pressures, stresses, movements), laboratory tests, results, availability of samples and data to WES.

b. Embankment evaluation techniques: type of tests conducted, were results meaningful (if not, why not), which types of tests are more promising, could predictions be made of future competency of the embankment section, and were predictions accurate.

c. Available reports.

Specific Projects

a. Availability and types of information on specific project, where located and who to contact, and can copy of information and data be mailed to WES.

b. Availability of current problem embankment section for detailed study by WES, what assistance could be given by highway department forces, and type of information available on borrow areas, design investigations, and construction.

c. Are there any test fills or special embankment sections being investigated or planned for investigation within next two years in which WES could participate by providing technical input, observation, and modest field investigations?

d. Are there any remedial projects under way or planned in which WES could participate within the next two years?

e. Are there any studies of geological and engineering properties of sedimentary formations within the State and variability of properties of different strata with areal extent or along particular highway routes?

Engineering Properties of Material Used in Compacted Fill and Test Procedures

a. Classification data.

   (1) Gradation.

   (2) Atterberg limits.

   (3) Specific gravity.

b. Test data on degradation, slaking, abrasion, etc.

c. Compaction data.

   (1) Type of test and materials tested.

   (2) Size of mold and rammer.

   (3) Test procedures.

   (4) Test results (optimum water content, maximum dry density).

d. Consolidation data.

e. Shear strength data.

f. Field test data.

Geological Information Desired

a. Petrology.

   (1) Classification.

   (2) Grain size distribution, including percent clay (-2μm).

   (3) Cement(s).

   (4) Color.

   (5) Other visual identification characteristics.

   (6) Fabric (micro), bedding, etc.

b. Mineralogy.

   (1) Clay-fraction; clay minerals + others.

   (2) Nonclay fraction.

   (3) Cement(s).

   (4) Quantitative mineralogy.

   (5) Organic.

c. Stratigraphy.

(1) Formation.

(2) Age.

(3) Thickness.

(4) Facies changes.

(5) Vertical homogeneity.

(6) Areal extent.

(7) Environment of deposition.

d. Physiography.

(1) Outcrop occurrences.

(2) Exploration techniques.

(3) Soil development; type, amount, etc.

e. Structure.

(1) Shear zones.

(2) Slickensides.

(3) Folding.

(4) Faulting.

(5) Evidence of metamorphism or hydrothermal alteration.

(6) Extent of diagenesis and/or consolidation.

Variation in Material Properties of Problem Shale

When problem shale strata or formations are indicated during visit, the following information is needed (from State geological agencies or other sources that may be suggested).

a. Source of information, name of person, title, address, telephone number.

b. Areal extent, access, and exposure for further field study.

c. Availability of existing data.

(1) Air photos, geological data, extent of areal coverage, and extent of study.

(2) Borings and samples.

(3) Field and laboratory test data.

(4) Available reports.

d. Arrange to obtain as much detailed data as possible during visit or by mail or sources for further contacts.

C. DESCRIPTION OF ROLLED SHALE-LIMESTONE FILL AT TUTTLE CREEK DAM, KANSAS

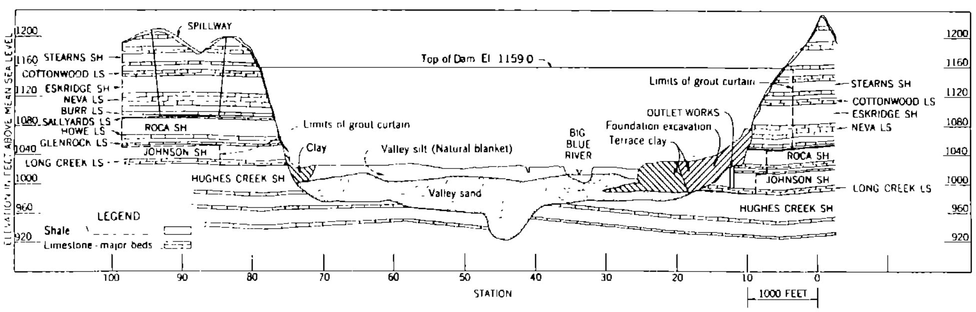


Figure 2. Geological profile along axis (looking downstream).

* Reprinted by permission from:

Lane, K. S., and Fehrman, R. G., "Tuttle Creek Dam of Rolled Shale and Dredged Sand," Transactions of the American Society of Civil Engineers, Vol 127, Part I, 1962, p 492.

SHALE-LIMESTONE FILL*

Although there has been considerable usage of the soft rock variety of shales in dams (such as the huge volumes of clay shale excavated by earth moving equipment and rolled into Garrison[5] and Oahe Dams[6]), there has been comparatively little experience in utilizing the harder type of shales which are generally excavated by blasting. Youghiogheny Dam is one of the few recorded cases.[7] There shale chunks were reduced to minus 6-in. size by a crusher, delivered to the dam by a belt conveyor, and then compacted by tamper rollers. With the good experience in subsequent years using a heavy rubber tire roller for compacting granular soils containing aggregate to cobble size, it was believed entirely practical to utilize as compacted fill some 7,300,000 cu yd of bedrock excavated from the spillway. As shown by Fig. 2, this consisted mostly of shale but also included numerous limestone beds, which it was neither practical nor desirable to excavate separately.

The problem was approached by constructing several test fills with a variety of shales and compaction equipment,[6] and then by including about 1,000,000 cu yd of shale-limestone fill in the first contract to serve as a large scale test embankment. The methods evolved and used in the subsequent major embankment contracts were essentially as follows, with the objective being a well-graded dense fill. The shale-limestone mixture was placed in 18-in. lifts and compacted by 3 passes of a rubber tire roller carrying 100,000 lb on 4 wheels. To prepare the lift for rolling, the specifications required that the material be "conditioned" by removing over-size pieces and by breakage in blasting and/or on the fill to produce a well-graded mixture with sufficient fines to substantially fill the voids. Over-size pieces (those exceeding the lift thickness) were raked to the outer slope of the dam directly behind the riprap using a large rock rake capable of raking to full depth of the lift. The rake included 4 teeth, 5 in. thick, spaced 24 in. apart, and weighing about 400 lb per tooth. A dual drum spike-toothed roller, weighing about 55,000 lb or 4,500 lb per lineal foot of drum, was used for breakage on the fill. Drums were 6 ft in diameter and each was equipped with 36 spikes consisting of a 12-in. long triangular section of 4-in. plate. Generally it and the spreading bulldozer served to break most of the shale chunks, leaving only larger pieces of limestone and hard limey shale to be raked out as over-size. However, the real secret of conditioning the material was in good breakage during blasting. This was accomplished with 4-in. blast holes usually on 12-ft x 14-ft centers, although the pattern was varied some with the rock conditions. Powder was ammonium nitrate and consumption averaged 0.5 lb per cu yd. With thorough breakage in blasting, the slow and more costly breakage on the fill was greatly minimized, and the material could be rolled promptly after placing, as no moisture control was required.

Results were excellent and exceeded expectations. A very dense fill was produced with only a minor amount of visible voids, as revealed by occasional test pits. Such were hand dug since a power auger proved unable to penetrate through the dense chunky fill. Dry density averaged about 115 lb per cu ft, and was determined by excavating a hole of about 1 cu ft size, lining it with a plastic membrane and then filling with water to determine the volume. Bulking factor from excavation to embankment was only about 5%. Costs for the two main earth-work contracts were $0.15 per cu yd for placing on the

[5] "Embankment Soil Characteristics, Garrison Dam," Corps of Engrs. Report, Garrison Dist., June, 1951.

[6] "Materials and Compaction Methods, Missouri Basin Dams," by P. T. Bennett, Transactions, 6th World Congress on Large Dams, 1958. Data presented for Tuttle Creek Dam is largely that from the test fills.

[7] "Belt Delivers Fill for Earth Dam," Engineering News Record, December 3, 1942.

fill and \$0.45 per cu yd for excavation, including haul from spillway to left bank embankment, in which respect the latter figure increased to \$0.61 for the longer haul to the opposite bank involved in the second contract.

For analysis purposes, two limiting cases were assumed and the dam designed to be safe for either condition. *First, it was considered the shale-limestone* could act as a highly pervious fill and produce full reservoir *pressure at upstream side of the central core. Second,* it was assumed the material would ultimately weather to a clay, for which effective shear strength was assumed as a 24° friction angle and zero cohesion. After seeing the fill actually produced plus the relatively minor slaking of the shale on the excavated slopes, it now seems that the second assumption was unnecessarily conservative. For such a dense fill, it seems more likely that circulation of the weathering agents, air and water, will be relatively minor so that the time required for the great bulk of the mass to weather to a clay would be measured in hundreds of years and possibly thousands. Hence, for the useful life of the project, it would seem reasonable to assume strength and permeability gradually reduced by weathering only in the outer part of the fill, and for the inner portion to assume a strength *perhaps only slightly lower than* that for a normal granular material. (As a word of caution, there are other types of shale for which this assumption would be unsound. Examples are those which slake severely, and the weaker clay shales which soften and swell when unloaded *in the presence of* water.) Possible economies from such approach are now being explored for two other dams in the Kansas City District involving similar shale, and the concept seems realizable with some selection to avoid the weaker shales, using such in berm fill.

TUTTLE CREEK DAM

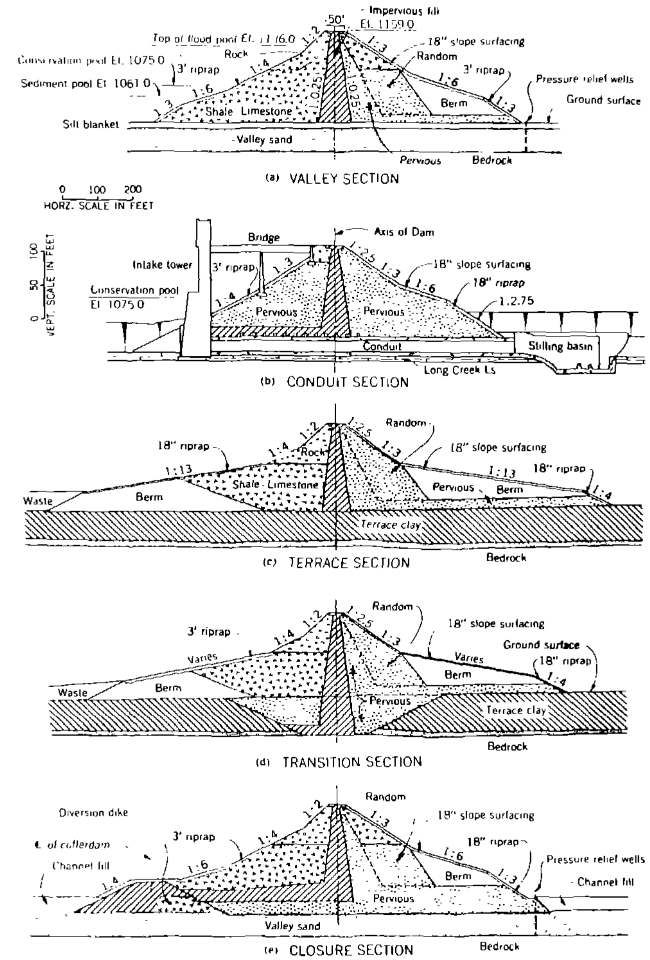


Figure 3. Typical embankment sections.

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Table !

	Project	Sample Description	Natural Water Content %	Maximum Size in.	Gradation: Passing Sieve, % No. 10	No. 40	No. 200	Classification: Uniform Soil Class Systems	AASHTO	Specific Gravity Solids	Atterberg Limits LL	PI	Standar… Mold Diameter in.	M Fa…
1.	Warm Springs Dam, CA Dry Creek, 6 miles northeast of Healdsburg	Fine grained black shale, brown sandy shale, and arkosic graywacke crushed from 6-in.-diameter cores (Franciscian Formation, Cretaceous Age)	--	3 1	41 62	26 42	17 30	GC SC	A-2-6 A-2-6	2.72	30	14	12	
				(Gradation used for tests)										
2.	Cachuma Dam, CA Santa Ynez River, 18 miles northwest of Santa Barbara (Gibbs et al, 1960)	Siltstone crushed from cores (Monterey Formation, Tertiary Age)	33	3/8	--	--	16	--	--	--	40 - 50	8 - 12		
3.	Bear Creek Lake, CO Bear Creek, 2 miles east of Morrison, near Denver	Clay shale, composite of cores crushed to (-) 1-in. (Denver Formation, Tertiary Age)	--	1	14	6	2	CH	A-2-7	2.78	68	42	6	
		Clay shale, composite of test pit samples (Arapahoe Formation, Tertiary Age)	--	3/4	50	14	4	CH	A-2-7	2.77	72	49	4	
4.	East Fork Lake, OH East Fork of Little Miami River, 4 miles southeast of Batavia	Crushed gray shaley limestone and calcareous shale (50% limestone, 50% shale) (Maysville Group, Ordovician Age)	2.1 (Shale 6.1)	3 3**	10 26	5 11	2 5	GW GW-GC	A-1a A-2-6**	2.73	30	14	12	
				3 3**	10 24	5 11	2 4	GW GW	A-1-a A-2-6**	2.73	30	14	--	
				3 3**	10 28	5 13	2 6	GW GW-GC	A-1-a A-2-6**	2.73	30	14	--	
				3 3**	10 25	5 11	2 4	GW GW	A-1-a A-2-6**	2.73	30	14	--	
				3 3**	10 26	5 14	2 7	GW GW-GC	A-1-a A-2-6**	2.73	30	14	--	
				3 3**	10 26	5 13	2 6	GW GW-GC	A-1-a A-2-6**	2.73	30	14	--	
5.	Carr Fork Lake, KY Carr Fork, Kentucky River 2 miles from Vicco	Crushed shale and sandstone (Pottsville Series, Pennsylvanian Age)	--	1 1**	18 31	9 20	4 11	GP GP-GM	A-1-a A-1-a**	2.73	--	--	6	1
6.	Fishtrap Lake, KY Levisa Fork, Big Sandy River, 7 miles southeast of Pikeville	Crushed sandstone and shale from NX cores	0.8 - 3.7	2 2**	10 13	6 10	3 6	GP GP-GM	A-1-a A-1-a**	2.73	--	--	6	1
			0.8 - 3.7	2 2**	10 21	6 13	3 7	GP-GM GP-GM	A-1-a A-1-a**	2.73	--	--	--	
		Crushed gray shale (Norton and Gladeville Formation, Pennsylvanian Age)	--	2 1**	25 35	14 40	3 12	GP GW-GM	A-1-a A-1-a**	2.67	--	--	--	
7.	Paintsville Lake, KY Paint Creek, Paintsville	Gray, silty clayey shale crushed from 6-in.-diameter cores (Breathitt Formation, Pennsylvanian Age)	--	3	15	13	11	GP-GM	A-1-a	2.74	28	6	12	3
8.	Red River Lake, KY Red River, 45 miles east of Lexington.	Gray and green-gray shaley limestone and shale crushed from 6-in.-diameter cores	3 -- 5	3 3**	9 24	5 12	3 7	GP GP-GC	A-2-6 A-2-6**	2.74	30	10	12	2
		Light gray clayey shale crushed from 6-in.-diameter cores. (Waverly Group, Pennsylvanian age)	5.3	3 3**	8 40	8 32	7 30	GW-GC GC	A-2-6 A-2-6**	2.77	32	12	12	2
9.	Taylorsville Lake, KY Chaplin River, 20 miles southeast of Taylorsville	Brown to green-gray weathered soft shale and gray interbedded limestone (68% shale, 32% limestone) crushed from 6-in.-diameter cores	11 -- 14	3	67	66	62	CH	A-7	2.77	41	18	12	2

* Deviator stress continued to increase during the test.
** After triaxial test.
\- Apparent cohesion neglected in S test; values are average for curved envelope.

Table 14. Summary of test data on compacted shale.

ific ity ids	Atterberg Limits LL	Atterberg Limits PI	Standard Effort Compaction: Mold Diameter in.	Standard Effort Compaction: Maximum Particle Size in.	Standard Effort Compaction: Optimum Water Content %	Standard Effort Compaction: Maximum Dry Density pcf	Permeability: Mold Diameter in.	Permeability: Dry Density pfc	Permeability: Permeability k cm/sec	Triaxial Shear Test Data: Sample Diameter and Height in.	Triaxial Shear Test Data: Maximum Particle Size in.	As Compacted: Water Content %	As Compacted: Dry Density pcf	As Compacted: Saturation %	As Tested: Saturation %	As Tested: Confining Consolidation Pressure tsf
72	30	14	12	3	10.5	129.5	--	--	--	6 by 14	1	11	121	75	100	3, 6, 12*
-	40 - 50	8 - 12					Standard Proctor Effort:			3.25 by 9	3/8	32	65	57	--	0.9, 1.8, 3.6, and 7.2
							Double Standard Proctor Effort:				3/8	32	69	63	--	
78	68	42	6	1	23.8	98.5	--	--	--	5.8 by 12	1	26	98	91	93	4, 8, 12*
											1	26	95	86	86	4, 8, 12*
77	72	49	4	3/4	25.3	96.5	--	--	--	1.4 by 3	3/4	21	94	69	87	4, 8, 12
												--		--	--	
											3/4	27	94	89	98	4, 8, 12
										1.4 by 3	3/4	28	93	91	100	4, 8, 12
											3/4	28	94	95	100	4, 8, 12
										3-in square	0.2	26	93	82	100	4, 8, 12
.73	30	14	12	3	2 Natural water cont	129	--	--	--	15 by 32	3	1.9	129.4	16.7	100	4, 8, 12*
.73	30	14	--	--	--	--	--	--	--	15 by 32	3	1.5	130.1	12.9	--	4, 8, 12*
.73	30	14	--	--	--	--	--	--	--	15 by 32	3	1.9	129.5	16.5	19.2	4, 8, 12
.73	30	14	--	--	--	--	--	--	--	15 by 32	3	2.3	129.0	19.6	25.5	4, 8, 12
.73	30	14	--	--	--	--	--	--	--	15 by 32	3	2.1	129.5	18.6	--	4, 8, 12
.73	30	14	--	--	--	--	--	--	--	15 by 32	3	1.6	130.0	13.4	100	4, 8, 12*
.73	--	--	6	1	10.7	128	6	124	5×10^{-2}	15 by 38	1	5.8	123.1	41.1	100	0.5, 1, 3, 6
.73	--	--	6	1.5	4.7	123	6	123	2×10^{-1}	12 by 28	2	4	120	25	--	1, 4, 6, 9
.73	--	--	--	--	--	--	--	--	--	12 by 28	2	4	120	26	--	4, 9
.67	--	--	--	--	--	--	--	--	--	6 by 15	2	0-3	128	0-29	--	0.5, 1, 5, 9
.74	28	6	12	3	7.8	135.6	--	--	--	15 by 32	3	7	129	59	100	2, 6, 13*
											3	7	129	58	100	
.74	30	10	12	2	4.2 Natural water cont	121	--	--	--	15 by 32	3	5	120	30	--	4, 8, 12*
											3	5	120	30	100	4, 8, 12*
.77	32	12	12	2	5.3 Natural water cont	124	--	--	--	15 by 32	3	6	123	40	--	4, 8, 12*
											3	5	124	35	--	4, 8, 12*
											3	5	124	35	100	4, 8, 12*
.77	41	18	12	2	11 Natural water cont	116	--	--	--	15 by 32	3	12	115	66	--	2, 6, 10.5*
											3	11	116	63	100	2, 6, 10.5*
											3	11	116	65	100	2, 6, 10.5*
											3	12	115	65	--	2, 6, 10.5*
											3	12	116	64	--	2, 6, 10.5*

(Continued)

B

Triaxial Shear Test Data											
As Compacted			As Tested		Shear Strength Permeability						
r nt	Dry Density pcf	Saturation %	Saturation %	Confining or Consolidation Pressure tsf	Q (UU) c tsf	Q (UU) φ deg	R (CU) c tsf	R (CU) φ deg	R̄ or S (CD) c tsf	R̄ or S (CD) φ deg	Design Strengths
	121	75	100	3, 6, 12*	--	--	0.0	19	--	--	Q: c = 2 tsf φ = 0 deg R: c = 0.0 tsf φ = 16 deg S: c' = 0.0 tsf φ' = 30 deg Q and S from tests on undisturbed foundation samples
	65	57	--	0.9, 1.8, 3.6, and 7.2	--	--	0.5	36.4	0.56	38	
	69	63	--		--	--	0.8	36.2	0.86	37.2	
	98	91	93	4, 8, 12*	0.8	23	--	--	--	--	Q: c = 0.8 tsf φ = 2.3 deg R: c = 0.25 tsf φ = 15.5 deg S: c' = 0.0 tsf φ' = 23.2 deg
	95	86	86	4, 8, 12*	--	--	0.15	18.2	0.0	31.5	
	94	69	87	4, 8, 12	0.4	0.0	--	--	--	--	
		--	--		--	--					
	94	89	98	4, 8, 12	2.0	0.8	--	--	--	--	
3	93	91	100	4, 8, 12	--	--	0.33	16.0			
3	94	95	100	4, 8, 12	--	--	0.28	14.2			
5	93	82	100	4, 8, 12			Direct Shear:		0.5	25	
1.9	129.4	16.7	100	4, 8, 12*	--	--	1.0†	14†	0.7†	36†	Q: c = 2.0 tsf φ = 17 deg R: c = 1.5 tsf φ = 28 deg S: c' = 0.0 tsf φ' = 40 deg
1.5	130.1	12.9	--	4, 8, 12*	1.40	38.5	--	--	--	--	
1.9	129.5	16.5	19.2	4, 8, 12	--	--	1.9	36	--	--	
2.3	129.0	19.6	25.5	4, 8, 12	--	--	1.6	35.5	--	--	
2.1	129.5	18.6	--	4, 8, 12	1.30	36.5	--	--	--	--	
1.6	130.0	13.4	100	4, 8, 12*	--	--	1.5†	11.5†	0.7†	38†	
5.8	123.1	41.1	100	0.5, 1, 3, 6	--	--	1.2	16	--	--	R: c = 1.2 tsf φ = 16 deg S: c' = 0 tsf φ' = 33 deg
4	120	25	--	1, 4, 6, 9	---	--	1.15	10.7	0.5	35	
4	120	26	--	4, 9	--	--	--	--	0.5	36	
0-3	128	0-29	--	0.5, 1, 5, 9	--	--	1.0	18.5	0.5	36	
7	129	59	100	2, 6, 13*	--	--	1.0	18.5	0.0	37†	R: c = 0.9 tsf φ = 16.6 deg S: c' = 0.0 tsf φ' = 29.6 deg
7	129	58	100						0.0	33†	
5	120	30	--	4, 8, 12*	0.5	35	--	--	--	--	R: c = 0.5 tsf φ = 19.8 deg S: c' = 0.0 tsf φ' = 33 deg
5	120	30	100	4, 8, 12*	--	--	0.4	16	0.6	36	
6	123	40	--	4, 8, 12*	1.5	28.5	--	--	--	--	
5	124	35	--	4, 8, 12*	0.5	34.5	--	--	--	--	
5	124	35	100	4, 8, 12*	--	--	0.25	18	0.0	39.5	
12	115	66	--	2, 6, 10.5*	1.4	15	1.2	20	--	--	Not yet available
11	116	63	100	2, 6, 10.5*	--	--	0.0	14	0.3	35	
11	116	65	100	2, 6, 10.5*	--	--	0.3	10.5	0.0	40	
12	115	65	--	2, 6, 10.5*	1.25	18	--	--	--	--	
12	116	64	--	2, 6, 10.5*	--	--	1.1	21	--	--	

Project	Sample Description	Natural Water Content %	Maximum Size in.	Gradation: Passing Sieve, % No. 10	No. 40	No. 200	Classification: Uniform Soil Class Systems	AASHTO	Specific Gravity Solids	Atterberg Limits LL	PI	Standar[d] Mold Diameter in.	M P
9. Taylorsville Lake, KY (Continued)	Gray interbedded moderately hard to moderately soft calcareous shale and hard fossiliferous limestone crushed from 6-in.-diameter cores	-- -- --	3 3** 3**	12 38 27	9 27 18	8 26 16	GP-GC GC GC	A-2-6 A-2-6** A-2-6**	2.76	38 32	17 15	12	
10. Yatesville Lake, KY Blaine Creek, 4 miles south of Louisa	Red to gray clay shale, indurated clay and siltstone crushed from 6-in.-diameter cores (Allegheny Series, Pennsylvanian Age)	4.2 -- 5.4	3 3**	14 36	12 21	9 13	GP-GC GC	A-1-a A-2-6**	2.75	37	15	12	
11. Beech Fork Lake, WV Beech Fork, Twelve Pole Creek, 6.5 miles south of Huntington	Indurated clay crushed from 6-in.-diameter cores (Conemaugh Series, Pennsylvanian Age)	12.7	1	26	9	3	GW-GC	A-2-6	2.82	32	13	5.8	
12. R. D. Bailey Lake, WV Guyandot River, Justice	Brown clay shale crushed from 6-in.-diameter core	3.8	3 3**	11 33	3 18	1 13	GW GM-GC	A-1-a A-1-b**	2.76	24	5	--	
	Gray clay shale crushed from 6-in.-diameter core	3.1	3 3**	12 33	6 18	3 10	GP GM-GC	A-1-a A-1-b**	2.77	26	4	--	
	Brown weathered shale and claystone from test fill (Kanawha Formation, Pennsylvanian Age)	7.3 -- 10.3	12 2**	40 50	28 38	22 36	GM GM	A-2-4 A-4**	2.76	32	8	12	:
13. Beltzville Dam, PA Pohopoco Creek River, 4 miles east of Lehighton	Weathered shale from test pit, crushed	13.8	1 1**	25 38	18 31	16 26	GM GM	A-1-a A-2-4**	2.72	38	8	6	1
	Brown broken weathered shale from test fill (all materials from Trimmers Rock Formation, Devonian Age)	14 -- 15	1 1	23 49	17 36	14 27	GM GM	A-1-a A-2-4**	2.73	34	10	6	1
	Gray silty weathered shale from test fill	6 -- 10	> 6 3**	17 29	12 20	9 14	GP-GM GM	A-1-a A-2-4**	2.75	32	8	--	-
	Partly weathered shale from test fill	7.6	2 2**	17 25	10 19	7 17	GP-GC GM	A-1-a A-2-4**	2.76	30	8	12	2
		7.6	3 3**	20 24	15 17	14 15	GM GM	A-1-a	2.76	31	8	--	-
14. Raystown Dam, PA Raystown Branch, Juniata River, 3 miles southeast of Huntingdon	Mixture of siltstone, shale, and sandstone from test quarry plus 3 in. crushed	1.8	2 2**	5 17	3 9	2 7	GP GP-GC	A-1-a A-2-4**	2.70	31	9	12	2
	Mixtures of shale, siltstone, and sandstone from test fills plus 3 in. crushed (Catskill Formation, Devonian Age)	-- 3.5	26 3** 15 3**	4 20 15 28	3 11 11 17	2 9 8 14	GP-GM GP-GC GP-GC GC	A-1-a A-2-4** A-1-a A-2-4**	2.79 2.78 2.77	21 27 25	3 10 8	12 12 12	3 3 3
15. Trexler Lake, PA Jordan Creek, 12 miles northwest of Allentown	Brown-gray broken shale from test pit	--	6	10	5	3	GW	A-2-4	2.69	30	7	12	2
	Brown-gray broken clayey shale from test pit	--	3 3**	22 25	14 16	9 10	GP-GC GP-GM	A-2-4 A-2-4**	2.71	30	7	--	--
	Brown-gray broken silty clayey shale from test pit (Martinsburg Formation, Ordovician Age)	--	3 3**	25 26	17 14	9 10	GP-GM GP-GM	A-2-4 A-2-4**	2.70	25	4	--	--

* Deviator stress continued to increase during the test.
** After triaxial test.
† Apparent cohesion neglected in S test; values are average for curved envelope.

Table 14. Summary of test data on compacted shale. (concluded)

Standard Effort Compaction				Permeability			Triaxial Shear Test Data								
												As Tested		Shear S	
									As Compacted					Q (UU)	
Mold Diameter in.	Maximum Particle Size in.	Optimum Water Content %	Maximum Dry Density pcf	Mold Diameter in.	Dry Density pfc	Permeability k cm/sec	Sample Diameter and Height in.	Maximum Particle Size in.	Water Content %	Dry Density pcf	Saturation %	Saturation %	Confining or Consolidation Pressure tsf	c tsf	ø deg
12	2	2.5 Natural water cont	127	--	--	--	15 by 32	3	2.4	127	18	100	2, 6, 10.5*	--	--
								3	2.5	127	19	100	2, 6, 10.5*	--	--
								3	2.7	127	21	--	2, 6, 10.5	1.5	36
12	3	10.3	130	14	126	1×10^{-3}	15 by 32	3	4	128	35	100	2, 5, 8*	--	--
								3	5	127	41	--	2, 5, 8	0.75	31.5
5.8	1	11	128	3	128	1×10^{-3}	2.8 by 5.6	1	9.3	123	60	--	1.4, 2.9, and 4.3	1.28	17.6
								1	11.1	123	73	--	1.4, 2.9, and 4.3	0.98	14.5
								1	13.0	123	86	--	1.4, 2.9, and 4.3	0.68	11.3
							2.8 by 5.6	1	9.0	123	59	100	1.4, 2.9, and 4.3	--	--
								1	11.1	123	73	100	1.4, 2.9, and 4.3*	--	--
--	--	--	--	--	--	--	15 by 32	3	2.5	113	12	100	5, 10, 20	--	--
								3	2.3	114	13	100	5, 10, 20	--	--
--	--	--	--	--	--	--	15 by 32	2	1.1	115	6	100	5, 10, 20	--	--
								2	1.1	115	6	100	5, 10, 20	--	--
12	2	9.8	Field	12	124	2×10^{-3}	15 by 32	2	8.5	125	62	100	2, 7, 10*	--	--
								2	8.0	126	58	65	2, 7, 10	--	--
6	1	15	116.8	--	--	--	6 by 15	1	17.0	114	94	--	1, 3.5, 6, 8*	0.8	0.0
								1	15.1	117	90	--	1, 3.5, 6, 8*	2.6	0.0
								1	17.0	114	100	--	1, 3.5, 5.7	--	--
6	1	11.6	125	--	--	--	6 by 15	1	14.5	117	85	--	3.5, 8, 14*	2.5	1
								1	14.3	119	90	--	3.5, 8, 14*	1.9	4
								1	3.8	119	25	--	3.5, 8, 14	2.0	32
								1	11.1	120	71	--	3.5, 8, 14*	2.0	22.5
								1	14.2	121	94	--	3.5, 8, 14*	2.4	6
								1	14.0	120	89	100	3.5, 8, 14	--	--
								1	14.7	116	86	100	3.5, 8, 14*	--	--
								1	4.2	119	26	100	3.5, 8, 14	--	--
								1	12.2	118	76	100	3.5, 8, 14	--	--
--	--	--	--	--	--	--	6 by 15	3	8.3	121	55	98	2, 5, 10, 15*	--	--
								3	7.2	126	54	99	2, 5, 10, 15*	--	--
								3	8.8	130	76	100	2, 5, 10, 15*	--	--
								3	7.9	131	71	100	2, 5, 10, 15	--	--
								3	5.8	132	54	100	2, 5, 10, 15*	--	--
12	2	10.5	129.3	--	--	--	12 by 28	2	7.1	126	54	99	2, 5, 10, 15*	--	--
								2	9.1	126	68	97	2, 5, 10, 15	--	--
								2	8.8	130	76	100	2, 5, 10, 15*	--	--
--	--	--	--	--	--	--	12 by 28	3	8.4	121	55	99	2, 5, 10, 15*	--	--
12	2	2.4	124.7	--	--	--	12 by 28	2	3.0	124	21	100	2, 6, 6	--	--
								2	3.1	116	17	100	2, 6, 6*	--	--
12	3	7.5	136.4	18	125	1	12 by 28	3	6.9	128	50	92	2, 2, 6, 10	--	--
								3	7.3	136	72	99	2, 6, 10	--	--
12	3	7.5	142.6	18	140	2×10^{-3}	12 by 28	3	7.0	136	71	100	2, 6, 10	--	--
12	3	7.5	139.6					3	7.1	142	91	99	2, 6, 10	--	--
12	2	7	124	--	--	--	15 by 32	3	2.8	114	16	100	1, 4, 8*	--	--
--	--	--	--	14	116	5×10^{-3}	15 by 32	3	2.1	115	13	100	1.7, 4, 8*	--	--
				14	118	6×10^{-3}									
				14	122	4×10^{-3}									
--	--	--	--	--	--	--	15 by 32	3	2.5	122	17	100	1, 4, 8*	--	--

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Water Content %	Dry Density pcf	Saturation %	As Tested Saturation %	Confining or Consolidation Pressure tsf	Q (UU) c tsf	Q (UU) φ deg	R (CU) c tsf	R (CU) φ deg	R̄ or S (CD) c tsf	R̄ or S (CD) φ deg	Design Strengths
2.4	127	18	100	2, 6, 10.5*	--	--	0.25	16.5	0.25	37†	
2.5	127	19	100	2, 6, 10.5*	--	--	0.25	15.5	0.0	40†	
2.7	127	21	--	2, 6, 10.5	1.5	36					
4	128	35	100	2, 5, 8*	--	--	0.25	20	0.0	34.5†	Q: c = 0.75 tsf
5	127	41	--	2, 5, 8	0.75	31.5					φ = 31.5 deg R: c = 0.25 tsf φ = 20 deg S: c' = 0.0 tsf φ' = 25 deg
9.3	123	60	--	1.4, 2.9, and 4.3	1.28	17.6	--	--	--	--	Q: c = 0.8 tsf φ = 9 deg
1.1	123	73	--	1.4, 2.9, and 4.3	0.98	14.5	--	--	--	--	R: c = 0.17 tsf
3.0	123	86	--	1.4, 2.9, and 4.3	0.68	11.3	--	--	--	--	φ = 19 deg
9.0	123	59	100	1.4, 2.9, and 4.3	--	--	0.17	16.8	--	--	S: c' = 0.0 tsf φ' = 25 deg
1.1	123	73	100	1.4, 2.9, and 4.3*	--	--	0.21	16.7	--	--	
2.5	113	12	100	5, 10, 20	--	--	0.0	16	0.0	35†	R: c = 0.0 tsf
2.3	114	13	100	5, 10, 20	--	--	--	--	0.0	32†	φ = 16 deg
1.1	115	6	100	5, 10, 20	--	--	0.0	16	0.0	39	S: c' · 0
1.1	115	6	100	5, 10, 20	--	--	--	--	0.0	31	φ' = 32 deg
8.5	125	62	100	2, 7, 10*	--	--	0.5	18	0.0	35	
8.0	126	58	65	2, 7, 10	--	--	--	--	0.0	31.5	
7.0	114	94	--	1, 3.5, 6, 8*	0.8	0.0	--	--	--	--	
5.1	117	90	--	1, 3.5, 6, 8*	2.6	0.0	--	--	--	--	
7.0	114	100	--	1, 3.5, 5.7	--	--	0.0	24	0.0†	35†	Weathered silty shale:
4.5	117	85	--	3.5, 8, 14*	2.5	1	--	--	--	--	Q: c = 1.9 tsf
4.3	119	90	--	3.5, 8, 14*	1.9	4	--	--	--	--	φ = 4 deg
3.8	119	25	--	3.5, 8, 14	2.0	32	--	--	--	--	R: c = 0.3 tsf
1.1	120	71	--	3.5, 8, 14*	2.0	22.5	--	--	--	--	φ = 19.2 deg
4.2	121	94	--	3.5, 8, 14*	2.4	6	--	--	--	--	S: c' = 0.0 tsf
4.0	120	89	100	3.5, 8, 14	--	--	0.0	22	0.0	37	φ' = 32.1 deg
4.7	116	86	100	3.5, 8, 14*	--	--	0.7	19.5	0.0	36	
4.2	119	26	100	3.5, 8, 14	--	--	0.6	25	0.0	30.5	
2.2	118	76	100	3.5, 8, 14	--	--	2.0	20	0.0	36	
8.3	121	55	98	2, 5, 10, 15*	--	--	1.5	10			Partly weathered
7.2	126	54	99	2, 5, 10, 15*	--	--	1.5	13			shale:
8.8	130	76	100	2, 5, 10, 15*	--	--	2.3	19.5			R: c = 0.0 tsf
7.9	131	71	100	2, 5, 10, 15	--	--	0.0	25.5	0.0	38	φ = 25.5 deg
5.8	132	54	100	2, 5, 10, 15*	--	--	0.4	28	0.0	38.5	
7.1	126	54	99	2, 5, 10, 15*	--	--	1.5	13	0.0†	40†	
9.1	126	68	97	2, 5, 10, 15	--	--	--	--	0.0†	35†	
8.8	130	76	100	2, 5, 10, 15*	--	--	3.3	14	0.0†	43†	
8.4	121	55	99	2, 5, 10, 15*	--	--	1.5	10	0.0†	42†	
3.0	124	21	100	2, 6, 6	--	--	--	--	0.0	37	S: c' = 0
3.1	116	17	100	2, 6, 6*	--	--	--	--	0.0	34	φ' = 45 deg for $\bar{\sigma}_n < 3.6$ tsf
6.9	128	50	92	2, 2, 6, 10	--	--	--	--	0.0	39	φ' = 34 deg
7.3	136	72	99	2, 6, 10	--	--	--	--	0.0	35	for $\bar{\sigma}_n > 3.6$ tsf
7.0	136	71	100	2, 6, 10	--	--	--	--	0.0	33	
7.1	142	91	99	2, 6, 10	--	--	--	--	0.0	35	
2.8	114	16	100	1, 4, 8*	--	--	1.15	16	0.65†	43.5†	R: c = 0.65 tsf φ = 19 deg
2.1	115	13	100	1.7, 4, 8*	--	--	0.65	19	0.65†	45†	S: c' = 0 φ' = 40 deg
2.5	122	17	100	1, 4, 8*	--	--	0.80	20	0.60†	45†	

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