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NCG TECHNICAL REPORT No. 15

EMPIRICAL STUDY OF BEHAVIOR OF CLAY SHALE SLOPES

VOLUME I



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LIVERMORE, CALIFORNIA

DECEMBER 1970

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EMPIRICAL STUDY OF BEHAVIOR OF CLAY SHALE SLOPES

VOLUME 1

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Sponsored by

U.S. Army Engineer Nuclear Cratering Group
Livermore, California

December 1970

Abstract

This study was undertaken to determine the factors that lead to instability in clay shale, and thus to provide a basis for assessing the probable long-term stability of high crater slopes in clay shales.

The chief features contributing to the engineering behavior of clay shales are degree of overconsolidation and lithology, both of which reflect geologic history. Local geologic structure and hydrologic conditions affect individual slopes. Weak layers may present slope hazards, while conversely a few stronger layers may materially strengthen an entire clay shale slope. Time-dependent phenomena are important in clay shale slopes, which may fail after standing apparently stable for many years.

Intensive studies were made of natural slopes in five clay shale units in the upper Missouri Basin. The Claggett, Bearpaw, and Pierre formations, all marine-deposited shales of Late Cretaceous age, showed extensive slope failures; high slopes generally stood at overall inclinations of only 5 to 10 deg, inclinations comparable to the residual angles of internal friction of the materials. The Cretaceous Colorado group and Lower Tertiary Fort Union group showed steeper slopes with fewer failures. Intensive laboratory testing of the physical properties of all materials indicated that only gross differences in slope behavior can be related to any specific test or series of tests.

Engineering practice in clay shale materials is customarily based on an empirical approach, attention being given to local site conditions and to the observed behavior of existing nearby slopes. Observations of experimental crater slopes in clay shale are limited. The conclusion is reached that future design crater slopes must be based on experiences with conventionally excavated slopes. Furthermore, the design of cratered slopes must be conservative, although no experience indicates that the slope inclination must be as flat as the residual angle of internal friction of the site materials.

Foreword

The U. S. Army Engineer Nuclear Cratering Group (NCG) sponsored several coordinated study phases, under the project "Empirical Study of Behavior of Clay Shale Slopes," to assemble and to analyze data which related to the behavior of clay shale slopes. The study was conducted under the overall technical direction of NCG. Major participants in the study, in addition to NCG, were the U. S. Army Engineer Waterways Experiment Station (WES), the U. S. Army Engineer Missouri River Division (MRD), and the U. S. Army Engineer District, Omaha (MRO).

The U. S. Geological Survey (USGS), Engineering Geology Branch, Denver, provided important assistance for several phases of the study. In addition, several individuals and private, local, state, and federal agencies generously contributed time and information to the major participants during the conduct of the study phases; these individuals and agencies are acknowledged within the report.

Messrs. R. W. Fleming, NCG, G. S. Spencer, MRD, and D. C. Banks, WES, were the principal investigators in the study and were responsible for overall coordination and review of the various study phases. This volume contains a review of the literature, a discussion and interpretation of the data assembled, and an assessment of the important characteristics of clay shale. Appendixes prepared by members of the participating agencies are contained in Volume 2.

Valuable guidance in the planning and conduct of the study was provided by Messrs. L. B. Underwood, Division Geologist, MRD, S. J. Johnson, Special Assistant, Soils Division, WES, W. C. Sherman, Jr., Chief, Soil and Rock Mechanics Branch, WES, R. R. W. Beene, Soil Mechanics Branch, Engineering Division, Directorate of Civil Works, Office of the Chief of Engineers, and P. R. Fisher, Chief, Earth Sciences Division, NCG. The final report was revised by Mr. D. C. Banks, WES, in coordination with Mr. C. C. McAneny, Engineering Geology Division, NCG.

Directors of NCG during the conduct of the study and preparation of the report were LTC Bernard C. Hughes and COL William E. Vandenberg. Technical Director of NCG was Mr. Walter C. Day.

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NCG TECHNICAL REPORT NO. 15

EMPIRICAL STUDY OF BEHAVIOR OF

CLAY SHALE SLOPES

Section 1. Introduction

GENERAL

The U.S. Army Corps of Engineers and the U.S. Atomic Energy Commission are engaged in a joint research program to develop the basic technology necessary to apply nuclear detonations to major construction projects. The U.S. Army Engineer Nuclear Cratering Group (NCG) has responsibility for the technical direction of the Corps' effort in this area. The assessment of the long-term stability of crater slopes is one aspect of the overall research and development program. Studies have been sponsored by NCG in which empirical data related to rock slope stability were collected (Kley and Lutton, 1967; Lutton, 1969); other studies have evaluated methods available for making stability calculations for crater slopes (Banks, 1968; Banks and Palmerton, 1968). One study reviewed available information on conventionally excavated slopes in weak shales (Hirschfeld, Whitman, and Wolfskill, 1965). Several high explosive (HE) experiments conducted by NCG in clay shales have produced small craters (Fisher, Kley, and Frandsen, 1968; Fisher, Kley, and Jack, 1969). Studies have been made which summarized the engineering properties of craters (Fisher, 1968). To date the studies have not dealt

with an evaluation of the anticipated behavior of high crater slopes produced in clay shale formations. Feasibility studies currently being conducted by NCG are vitally concerned with the potential long-term stability of high crater slopes in clay shales.

PURPOSE

The purposes of this study were to define characteristics which typify a potentially troublesome clay shale, to determine the factors which lead to instability, and to provide a basis for the assessment of the long-term stability of high crater slopes in clay shales.

SCOPE

This study consisted of field and office investigations. The field program concentrated on slopes along the upper Missouri River and was divided into three parts. First, five different geologic formations along the upper Missouri River were selected for mapping. Engineering geologists from the U.S. Geological Survey (USGS) led an initial field reconnaissance to select particular slopes for study. Emphasis was placed on selecting the highest and steepest that could be

found in each geologic unit. The slopes were mapped and one slope in each unit or a total of five slopes were selected for more detailed investigation. (Detailed results of this study phase are contained in Volume 2, Appendix A.) Second, borings were made in each of the five slopes. One boring was drilled at a site in each of the Colorado and Fort Union groups. Two borings were drilled at a site in each of the Claggett, Bearpaw, and Pierre formations. At the latter three sites, one boring was placed near the backscarp of the slide and extended to a considerable depth to sample unfailed intact clay shale; the second boring was placed midway down the slope and encountered failed material throughout. (Detailed results of this study, including Atterberg limits and natural water contents, are contained in Volume 2, Appendix B.) Third, approximately 2 ft of every 10 ft of core recovered were preserved for extensive testing by the Corps of Engineers Missouri River Division Laboratory. (Test procedures and results, with a discussion of their significance, are contained in Volume 2, Appendix C.)

In addition, two office studies were conducted. One study developed case histories of dam construction experience at five projects. Projects investigated were Fort Peck Dam (constructed on the Bearpaw formation), Garrison Dam (con-

structed on the Tongue River formation of the Fort Union group), Oahe Dam (constructed on the Pierre formation), Gardiner Dam (constructed on the Bearpaw formation), and Chatfield Dam (presently under construction on the Dawson formation). (Case histories are contained in Volume 2, Appendix D.) A second office study developed a summary of slope experiences and design practices in clay shales from other geographic areas. (Results of this study phase are contained in Volume 2, Appendix E.)

ORGANIZATION OF REPORT

The results of the various study phases are presented as five appendixes in Volume 2 of this report. Pertinent information from Volume 2 is presented and discussed in this volume. Section 1 of this volume is an introduction. Section 2 is a disucssion of factors that affect clay shale slope behavior. Information presented was obtained principally from the published literature. Section 3 is a summary and discussion of data obtained from the field investigation of slopes along the upper Missouri River. Section 4 presents an assessment of (1) the important characteristics of clay shale and (2) the potential long-term stability of high crater slopes in clay shale. Section 5 summarizes the major findings and conclusions of the study.

Section 2. Factors That Affect Clay Shale Slope Behavior

INTRODUCTION

General

While it is true that problems in clay shale have been encountered for many

years, only in the past decade or so have the problems been studied extensively. Considerable data are available which pertain to specific characteristics of slope behavior in clay shales. In this

country, the Corps of Engineers has accumulated considerable experience with clay shale behavior over the past thirty years beginning with the construction of Fort Peck Dam on Bearpaw shale (Middlebrooks, 1942). More recent experiences in the construction of Garrison Dam (Lane, 1961; Smith and Redlinger, 1953), Oahe Dam (Knight, 1963; Underwood, Thorfinnson, and Black, 1964), and Waco Dam (Beene, 1967) have added to the understanding of the behavior of clay shales. In Canada, papers by Peterson (1958), Peterson, Jaspar, Rivard, and Iverson (1960), Hardy (1957), Ringheim (1964), Scott and Brooker (1968), and Sinclair and Brooker (1967) summarize experiences with clay shales in Saskatchewan and Alberta. European investigators, notably Skempton (1964) and Bjerrum (1967), have studied the time-dependent behavior and mechanism of failure of overconsolidated clay and clay shale slopes. Several technical notes were presented and formed the basis of discussion at the Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City (Johnson, 1969). These papers and studies provided examples of problems in clay shales, presented material characteristics and geologic settings of the various materials, and illustrated analytical or empirical analyses of slope failures.

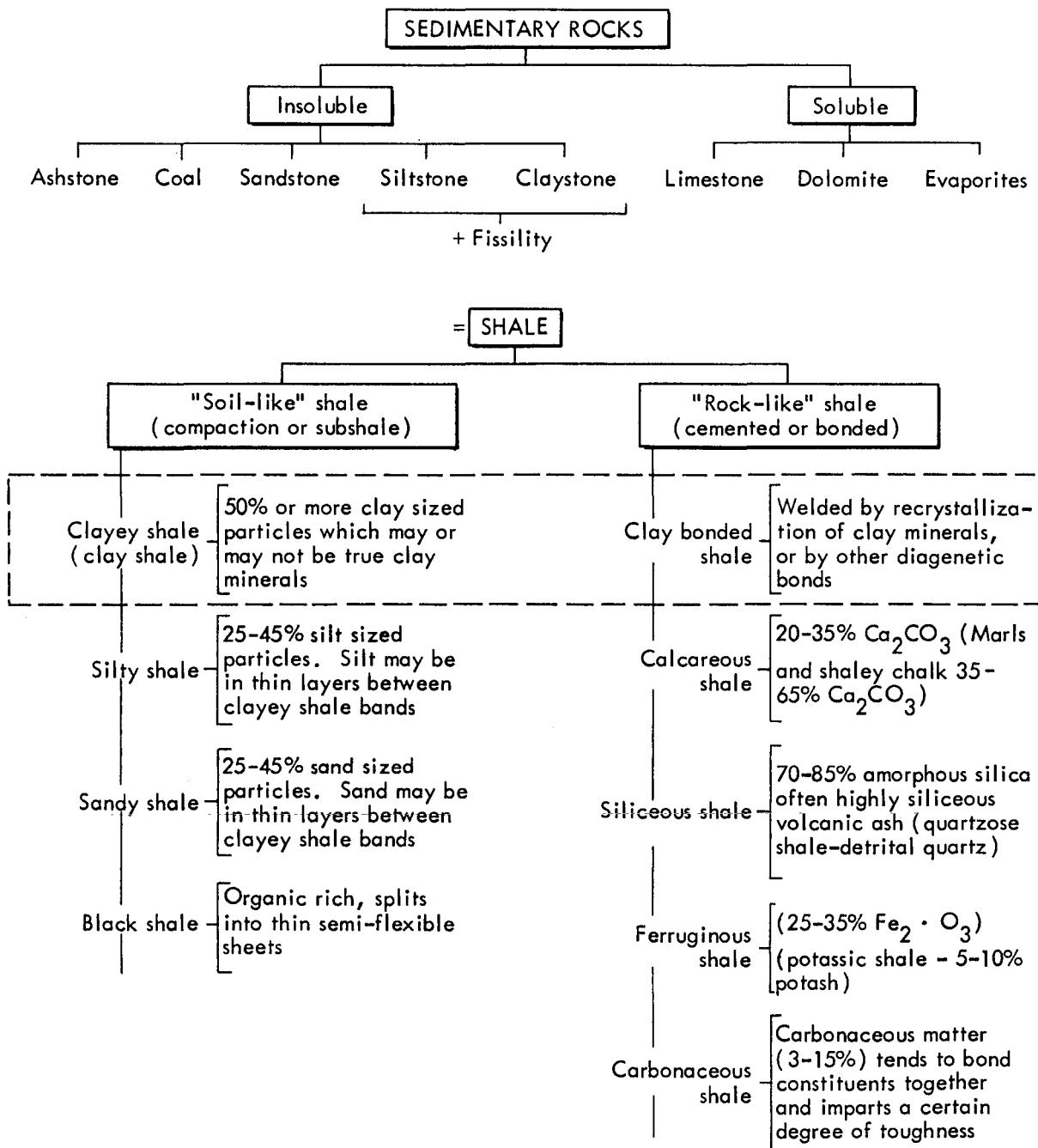
Definition of Clay Shale

Numerous attempts have been made to classify and identify clay shale. At least part of the problem is the inability to ascribe the material to either soil or rock. Terzaghi (1936) suggested that clays be grouped into three categories: (1) soft, intact clays free from joints and fissures,

(2) stiff, intact clays free from joints and fissures, and (3) stiff, fissured clays. Clay shales fall in the last category. Underwood (1967) attempted to classify and identify shales combining the genetic and compositional classifications of Philbrick (1950) and Mead (1936) as shown in Fig. 1. The subdivision of shales in Fig. 1 indicates two major groups, "soil-like" shale and "rock-like" shale, both containing descriptions based on composition with the exception of clay-bonded shale. Underwood emphasized that the geologic classification is not precise enough for engineering purposes and developed an evaluation system based on the engineering properties shown in Table 1.

Bjerrum (1967), preferring not to present a formal classification, referred in general to "overconsolidated plastic clays and clay shales," but emphasized a distinction between weak and strong bonding within the material (much like the distinction made in Fig. 1). Other terms frequently found in the literature, such as stiff, stiff-intact, firm, fissile, brittle, stiff-fissured or compacted clay and clay shale, generally refer to the same types of materials as those described by Bjerrum. The term "clay shale" has been widely adopted but has not achieved a precise and universal meaning; instead, it is becoming increasingly nebulous. Several attendees at the Specialty Session No. 10, Mexico City, discussed the meaning of the term (Johnson, 1969). The only characteristic common to all definitions advanced was the high degree of overconsolidation.

In this report the term clay shale refers to materials of sedimentary origin composed largely of silt- and clay-sized



(After Underwood, 1967)

Fig. 1. A geologic classification of shales.

particles, which may or may not be slightly cemented by foreign agents, such as iron oxide, calcite, or silica, and which have been subjected to consolidation loads greatly in excess of their present overburden loads. The material is com-

posed principally of clay minerals and pieces of intact material tending to slake when exposed to cyclic wetting and drying. Materials cemented to the extent that they do not slake when exposed to cyclic wetting and drying are termed siltstone or

Table 1. An engineering evaluation of shales^a

Laboratory tests and <u>in-situ</u> observations	Physical properties		Probable <u>in-situ</u> behavior					
	Average range of values		High pore pres- sure	Low bearing capacity	Tendency to rebound	Slope stability problems	Rapid slaking	Rapid erosion
Unfavorable	Favorable							
Compressive strength, psi	50 to 300		✓	✓				
	300-5000							
Modulus of elasticity, psi	20,000 to 200,000			✓				✓
	200,000 to 2×10^6							
Cohesive strength, psi	5 to 100				✓	✓		✓
	100 to > 1500							
Angle of internal friction, deg	10 to 20				✓	✓		✓
	20 to 65							
Dry density, lb/ft ³	70 to 110		✓					✓ (?)
	110 to 160							
Potential swell, %	3 to 15			✓	✓	✓	✓	✓
	1 to 3							
Natural moisture content, %	20 to 35		✓			✓		
	5-15							
Coefficient of permeability, cm/sec	10^{-5} to 10^{-10}		✓			✓	✓	
	$> 10^{-5}$							
Predominant clay minerals	Montmoril- lonite or illite		✓			✓		
	Kaolinite & chlorite							
Activity ratio = plasticity index clay content	0.75 to >2.0				✓			
	0.35 to 0.75							
Wetting and drying cycles	Reduces to grain sizes					✓	✓	
	Reduces to flakes							
Spacing of rock defects	Closely spaced		✓		✓		✓ (?)	✓
	Widely spaced							
Orientation of rock defects	Adversely oriented		✓		✓			✓
	Favorably oriented							
State of stress	> Existing over- burden load			✓	✓			✓
	≥ Over- burden load							

^a(Underwood, 1967)

claystone, depending on the particle gradation.

EFFECT OF PHYSICAL CHARACTERISTICS OF CLAY SHALES ON SLOPE BEHAVIOR

Two fundamental factors which influence the properties and behavior of clay shale, such as strength, swelling characteristics, slaking behavior, internal structure, and mode of failure, are (1) the degree of overconsolidation and (2) the lithology.

The degree of overconsolidation is related to the geologic history of the clay shales; the lithology is related to the origin of the sediments, to the transportation agencies involved, and to some extent to the environment of deposition, i.e., also to the geologic history. Thus, while it may be possible to obtain measurements of the overconsolidation of the clay shale and its composition, the role of the geologist in determining the geologic history of

a site or formation is fundamental to a study of clay shales.

Development of Clay Shale

Consolidation is the gradual process which involves a decrease in the volume of voids, and thus an increase in density, under the action of loads; in saturated materials the process simultaneously involves the expulsion of water from the voids. Overconsolidation refers to a condition where the existing overburden or load is less than the load at which the material was consolidated. In a geologic sense, the consolidation history of some clay shales is an extremely complex phenomenon caused by advances and retreats of glaciers, deposition and erosion of sediments, and wetting and drying cycles experienced by the materials.

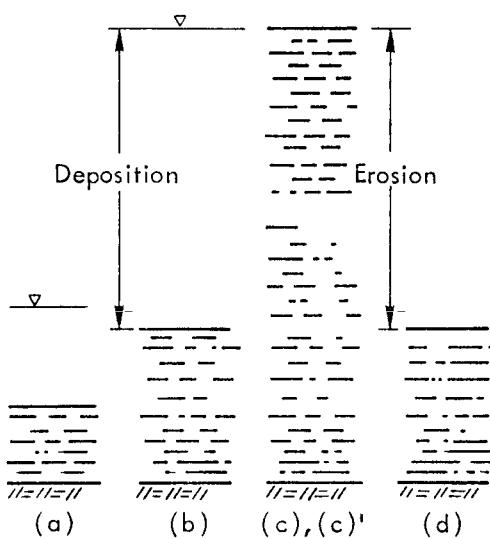
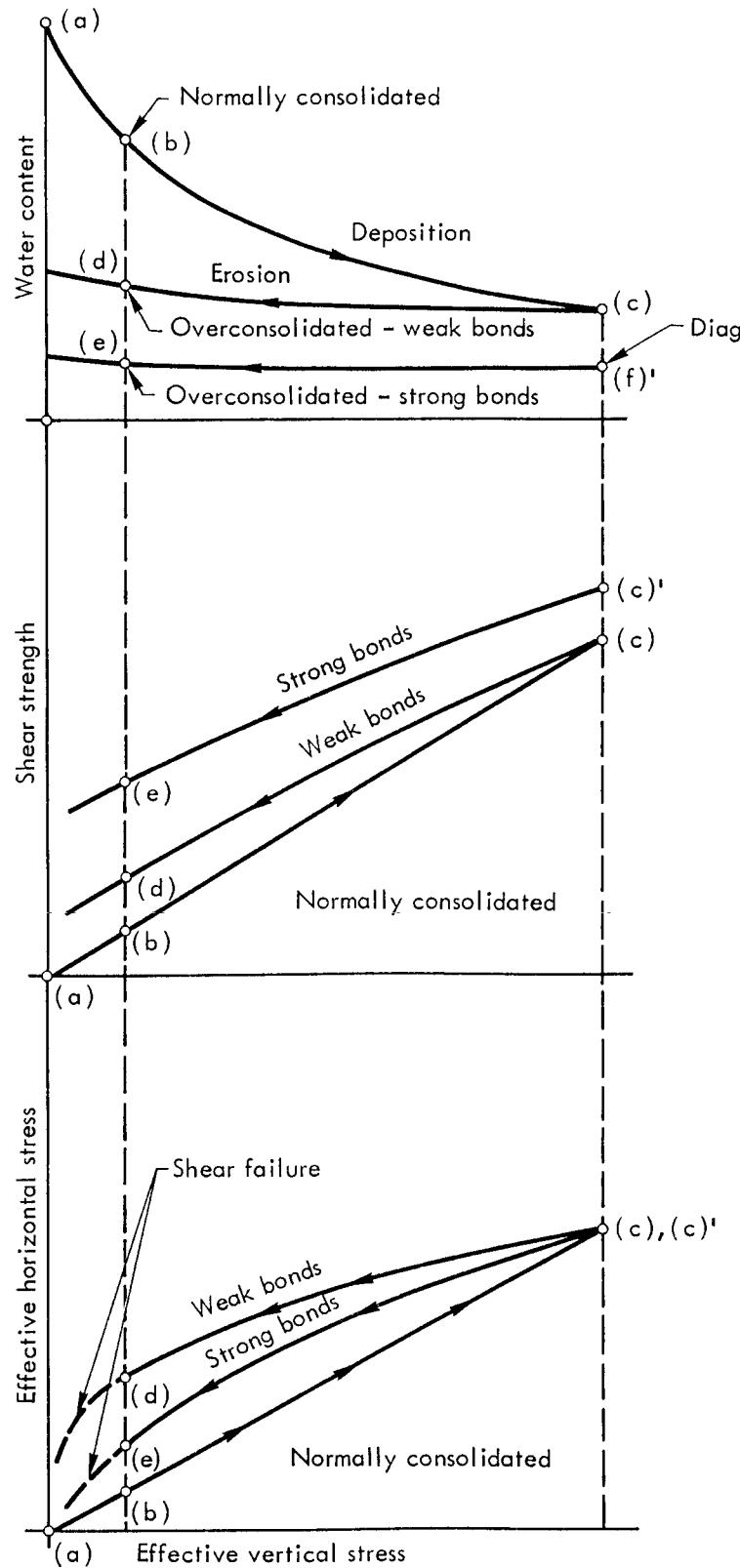
Skempton (1964) indicated the historical development of an overconsolidated clay shale. In the upper part of Fig. 2, point (a) schematically represents a clay immediately after deposition. The deposition of more clay causes an increase in the effective stress and a decrease in water content. At a stage represented by point (b) the clay is "normally-consolidated," since it has not been subjected to a load greater than that caused by the existing overburden. The shear strength of normally consolidated clay is proportional to the existing overburden as indicated by point (c) in Fig. 2. After consolidation under some large overburden load, point (c), the clay is unloaded by some process such as erosion of sediments and is left in an "overconsolidated" state represented in Fig. 2 by point (d).

The removal of pressure is accompanied by an increase in water content, but this increase is far less than the decrease in

water content during consolidation. Thus, although the clay at point (d) is under the same effective stress as at point (b), the water content of the overconsolidated clay is considerably less. The particles are in a denser state and the shear strength is greater than that of a normally consolidated clay.

Bjerrum (1967) presented a similar picture for the consolidation process, with the exception of the formation of diagenetic bonds. After the clay is normally consolidated to point (c), considerable time may elapse with little or no change in load. Due to the large overburden stress, time, and other agents, physical and chemical alterations may take place within the clay. The processes causing the alteration are collectively termed diagenesis; the bonds arising from recrystallization of particles, adhesion between particles, or precipitation of cementing agents in the zones of contact between particles are collectively termed diagenetic bonds.

Thus, under high stresses and over long periods of time, the clay can become stronger and more brittle with small volume change. The diagenetic bonds can be so strong that the clay is indurated and can be classified as a soft rock. During unloading the clay tends to swell and to take on additional water; the stronger the bonds, however, the more the clay will be inhibited from expanding and therefore the smaller the increase in water content—point (e), Fig. 2. Upon removal of the consolidation load, energy may be retained or released, depending on the strength of the diagenetic bonds. Bjerrum classified these bonds as weak, strong, or permanent. Clays forming weak bonds release nearly all their energy on unloading; clays forming strong bonds release energy



(After Skempton, 1964 and Bjerrum, 1967)

Fig. 2. Schematic of geologic history of overconsolidated clay.

slowly in response to weathering processes; and materials forming permanent bonds never release their energy.

Quantitative data on the amount of energy retained during consolidation were studied by Brooker (1967). Five remolded samples of natural clays were consolidated in a one-dimensional consolidation apparatus. The axial strain was determined on loading and unloading for each test as shown in Fig. 3. The absorbed strain energy is the area bounded by the loading and unloading axial stress-strain curves (Seelye and Smith, 1952). Table 2 shows the Atterberg limits, clay fraction, and mineralogy of the materials studied by Brooker. The amount of energy absorbed, according to Brooker, includes:

1. Work expended in consolidation (partially recoverable)
2. Elastic deformation (recoverable on release of constraint)
3. Work in the formation of diagenetic bonds (partially recoverable depending on the strength of the bonds)

The consolidated materials were then slaked to relate the visually estimated percent of disintegration to the absorbed strain energy. From the results, Fig. 4, Brooker concluded that disintegration, as indicated by slaking, was a function of absorbed strain energy, and suggested that the absorbed strain energy was dependent on the clay mineralogy.

The full impact of the effect of diagenetic bonds is shown when effective

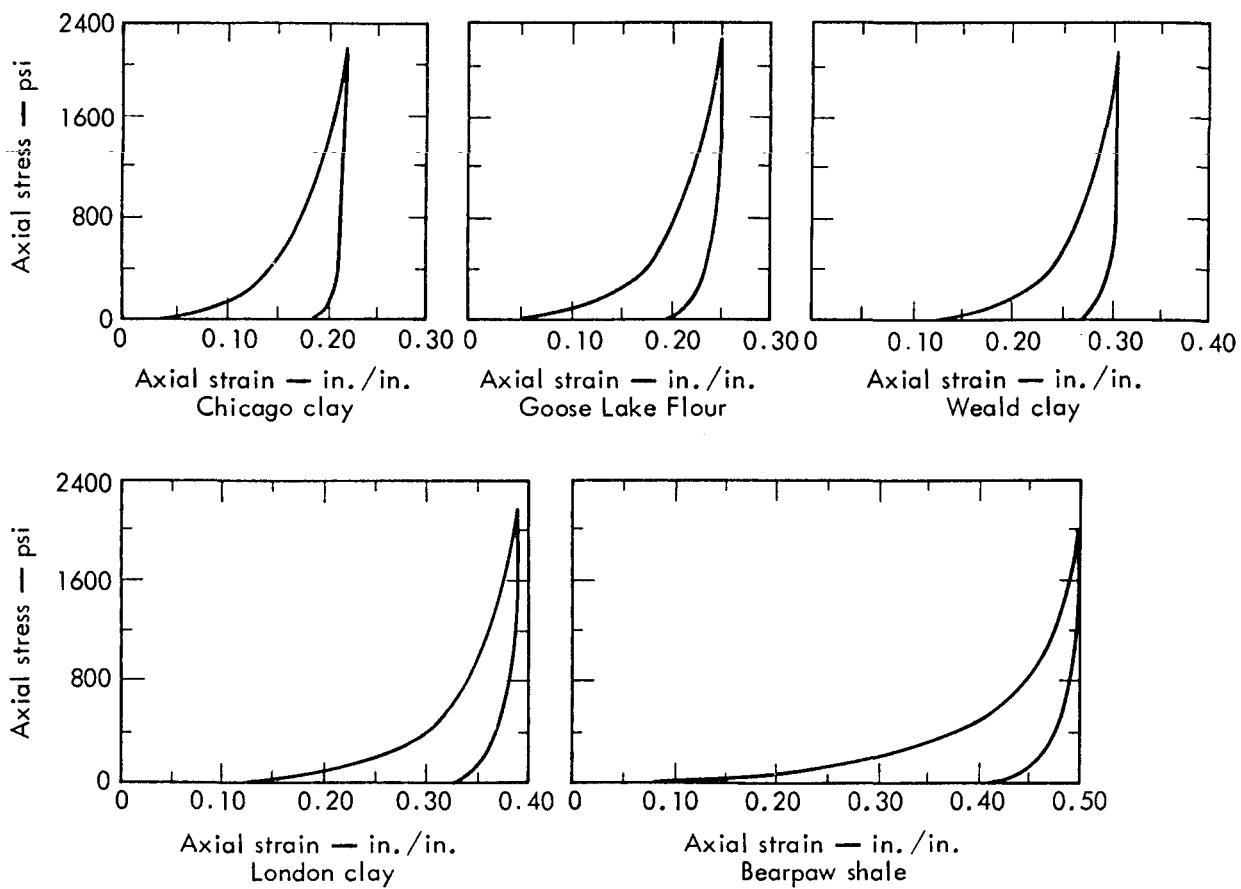


Fig. 3. Stress-strain relationships for one-dimensional consolidation tests (Brooker, 1967).

Table 2. Characteristics of clays tested by Brooker (1967)

Material	LL ^a	PL ^b	PI ^c	<2 μ ^d (%)	Activity ^e	Mineralogy (%)			
						Mont.	Illite	Kaol.	Non-clay
Chicago clay	28	18	10	36	0.29	5	40	—	55
Goose Lake flour	32	16	16	31	0.50	—	15	65	20
Weald clay	41	21	20	39	0.53	10	15	15	60
London clay	64	26	38	64	0.60	15	35	35	15
Bearpaw shale	101	23	78	59	1.53	60	—	5	35

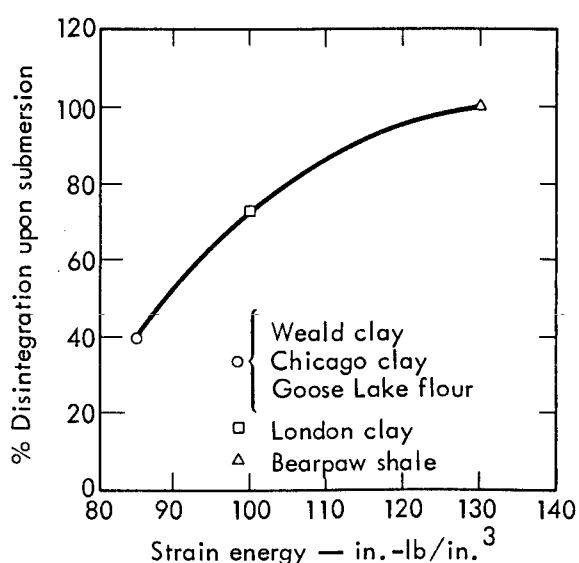
^aLL - liquid limit^bPL - plastic limit^cPI = LL - PL^dPercent, by weight, finer than 2μ^eActivity = PI/<2 μ (Skempton, 1953)

Fig. 4. Relationship between degree of disintegration and strain energy (Brooker, 1967).

horizontal stresses are considered, as schematically illustrated in the lower part of Fig. 2. During deposition the effective horizontal stresses, as well as the effective vertical stresses, increased. Upon unloading, the clay expanded because it possessed some recoverable strain energy. Since the clay was more free to expand in a vertical direction than in a horizontal

direction, the changes in effective vertical stress were larger than those in effective horizontal stress. For clays having weak diagenetic bonds, the behavior is represented by point (d), Fig. 2. If the clay had been one in which strong diagenetic bonds had developed, the bonds would have served to prevent the clay from expanding and, as a result, the horizontal effective stresses would be smaller than those for an unbonded or a weakly bonded clay (compare point (e) to point (d), Fig. 2). Destroying these bonds, by weathering processes for example, causes high horizontal stresses to be developed later in the relatively shallow weathered zone.

Peterson (1954) showed that the *in situ* horizontal stresses may be 1.5 times the vertical stresses in highly overconsolidated clay shales. Skempton (1961) determined in London clay that the ratio of horizontal to vertical effective stresses increased toward the surface from 1.5 at a depth of 100 ft to 2.5 at 10 ft. Brooker and Ireland (1965) gave experimental evidence that large horizontal stresses could occur in overconsolidated clays. Brooker (1967)

showed that at high overconsolidation ratios the clays with low plasticity gave higher values of horizontal pressure than the clays with high plasticity (Fig. 5). In spite of the relatively short time of consolidation, clays with higher plasticity apparently formed the stronger diagenetic bonds; however, when these interparticle bonds were destroyed during slaking, the release of strain energy severely disintegrated the clay. It should be noted that the absorption of large amounts of strain energy by a clay does not necessarily mean the formation of strong diagenetic bonds. The formation of strong bonds due to recrystallization is believed to be a function of the duration of loading as well as of mineralogy. Temperature may also be an important factor (Muffler and White, 1969).

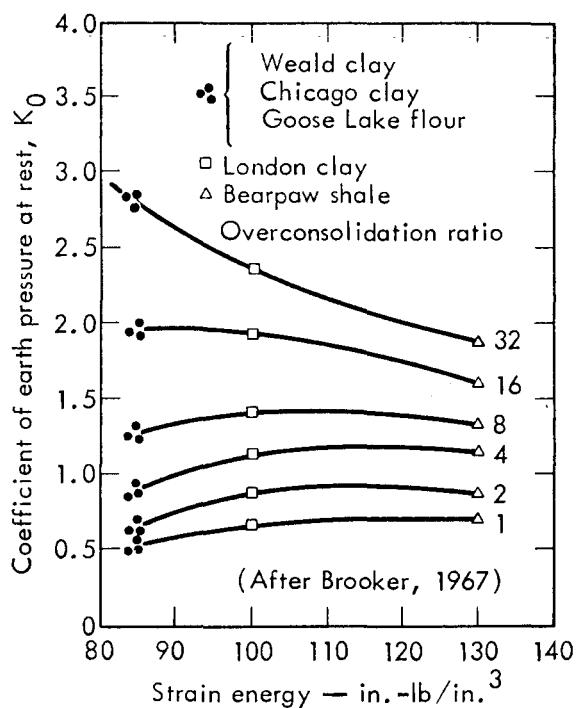


Fig. 5. Relationship between earth pressure at rest and strain energy for various values of overconsolidation ratio.

In clay deposits an upper limit to possible horizontal effective stress exists (Terzaghi, 1962). At a certain difference between the vertical and horizontal effective stresses, the shear stress will become equal to the shear strength and a shear failure will occur (Fig. 2). Due to the brittleness of overconsolidated clays, a shear plane may develop when small movements occur. In hard clay shales with strong bonds, only small movements are required to reduce the horizontal stresses, whereas larger movements are needed in clays with weaker or no bonds (Bjerrum, 1967).

Lithology

The lithology refers to the composition and texture of the constituents and the degree of cementation or bonding. The mineralogy of all constituents in general, and the mineralogy of the clay fraction in particular, affect the formation of diagenetic bonds, and the strength, the potential swelling, and the slaking action of clays. Brooker (1967) showed that the strain energy absorbed during loading and unloading was generally related to both the mineralogy and the fraction of clay sizes present (Table 2). The more plastic the soil, as evidenced by increased clay content and increased amounts of montmorillonite relative to illite and kaolinite, the more strain energy could be absorbed under consolidation.

The tendency of a clay shale to take on water and at the same time suffer destruction of bonding is related to the mineralogy and is reflected by large swell pressures. Clay shales composed mainly of montmorillonite and illite tend to absorb large amounts of water into their crystalline

structures. The result of the absorption of water is the tendency to swell to much larger volumes than evidenced by kaolinitic clays. Casagrande (1949) observed that overconsolidated, highly slickensided clay shales have greater swelling tendencies than clay shales having only a few or no slickensides. The slickensides possibly increase the permeability of the clay shale mass by providing additional passageways for available water to reach the clay minerals (Underwood, 1967).

Two factors, the tendency of a clay mineral to swell by absorbing water and the tendency of the water to destroy any diagenetic bonds, are reflected in the slaking behavior of a clay shale. Slaking refers to the tendency of certain materials to disintegrate when exposed to alternate cycles of wetting and drying and is considered an index of susceptibility to weathering (Philbrick, 1950). However, the results of slaking tests are strongly influenced by the method of testing, and care must be taken in inferring the performance of clay shales in slopes from slaking behavior observed in the laboratory. Samples slaked at natural water content are less affected than samples air-dried prior to slaking (Volume 2, Appendix C). An empirical method of measuring the tendency of a clay or clay shale to slake was given by Skempton (1953) through the activity ratio. The activity is defined as the ratio of the plasticity index to the clay fraction (percent, by weight, of particles smaller than $2\ \mu$). Inactive clays (activity ratio less than 0.75) include kaolinite and clay-size particles of mica, calcite, and quartz. Active clays include illite (activity ratio, 0.90), calcium montmorillonite (1.5), and

sodium montmorillonite (7.2). The tendency to slake increases as the activity increases, all other factors being equal.

The effect of cementation of clay shales as a stabilizing factor is an area in which little information is available. A relatively large amount (35 to 40%) of calcite effectively cements the Crow Creek member of the Pierre formation (Tourtelot, 1962). The Lykins formation in the Denver, Colorado area is strongly cemented with iron oxide. That material contains 30% clay minerals, 90% of which is illite and mixed-layer calcium montmorillonite-illite. The Mowry shale, also from the Denver area, contains 65% clay minerals, half kaolinite and half illite, but is indurated with silica—35% (Scott et al., 1966). On the other hand, a sample of Claggett clay shale (Volume 2, Appendix C) contained over 11% material, presumably calcite, that was soluble in dilute hydrochloric acid; yet a piece of the sample slaked readily when exposed to alternating cycles of wetting and drying. The soluble material was evidently concentrated as particles in the shale and did not effectively bond the clay particles. Known cemented materials, such as the Mowry shale, the Lykins formation, and the Crow Creek marl, will not slake even after repeated cycles of air-drying and slaking. However, the interpretation of a slaking test on a slightly cemented clay shale is difficult and somewhat questionable, since it appears that the forces involved in the slaking process could disintegrate the specimen by overcoming the effect of cementation.

Strength

The previous discussion has illustrated the effect of overconsolidation and lithology

on the creation of diagenetic bonds and the subsequent creation of high horizontal stresses caused by the combined action of swelling and weathering. Both the overconsolidation process and the lithology influence the strength characteristics of a clay shale. Work by many investigators has shown that the ultimate stability of clay shale slopes, once failure has been initiated, is controlled to a large extent by the residual shear strength. The lithology is by far the more important factor (see later discussion). The concept of residual shear strength of clay shale was emphasized by Skempton (1964) in his discussion of the long-term stability of clay slopes. Skempton suggested that the field behavior of overconsolidated clays could be explained on the basis of two strengths as interpreted from a strength test (Fig. 6).

If a specimen of clay is placed in a shearing apparatus and subjected to displacements at a very slow rate (drained conditions), an increasing resistance to shear will be observed with increasing displacement. At some displacement, the limiting shear resistance will be reached; this value is termed the peak strength, s_p . If the displacements are carried further, the shear resistance will decline (strain-softening) until a second limit, the residual shear strength, s_r , is reached. The residual strength is the minimum strength reached for a constant effective normal stress no matter how large the displacement. The plot of strength (s_p , s_r) vs effective normal stress on the shear plane (Fig. 6) indicates two very different relationships. The peak shear strength can be expressed by:

$$s_p = c' + \sigma' \tan \phi',$$

and the residual shear strength by:

$$s_r = c'_r + \sigma' \tan \phi'_r,$$

where

s_p , s_r = peak and residual shear strengths, respectively

c' , c'_r = drained cohesion for peak and residual shear strengths, respectively

σ' = effective normal stress acting on failure plane

ϕ' , ϕ'_r = drained angles of internal friction for peak and residual shear strengths, respectively

Since c'_r is normally very small and in most cases zero, the residual shear strength is usually expressed by:

$$s_r = \sigma' \tan \phi'_r.$$

In other words, in moving from the peak to the residual shear strength, the cohesion approaches zero. During the same process the angle of internal friction also decreases. For normally consolidated clays the reduction from s_p to s_r is usually small, but for overconsolidated clays the reduction is usually dramatic.

During the shearing process, overconsolidated clays tend to expand, especially after passing the peak. Part of the drop in strength from the peak value is caused by an increase in water content (Fig. 6). Of comparable importance, however, is the development of thin bands or domains in which the clay particles are oriented in the direction of shear. Detailed fabric studies by Morgenstern and Tchalenko (1967) indicated that the displacement shears and preferred orientation were typically close to the boundaries

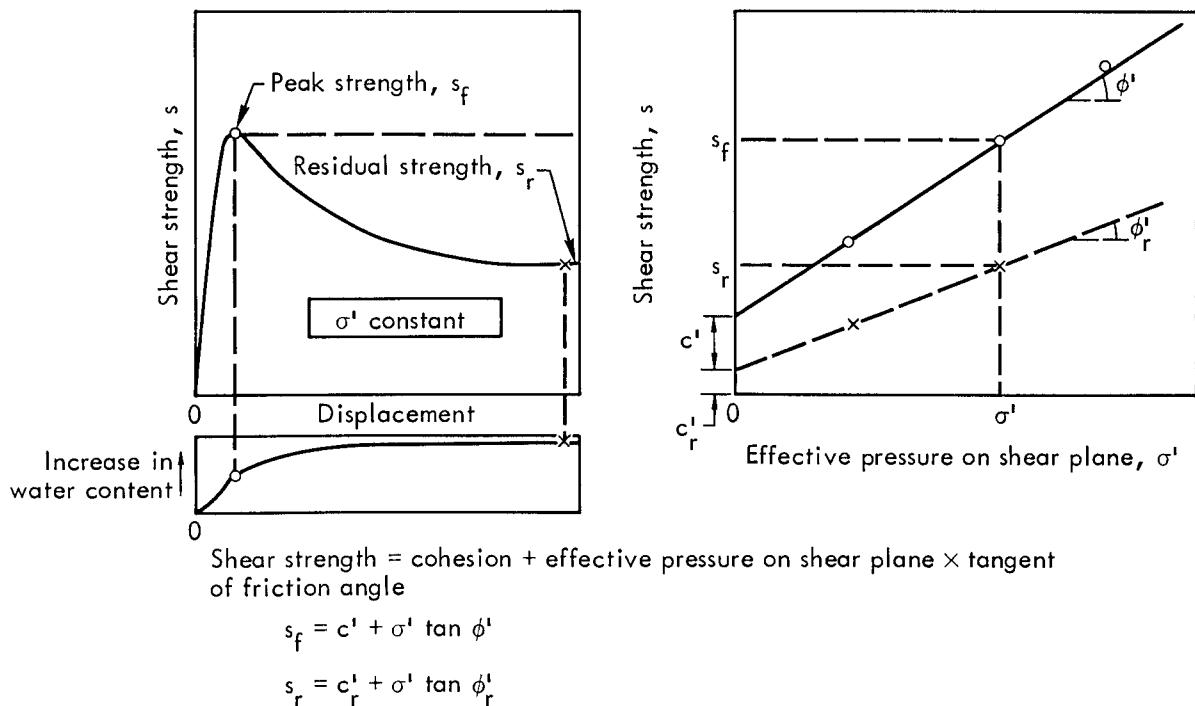


Fig. 6. Shear characteristics of overconsolidated clay (Skempton, 1964).

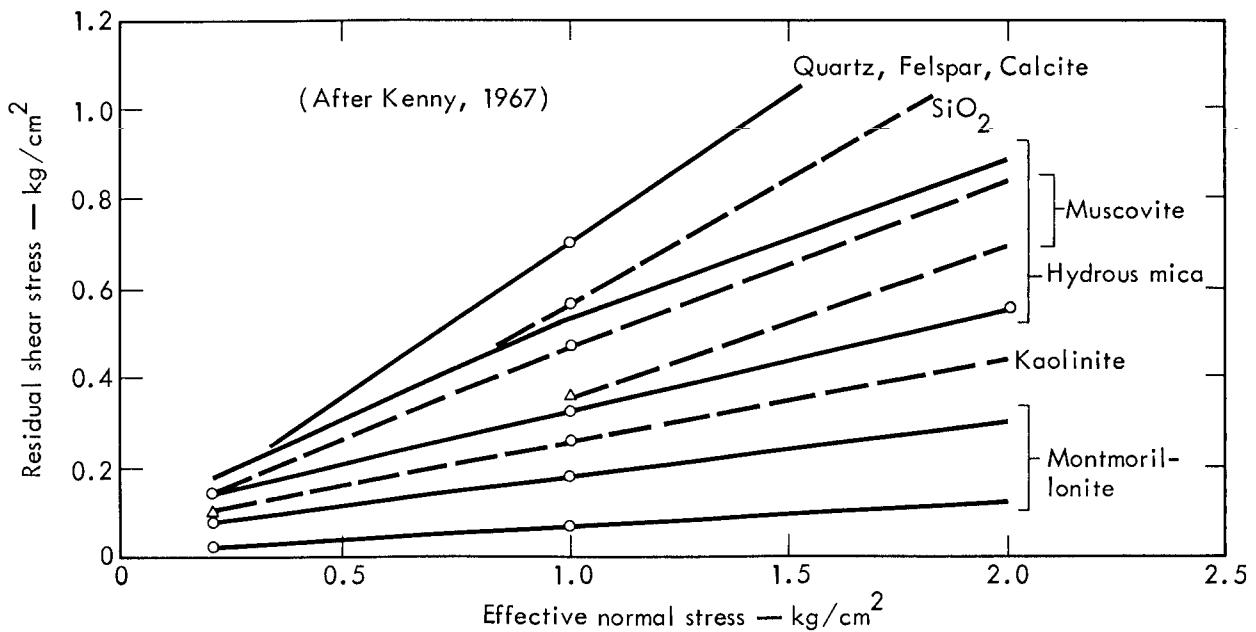


Fig. 7. Relationship between residual shear stress and effective normal stress for different materials.

of the shear zone (from 10 to 100 μ) and depended upon the composition of the sediment, its consistency, and the magnitude

of the relative displacement. The nature of the particle orientation has been studied by Skempton (1966) and Skempton and

Petley (1967). Apparently the polish of slickensides in clay shale is the result of particle orientation during shear displacements. Evidence presented by Smith, Albee, and Jahns (1967) suggested the formation of shiny surfaces without translational displacement. They preferred the term "parting surface" and explained the mode of formation as "pull-aparts resulting from volume change induced by weathering or other processes."

The residual shear strength parameter ϕ'_r is independent of the original strength of the material and such factors as the water content and the liquidity index (Skempton, 1964). The value of ϕ'_r seems to depend only on the size, shape, and mineralogical composition of the constituent particles (Bjerrum, 1967). Kenney (1967) determined the residual shear strength for pure clay samples consolidated from a slurry (Fig. 7). The

indicated residual friction angle for sodium montmorillonite was about 4 deg, calcium montmorillonite about 10 deg, kaolinite about 15 deg, and hydrous mica or illite from 16 to 24 deg. The composition of the pore fluid also exerts a pronounced effect on the residual strength. In the case of sodium montmorillonite, the shear resistance increased from about 4 deg for negligible salt dissolved in the pore fluid to 8-1/2 deg with 30 g/l sodium chloride in the pore water. Kenney concluded that the residual friction angle of natural clay materials depended primarily on mineral composition and to a lesser degree on the system chemistry and the effective normal stress. Studies by Skempton (1964) indicated that the residual friction angle depended upon the percentage of clay fraction (percent, by weight, smaller than 2 μ). See Fig. 8.

Testing of Dawson clay shale (USAE, Omaha District, 1968a) indicated an interesting relationship of ϕ'_r with the liquid limit (Fig. 9). For the range in the liquid limit found in the Dawson clay shale, ϕ'_r decreased strikingly with increases in the liquid limit. It should be pointed out that this relationship, while impressive, is limited in use to the Dawson formation. The mineralogy is relatively constant in the clay fraction, and the liquid limit varies (as does ϕ'_r) because of different percentages of clay fraction and coarser material. Similar investigations by the Corps of Engineers involving clay shales have shown decreasing residual friction angles with increasing liquid limits and plasticity indices. However, the correlation curve for one formation cannot necessarily be used in a different formation Beene (1969).

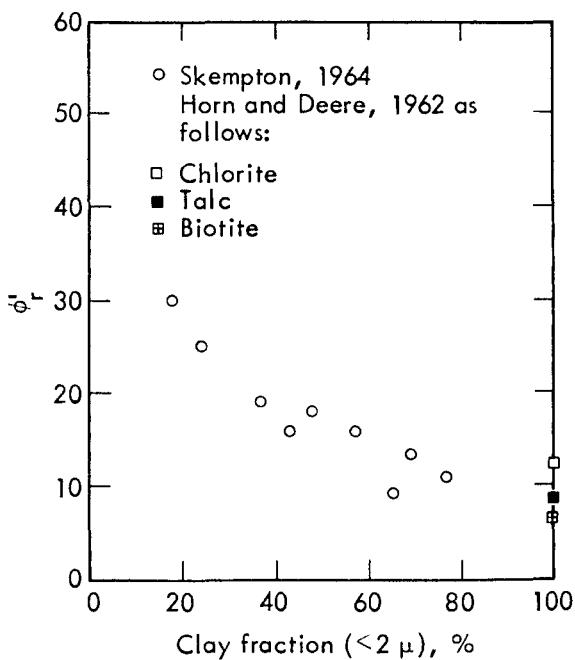


Fig. 8. Decrease in ϕ'_r with increasing clay fraction.

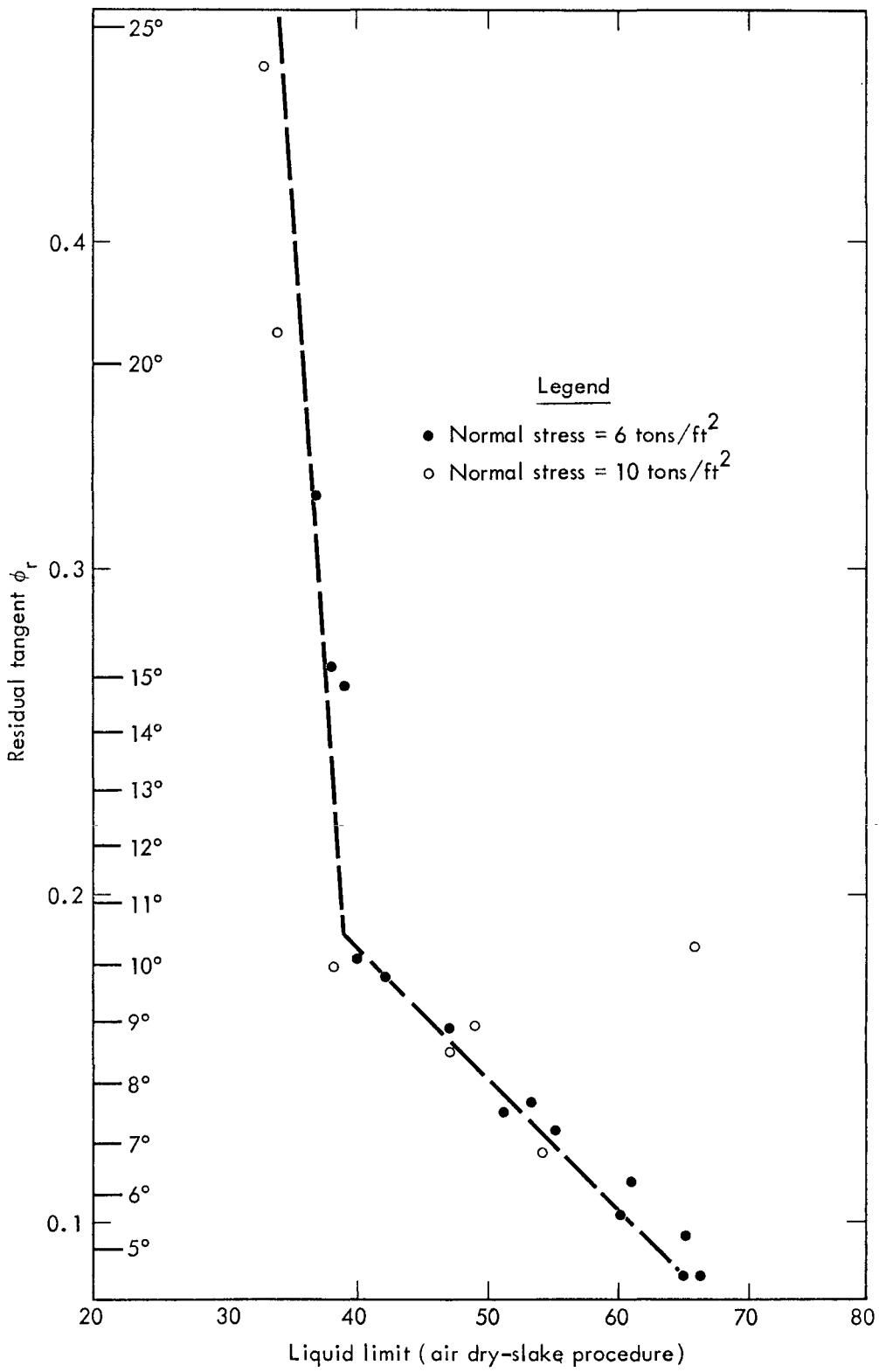


Fig. 9. Dawson clay shale, $\tan \phi'_r$ vs liquid limit.

Table 3. Measured residual friction angles

Material	w ^a (%)	LL ^b	PL ^c	<2 μ (%)	c' _p (lb/ft ²)	ϕ'_p	c' _r (lb/ft ²)	ϕ'_r	Residual factor ^d	Source
London clay (Kensal Green slide)	33	83	30	--	320	20	0	16	0.61	Skempton (1964)
London clay (Hendon)	--	78	29	--	--	--	0	15.5	--	Skempton (1964)
London clay (9 sites in weathered clay)	--	--	--	--	320	20	0	16	--	Skempton (1964)
Blue London clay (Wraysbury)	28	69	26	43	650	20	30	16	--	Skempton and Petley (1967)
Coalport beds (Jackfield slide)	21	44	22	36	220	25	0	19	1.12	Skempton (1964)
Boulder clay (Selset slide)	12	26	13	17	180	32	0	30	0.08	Skempton (1964)
Weathered mudstone (Walton's Wood)	29	57	26	69	320	21	0	13	0.98	Skempton and Petley (1967)
Atherfield clay (Sevenoaks)	34	75	29	57	430	18	70	10.5	0.98	Skempton and Petley (1967)
Weathered weald clay (Sevenoaks)	27	69	27	71	60	15	50	15	--	Skempton and Petley (1967)
Upper Siwalik clay (Jari)	15	53	25	45	1000	23	0	12	0.96	Skempton and Petley (1967)
Upper Siwalik clay (Sukian)	15	58	27	52	1200	22	0	14	0.70	Skempton and Petley (1967)
Culebra clay shale (Panama Canal)	12	80	35	--	--	--	--	10	--	National Academy of Sciences (1924)
Pierre clay shale (Oahe Dam)	--	--	--	--	--	--	--	3.2	--	Herrmann and Wolfskill (1966)
Weathered shales (Balgheim)	27	61	25	--	300	18	--	17	--	Einsele (1961)
Stiff-fissured plastic clay (Sandes)	36	60	30	--	260	22	--	12-18	--	Norwegian Geotechnical Institute (1964)
Pepper shale (Waco Dam)	22	80	22	50	800	14	--	6	--	Beene (1967), Van Auken (1963)
Edmonton clay shale (Fort St. John)	20	60	25	30	--	37	--	8.5	1	Sinclair and Brooker (1967) Sinclair, et al. (1966)
Bentonite (Laseur slide)	60	214	60	92	--	--	--	10	1	Sinclair and Brooker (1967) Sinclair, et al. (1966)
Bentonitic shale (Univ. of Alberta)	30	125	40	45	--	25	--	8.5	1	Sinclair and Brooker (1967) Sinclair, et al. (1966)
Bearpaw clay shale (South Saskatchewan)	32	115	23	--	775	20	--	6	--	Ringheim (1964)
Boom clay (Antwerp)	25-32	81	20	49	300	24	0	19-24	--	De Beere (1967)

^aw = natural water content^bLL = liquid limit^cPL = plastic limit^dResidual factor at time of failure; term is defined and discussed under heading, "Failure Modes in Clay Shales."

Numerous data are available which indicate the range in values of measured residual friction angle. Some of these data are presented in Table 3.

Structure

The structural features of a clay shale may be considered on two scales: small-scale features, such as fissures, slicken-

sides, and possibly joints, and large-scale features, such as bedding planes, faults, unconformities, and possibly joints. The importance of distinguishing between clay shale with and without fissures (Terzaghi, 1936), and the possible influence on swelling characteristics (Casagrande, 1949) have been noted.

Fissures, joints, and slickensides provide avenues for water to enter the materials, with the attendant problems of breaking of diagenetic bonds, swelling, and weathering. In addition, the presence of fissures, joints, and slickensides can adversely affect the strength of the clay shale in a number of ways:

1. Open fissures may form a portion of a potential failure surface across which no shear resistance can be developed.

2. Closed fissures may form a portion of a failure surface on which only the residual shear strength can be mobilized.

3. Fissures, whether open or closed, may adversely influence the stresses within a slope (Skempton and LaRochelle, 1965).

Skempton (1964) discussed the dual action of joints, fissures, and slickensides and suggested that they acted as stress concentrators, which could cause local overstressing, and as discontinuous planes of weakness with strengths at or near residual values. Bjerrum (1967), however, pointed out that the potential failure by a progressive mechanism does not depend on the presence of joints or fissures.

The influence of the fissures on the peak strength has been shown to be considerable. Skempton and LaRochelle (1965) concluded that the peak strength of the stiff-fissured London clay might be reduced as much as 30% by the presence of fissures. One aspect of the influence of fissures on peak shear strength is the influence of sample size on the test results. Bishop (1966) and Peterson et al. (1960) found that the peak shear strength of large samples, containing larger numbers of fissures, could be as much as 80% lower

than the strength determined from smaller, more intact samples. The effect of sample size on the unconfined compressive strength was determined on Bearpaw shale from the Pre-Gondola cratering sites at Fort Peck, Montana (Fisher et al., 1969). The results of tests on 1.4- and 5-in. specimens indicated substantially lower strengths for the larger-diameter samples (Fig. 10). Skempton (1964) noted that the residual strengths should be largely independent of sample size.

A detailed study of the strength characteristics along discontinuities in stiff clays led Skempton and Petley (1967) to establish a tentative classification of discontinuities based on occurrence and relative shear movement. Principal displacement shears, such as those found in landslides, faults, and bedding-plane slips were reported as subplanar and polished; the strength was at or very near the residual strength. Relative shear displacements on the shear surface were described as more than 10 cm. For minor shears, with somewhat irregular surfaces on which the relative movements have been small (less than 1 cm), the strength may be appreciably higher than residual. Joint surfaces, including systematic joints, displayed a "brittle-fracture" texture with little or no relative shear movement. The few tests available indicated that the fracture which produced the joint virtually destroyed the cohesion but reduced the friction angle ϕ' by only a very slight amount. Movements of not more than 5 mm, however, were sufficient to bring the strength along the joint to the residual and to polish the joint. Although information was largely lacking on the strength along bedding planes and faults, Skempton and Petley

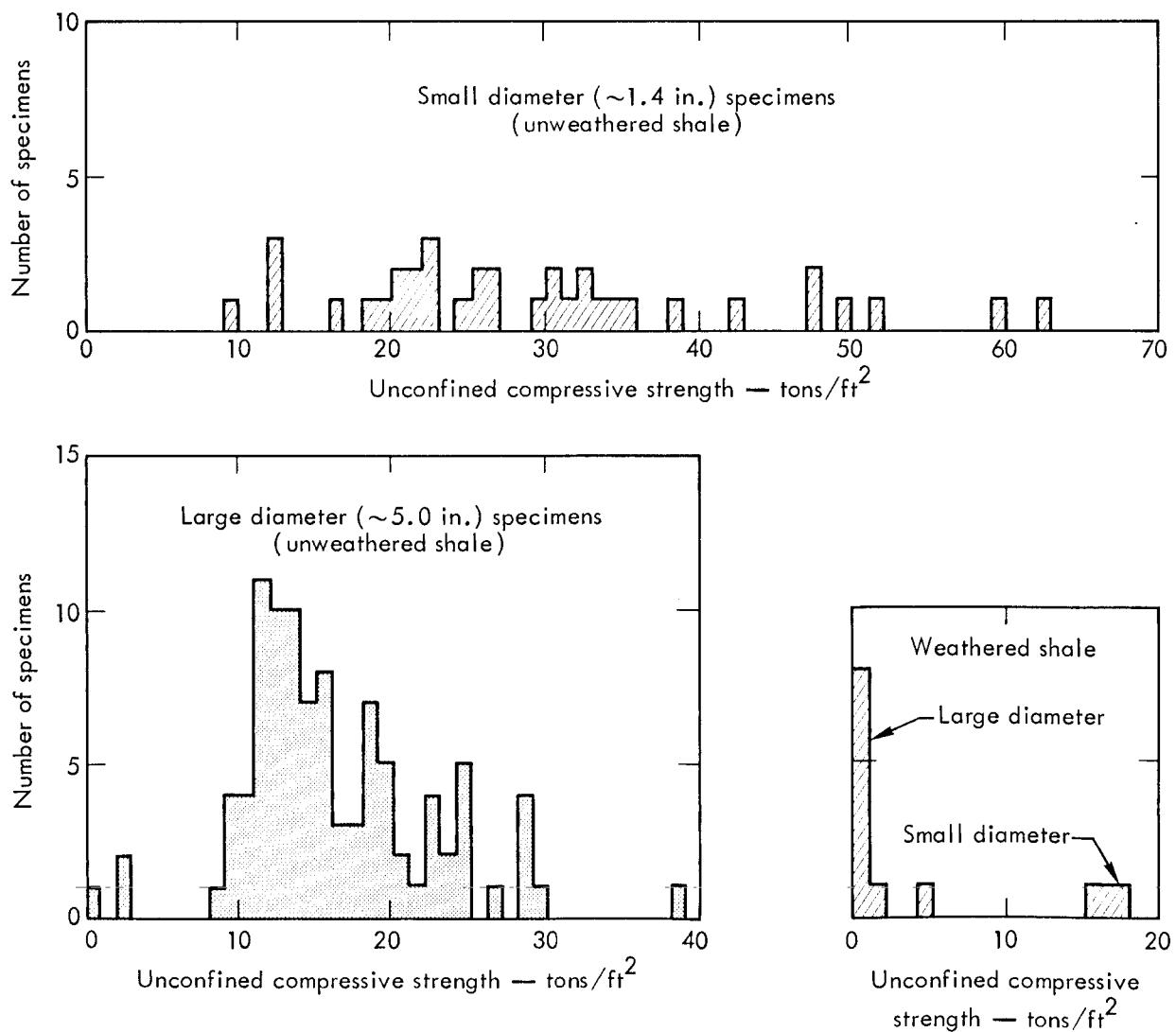


Fig. 10. Unconfined compressive strength of Bearpaw shale for large- and small-diameter specimens (Fisher et al., 1969).

(1967) concluded that there was no reason to suppose that the strength along faults differed appreciably from that along principal slip surfaces.

The large-scale features, bedding planes, faults, unconformities, and possibly joints, are also major considerations in the behavior of a slope. These features, while zones of structural weakness, may be oriented in such a manner that they control the location of failure surfaces.

Information contained in Volume 2, Appendix E, indicates a strong prevalence of landslides in situations where bedding offers a potential sliding plane out of the slope; joint planes and faults conceivably offer the same potential. Observations of slope movements along pre-existing discontinuities and bentonitic bedding planes are described in Volume 2, Appendix D, for the Oahe Dam and the Fort Peck Dam spillways.

EFFECT OF EXTERNAL AGENTS ON CLAY SHALE BEHAVIOR

Several external or environmental factors operate on clay shales to alter their physical characteristics from dense rock-like shale to soft, weak clay. For purposes of discussion, these are divided into geologic changes, weathering, and groundwater conditions. Weathering and groundwater conditions could be considered as special cases of geologic changes, but because of their importance they are treated separately.

Geologic Changes

During consolidation the clay particles and the water in a deposit tend toward equilibrium with the stress environment. As mentioned previously, removal of the load results in large in situ residual stresses, allowing the materials to expand

and to take on water. The unloaded system tends toward equilibrium with the existing stress state and moisture environment. Therefore, changes induced by geologic processes (such as valley erosion or advances and retreats of glaciers), changes in stream regimen (perhaps manifested by terrace development), and engineering activities (such as placement of fills or excavations) exert a pronounced effect on the behavior of clay shale slopes. Although important, the geologic history of a formation is known only in general terms, and the determination of historical details is at best only qualitative. For example, Scott and Brooker (1968) presented a qualitative history of the Bearpaw formation in Canada in a graphical form (Fig. 11) to illustrate the interaction of water content and effective overburden stress for a time span covering the complete

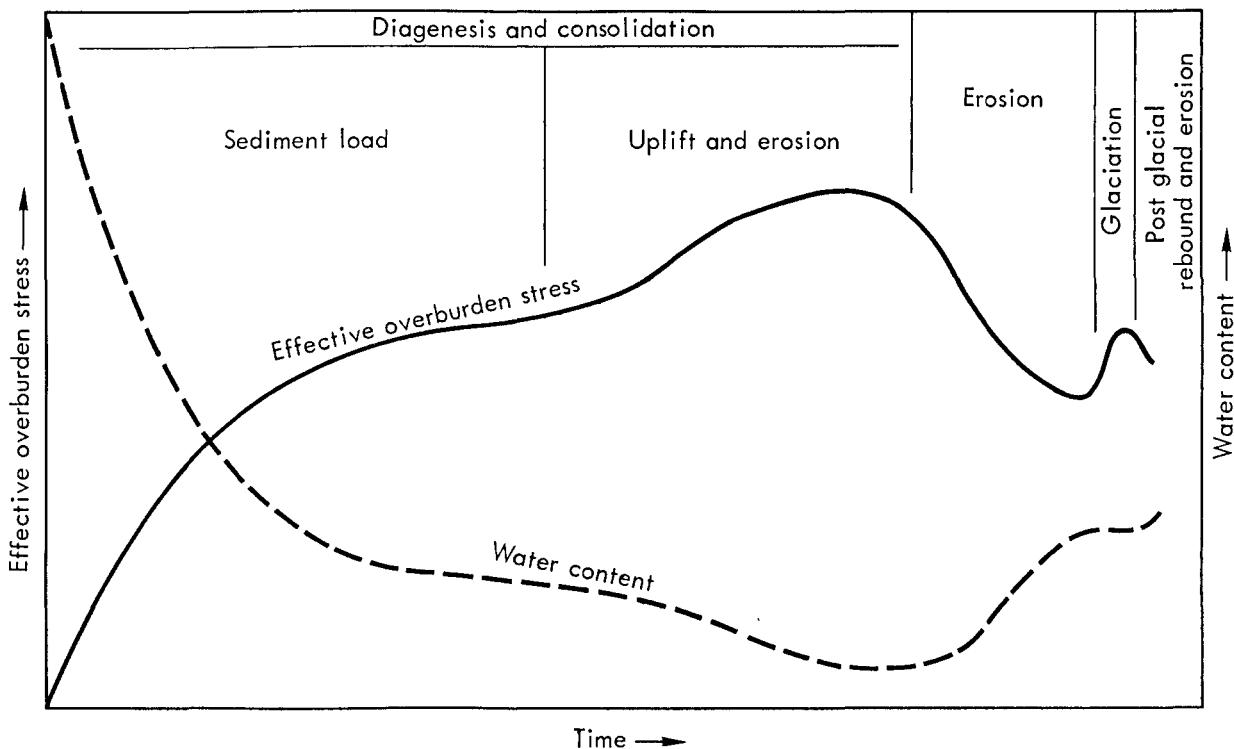


Fig. 11. Probable stress history of Bearpaw sediments (Scott and Brooker, 1968).

development of the Bearpaw shale (a period of time on the order of 70 million years).

The development of slopes along the Missouri River trench reflects the complications of geologic and recent history. Part of the trench was formed along the face of a receding ice sheet. Very rapid downcutting to depths of approximately a hundred feet below the existing river bottom caused massive slumping of the valley walls. The walls of the trench contained many discontinuities with the strength along them at or very close to the residual value. Later, through a change in base level or stream regimen, the river began to aggrade. The development of secondary drainage on the valley walls induced numerous smaller slides by local stress release and oversteepening. The manifestations of some environmental changes are noted in Volume 2. At Oahe Dam, excavation in the left abutment caused renewed movements along old failure planes (Appendix D); at Anderson Lake, California, filling of the reservoir caused renewed movement of older and apparently stable slides (Appendix E).

The depositional environment affects the lithology, the nature of the clays in the deposit, the composition of the pore fluid, the rate of consolidation, and the amount of consolidation loading. In this sense the occurrence and strength of diagenetic bonds, residual stresses, and the shear strength of the clay shale are dependent upon the depositional environment. Differences in slope behavior between the Dawson (Appendix D) and the Fort Union (Appendices A through D), both nonmarine materials, and the Bearpaw, the Pierre, and the Claggett (Appendices A through D), all marine materials, are at

least partly due to their different depositional environments.

Other external geologic factors, such as the base of the stream channel, the stream channel configuration, the terrace development, the rate of erosion, the slope exposure, and the variations in water table act collectively to vary shear stresses and shear resistances. Scott and Brooker (1968) cite an example along the Red Deer River in Alberta where extensive terrace development contributed to increased stability. They contrasted the greater prevalence of failures along unterraced tributaries with few slope failures in the main river valley. In the badlands area of western North Dakota, the effect of slope exposure is pronounced. Where the principal drainage is west to east, the north-facing slopes have developed a hummocky landslide topography with numerous ponded depressions which support trees and dense scrub brush. The south-facing slopes are much smoother and generally lack ponded depressions. The vegetation on these slopes consists mainly of grasses and sage.

Weathering

Weathering involves the various forms and combinations of chemical alteration including ion exchange, solution and recrystallization of minerals, and physical disintegration. The intensity of the weathering processes are strongly affected by local climatic factors such as temperature, the amount and intensity of rainfall, frost penetration, freeze-thaw action, and the chemical composition of groundwater. The maximum depth and rate of weathering penetration are influenced by the structural relaxation and rebound

features which result from release of load after consolidation. Structural relaxation increases toward a free surface; part of the volume increase is caused by expansion of the shale and part by cracks or fissures. The minute openings provide access for weathering agents, principally water.

Bjerrum (1967) used the term weathering to include all changes which occur in the upper layer of clay, including physical changes that do not originate from climatic conditions. Weathering can be thought of as including two phases:

1. Disintegration of diagenetic bonds
2. Chemical changes of the minerals

The major effect of disintegration is a gradual destruction of the bonds which have attempted to maintain the original structure of the clay. As the bonds are destroyed strain energy will be liberated, causing large in situ stresses. Depending on the overburden stress, the clay may expand, the water content may increase, and the shear strength may decrease. The expansion will occur in a direction normal to the slope but will be inhibited in a direction parallel to the slope, thus causing a relative increase in the effective stresses acting parallel to the slope. The importance of weathering or disintegration of bonds depends largely upon the strength of the bonds in the clay. If the bonds are weak initially, the increase in effective horizontal stresses upon weathering will be small; if the bonds are strong initially, the increase in effective horizontal stresses will be large.

In general, three zones have been distinguished beneath the surface in which the process of disintegration has proceeded to varying degrees. Immediately beneath the surface the clay may have been subjected to

temperature changes including freeze-thaw cycles and repeated wetting and drying. The strains caused by these processes are very effective in producing complete disintegration. Thus, the near-surface behavior will depend more on climatic conditions than on the initial water content and strength of the parent clay shale. Beneath the surface zone, a zone of active advanced disintegration occurs. The strains in this zone can originate from cyclic variations of pore pressure resulting from groundwater fluctuations, seasonal temperature variations, and unloading of the overlying soil. The lowest zone is the zone of medium disintegration. Here, changes in surface climatic conditions are not experienced. However, the water content of this zone varies considerably from point to point, indicating a point-to-point variation in mineralogical composition, extensiveness of surface jointing, or other factors. The clay will be subjected to nonuniform swelling, a function of mineralogy, with resultant nonuniform strains leading to local shear failures and the formation of cracks and fissures.

Weathering effects have been observed at considerable depths. In the Bearpaw shale in the South Saskatchewan (now Gardiner) Dam, Peterson (1954) found evidence of weathering at a depth of 50 ft. Skempton (1964) found weathering at depths of 30 to 40 ft in London clay. Gould (1960) observed weathered zones at depths of 25 to 45 ft in the Tertiary clay shales of California. In overconsolidated clay shales essentially free of bedding, Bjerrum (1967) postulated that the seat of movement for slides is located near the lower boundary of the zone in which weathering is in progress. He cited observations by

Einsele (1961) that slides in overconsolidated shales having strong diagenetic bonds mostly occur at the base of active weathering. Einsele further reported that where slides had occurred on natural slopes or had taken place some years after an excavation the water content in the slide zone was higher than in the clays above or below.

Groundwater Conditions

Groundwater is a most significant factor in clay shale behavior. The action of water in weathering, destroying diagenetic bonds, reducing the shear strength, and inducing swelling has been discussed. In addition, water may cause differences in pore pressure due to (1) seasonal variations of groundwater flow, (2) submergence of a portion of a slope, or (3) osmotic pressures resulting from chemical differences between the groundwater and the pore fluid. The increase in pore pressure causes a decrease in total shear resistance. Water also serves to increase the forces tending to cause instability by adding weight to the mass.

Regional climatic conditions and precipitation rates are important in their effect on slope behavior. Stable slopes in arid regions of New Mexico and elsewhere exhibit evidence of abundant ancient slides during Pleistocene time when the climate was wetter. Slopes in the Powder River Basin, Wyoming, are relatively stable near Gillette; near Sheridan, however, an increase of a few inches in annual precipitation causes numerous problems with slopes in the same materials (Volume 2, Appendix E).

Some massive-appearing clay shales are composed of layers that vary signifi-

cantly in permeability. Intact clay shale may have coefficients of permeability on the order of 10^{-10} cm/sec. The intact layers may be adjacent to other more pervious layers. Water seeping through the more permeable layers may develop large artesian or uplift pressures on the less permeable layers. Perched water tables are common in clay shale areas, particularly where clay shales are capped or overlain by pervious materials. The rapid entrance of water from such sources, once slides are initiated, may compound unfavorable field conditions. In the upper weathered zone, the permeability along cracks, fissures, and joints may be several orders of magnitude greater than the permeability of intact shale. The piezometric level in this zone varies seasonally.

FAILURE MODES IN CLAY SHALES

In the preceding discussion the physical characteristics of clay shale and the external agents which affect clay shale behavior were described, with frequent reference to the failure of clay shale slopes. Several studies were made on the occurrence, configuration, and classification of landslides (Sharpe, 1938; Varnes, 1958). Methods of analyzing slopes and slope failures have been developed and applied successfully in many situations.* Mounting experience has indicated, however, that methods successful for normally consolidated clay masses are not applicable to overconsolidated soils. Figure 12

* (See for example, the papers presented at the ASCE Conference on Stability and Performance of Slopes and Embankments, Berkeley, Calif., 1966).

indicates the factor of safety of a number of failed slopes computed from the un-drained or total stress method, as a function of the liquidity index (Peck, 1960). The liquidity index is defined as

$$I_L = \frac{w - PL}{LL - PL}$$

where

w = natural water content

PL = plastic limit

LL = liquid limit

In normally consolidated soils the liquidity index is near and usually greater than 0.5; for overconsolidated soils the liquidity index approaches zero and is often negative. Figure 12 illustrates the unreliability of the undrained or total stress method of analyzing slopes in overconsolidated materials. Table 4, compiled by Duncan and Dunlop (1968), further illustrates the general inability to analyze failures in

overconsolidated clays by conventional methods. In all the slides reported, the back-calculated factor of safety varied between 1.2 and 2.5. By painstakingly studying the properties of clay shale and observing the external agents and behavior of slopes and projects in clay shale, various investigators have begun to formulate the concept of progressive failure within clay shales.

Progressive Failure

When a slope in clay shale is created or loaded, for example by embankments or groundwater changes, corresponding changes in the shear stresses will occur. If at some point within the mass the shear stress exceeds the shear strength, the mass will fail at that point and the shear strength will decrease toward the limiting value of the residual strength, if it is possible for displacement to occur

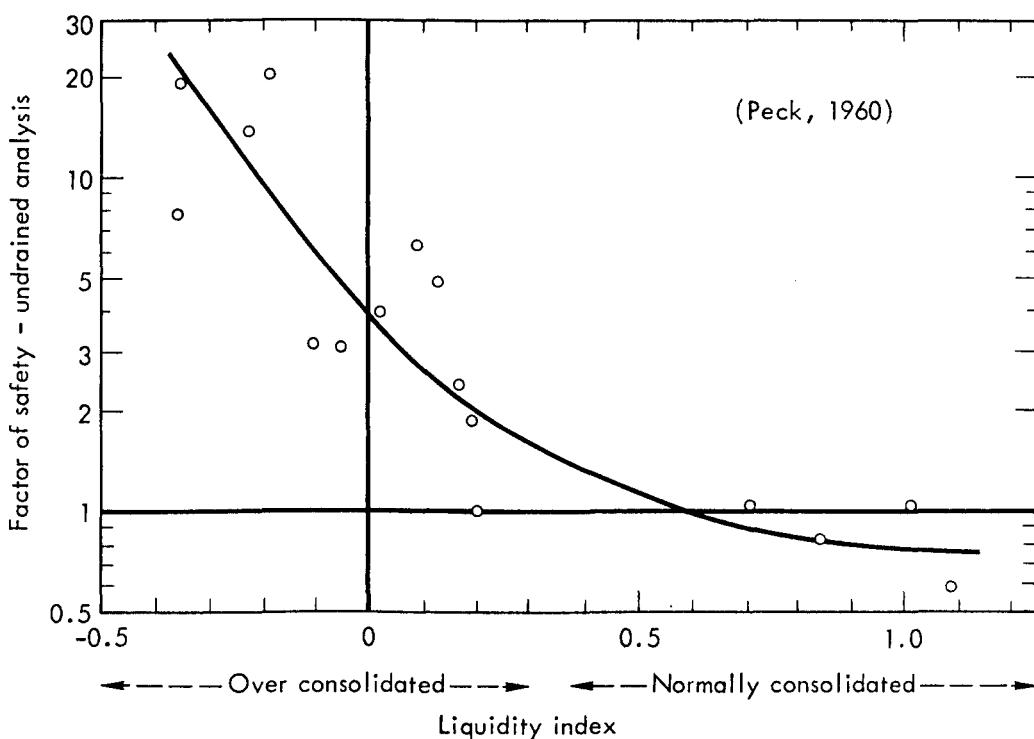


Fig. 12. Calculated factor of safety as a function of liquidity index for un-drained analyses (all slopes failed).

Table 4. Slides in stiff-fissured clays and shales^a

Slide	Type of slope	Height (ft)	Time from end of construction to failure	Clay characteristics			Factor of safety	Type of analysis	Reference
				LL	PL	w			
North Ridge Dam	Embankment	70	Failed during construction	72	21	36.5	1.23	$\phi_u = 0$	Peterson, et al. (1957)
Seven Sisters, S-1	Embankment	17	Failed during construction	85	26	45	1.8	$\phi_u = 0$	Peterson, et al. (1960)
Waco Dam	Embankment	85	Failed during construction	80	22	22	—	—	Van Auken (1963); Beene (1967)
Seven Sisters, S-6	Embankment	17	5 yr	85	26	45	1.33 to 1.40 ^b	Effective stress	Peterson, et al. (1960)
South Saskatchewan River Dam - Canal	Excavation	40	Failed during construction	80-150	18-27	19-35	-2.5 ^c	$\phi_u = 0$	Peterson, et al. (1960)
Bradwell	Excavation	60	5 da	95	30	33	1.8	$\phi_u = 0$	Skempton and LaRochelle (1965)
Northolt	Excavation	48.5	19 yr	79	28	30	1.35	Effective stress	Henkel (1957); Skempton (1964)
Kensal Green	Excavation	20	29 yr	78	29	29	1.6	Effective stress	Henkel (1957); Skempton (1964)
Sudbury Hill	Excavation	23	49 yr	82	28	31	-2 ^c	Effective stress	Skempton (1964)
Wood Green	Excavation	37	55 yr	78	30	31	1.32	Effective stress	Henkel (1957)
Jackfield	Natural slope	-95	--	44	22	21	1.45 to 2.06	Effective stress	Henkel and Skempton (1955); Skempton (1964)

^aDuncan and Dunlop, 1968.^bFactor of safety = 1.65 by effective stress analysis including side forces on slices.^cRatio of average peak strength to average shear stress at failure.

(Skempton, 1964). This action will cause a local redistribution of stresses, so that the peak strength of adjacent points in the mass will be exceeded. In such a manner a progressive failure can be initiated, ultimately causing large portions of the shear surface to be at the residual strength value. Although a slip may occur before the residual value is attained everywhere within the mass, continued sliding will cause the average strength to decrease toward that limiting value.

Bjerrum (1967) explained the progressive failure process in a uniform slope. His important conclusions are as follows:

1. The development of a continuous failure surface by progressive failure is possible only if there exist or can develop local shear stresses exceeding the peak shear strength of the clay. All other conditions being equal, the danger of local shear failures will increase with the ratio p_h/s_p , where p_h is the horizontal effective internal stress and s_p is the peak effective shear strength.

2. The advance of a failure surface must be accompanied by local differential strains in the zone of shear failure suf-

ficient to strain the clay beyond the peak failure. The ratio e_h/e_p is a measure of the degree to which the horizontal strain, e_h , caused by recoverable strain energy in the clay will exceed the peak failure strain, e_p , when the lateral stresses are reduced.

3. The clay must show a large and rapid decrease in shear strength with strain after the failure strength has been mobilized so that the shear resistance in the failure zone will not obstruct the movement required to obtain differential strain and to move the zone of stress concentration into neighboring zones of unfailed clay. The ratio s_p/s_r , where s_r is the residual effective shear strength, expresses the degree of strain-softening of the clay. The importance of these ratios in determining the relative danger of progressive failure in clay shale slopes is illustrated in Table 5.

The many factors which influence the shear strength, the stresses, the nature of the discontinuities, and the mineralogy of clay shale make it difficult to develop analytical tools for analyzing the stability of clay shale slopes. Some aspects of the

Table 5. Relative danger of progressive failure (Bjerrum, 1967)

	Overconsolidated plastic clay with weak bonds		Overconsolidated plastic clay with strong bonds		Overconsolidated clay with low plasticity
	Unweathered	Weathered	Unweathered	Weathered	
p_h/s_p^a	2 ^b	3 ^b	0 - 1 ^b	3 ^b	1 ^b
e_h/e_p^a	2	2	1	3	0 - 1
s_p/s_r^a	2	1	3	2	0 - 1
Relative danger	High	High	Low	Very high	Very low

^a p_h = horizontal effective internal stress^a s_p = peak effective shear strength^a s_r = residual effective shear strength^a e_h = horizontal strain^a e_p = peak failure strain^b Potential for progressive failure of various types of clay, based on an evaluation of the degree to which the three significant ratios are fulfilled. Notation used:

0 Fulfillment not pronounced

1 Fulfillment less pronounced

2 Fulfillment pronounced

3 Fulfillment very pronounced

problem, such as the influence of high initial horizontal stresses, were studied by Duncan and Dunlop (1968). In their finite element analyses, slopes were created by simulating the excavation of horizontal layers of material, and the distribution of maximum shear stresses was calculated. The material was assumed to be linearly elastic. The earth pressure coefficient, K, defined as the ratio of the total horizontal stress to total vertical stress, was the only parameter varied. A value of K larger than 1 is indicative of an overconsolidated clay. Figure 13 indicates the normalized maximum shear stresses for 1 on 3 slopes with K equal to 0.81 and 1.60. In addition to obtaining altogether different stress distributions for the two cases, the maximum shear stress near the toe of the slope, where progressive failure can readily commence, was approximately 10 times larger for the larger value of K. The higher stresses were so large that failure could be expected.

A method of demonstrating the applicability of residual strength in the progressive failure mechanism was proposed by Skempton (1964). He reasoned that it is not necessary for an entire slope to be failing progressively before actual failure occurs. The average shear strength over the failure surface can be compared to the peak and residual strengths to determine what portion of the total length of the slip surface was acting at the residual value. The amount by which the average strength along a slip surface fell from the peak to the residual strength was defined as the residual factor, R:

$$R = \frac{s_p - \bar{s}}{s_p - s_r}$$

where

$$s_p, s_r = \text{the peak and residual effective shear strengths, respectively}$$

$$\bar{s} = \text{the average effective shear strength acting at the time of failure}$$

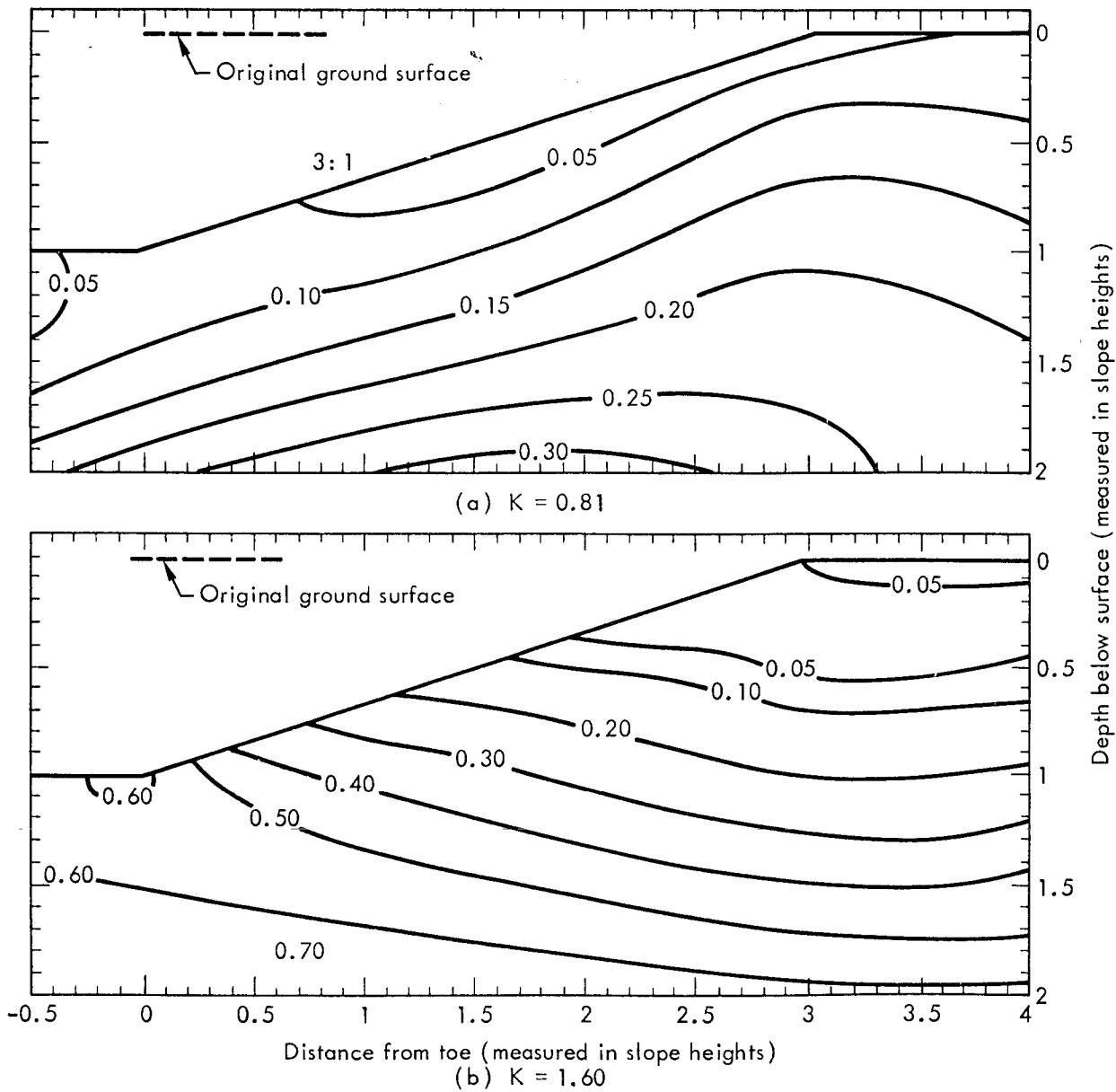


Fig. 13. Contours of normalized maximum shear stress, $\tau_{\max}/\gamma H$ adjacent to 3:1 slopes (Duncan and Dunlop, 1968).

In general, the residual factor varies from 0.0, when the average field shear strength equals the peak shear strength, to 1.0, when the average field shear strength equals the residual strength (Fig. 6). The residual factor was calculated for several slides and is included in Table 3. A residual factor of 0.08 was obtained by Skempton (1964) for a boulder clay (till) at Selset, indicating that the

average strength mobilized was very close to the peak strength. Other slope failures in highly overconsolidated clays had residual factors very close to unity, indicating that the average strength mobilized at failure was very close to the residual strength.

Geometry of Failures

Several types of failure geometries were observed in clay shale formations.

The different failure geometries are apparently caused by a complex of factors which include the effectiveness of different external agents and the applicability of different physical properties. Hutchinson (1967) recognized five different failure types in slopes of London clay. The failure types were classified as shallow rotational slips, successive or stepped rotational slips, undulations or soil waves, markedly noncircular shallow rotational slips, and predominantly translational slab slides. Hutchinson demonstrated a tendency for failures on steeper slopes (13 to 20 deg) to be primarily of the shallow rotational type and for failures on the flatter slopes (8 to 12 deg) to be translational slab slides. The data and inferred failure geometries are shown in Fig. 14. None of his inferred geometries are of the deep-seated rotational type.

Most attention to slope failures in clay shale has been concentrated on large-scale failures. Deformations such as soil undulations and small rotational slips have not been thoroughly investigated. The documented failures appear to fall into two general categories. In the first, the geometry appears to be structurally or stratigraphically controlled and most commonly consists of a steep head scarp and a long, slightly curved or planar failure base that developed along a discontinuity or weak seam. The initiation of movement removes lateral support at the head scarp, which fails; through repeated failures, the scarp migrates progressively up the slope. Several case histories of this failure geometry are documented in Appendixes D and E. The second principal failure geometry is much like the translational slab slide of Hutchinson (1967).

Sliding appears to occur in a zone near the base of the weathered shale. This zone, according to Bjerrum (1967), is the region of active swelling and destruction of diagenetic bonds. Residual stresses, loss of strength, and perhaps high pore pressures contribute to the mass failure. Failures of this type appear to be most commonly associated with more homogeneous deposits where structural features are generally lacking.

Creep

Creep in clay shale slopes is a recognized phenomenon but has received little attention and study. Gould (1960) described the results of a study of a number of slopes in California involving highly overconsolidated marine clays of Tertiary age with relatively strong bonds. Bjerrum (1967), in discussing the mechanism of creep, acknowledged that there were possibly a variety of different types of creep in clay slopes but distinguished two types:

1. Movements which occur in the zone to which frost penetrates (solifluction)
2. Movements seated at a greater depth

Bjerrum surmised that the deep-seated movements were more pronounced for increasing stiffness and hardness of the parent clay and were related closely to the geological history of the parent material. He suggested that creep was related to slow volumetric expansion accompanying disintegration of heavily overconsolidated clays and shales containing large amounts of stored strain energy. The continuous increase in lateral shear stresses and the simultaneous reduction in shear strength with increasing

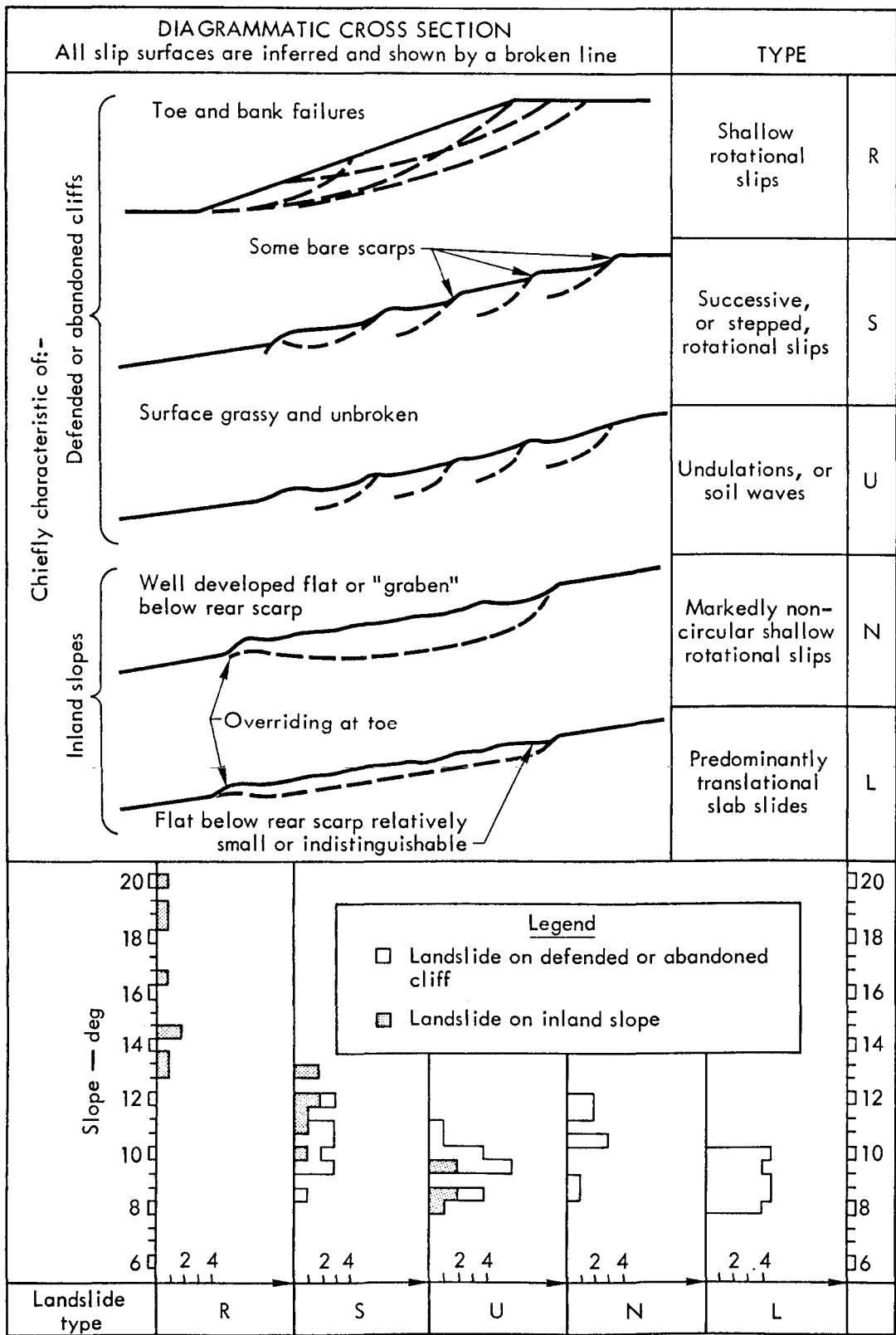


Fig. 14. Main types of landslides in London clay and their incidence by slope angle (Hutchinson, 1967).

water content are important contributing factors to the creep phenomenon. If a slope is so steep that the residual shear strength is less than the shear stresses, creep can lead to a slide such as described under the topic of progressive failure. The slide exposes fresh surfaces and allows the process to start again, thus allowing a slope to be "worked down." If, on the other hand, the shear stresses equal the residual shear strength, the slope will experience a steady slow creep but the movements will not lead to a slide. This description suggests that creep may occur as secondary movements after a continuous failure zone has been formed by progressive failure. For slopes flat enough for the shear stresses to be less than the residual shear strength no gravity creep will occur, although at local points of weakness there could occur some lateral movements caused by internal stresses.

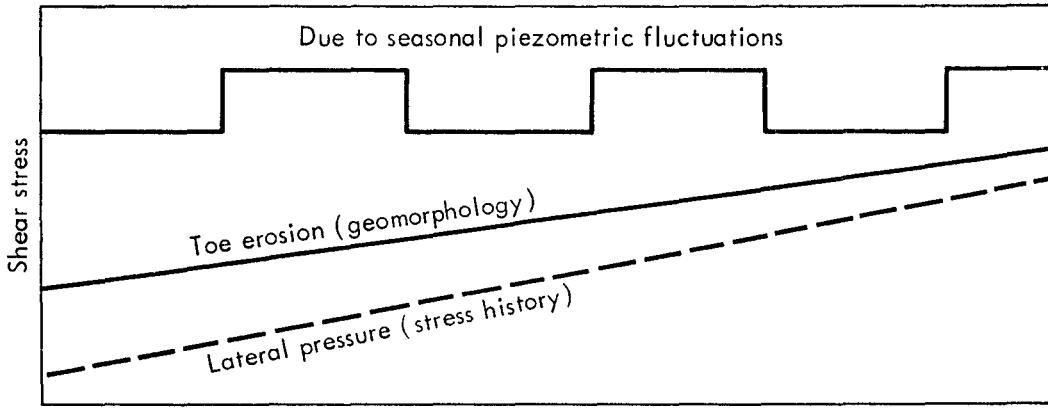
TIME FACTOR IN CLAY SHALE SLOPE FAILURE

Several examples of slopes and embankments constructed on clay shales which failed after varying lengths of time were cited in Table 4. The previous discussions have indicated that factors such as consolidation, creation of diagenetic bonds, erosion, and weathering contribute to clay shale slope behavior. All of these factors are time-dependent. Scott and Brooker (1968) showed diagrammatically how the shear stresses and shear resistance might vary with time (Fig. 15). In the construction of a slope or embankment in clay shales, the initial conditions play a very important part in determining when and if failure can reasonably be expected. For instance, if an embankment is being

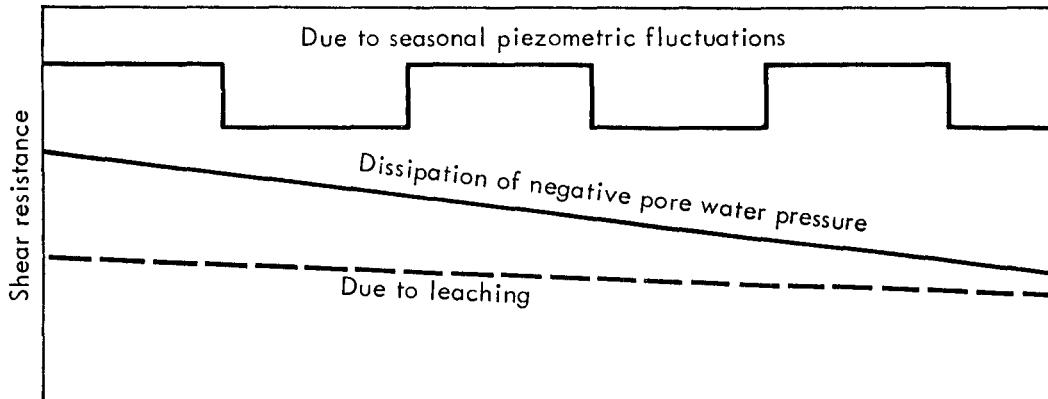
constructed on a clay shale, failure may occur during the construction stage if the mass is at or near the residual strength. Thus failure becomes a function of loading or embankment height. On the other hand, if the mass strength is appreciably above the residual strength, the change of loads caused by the construction of an embankment of the creation of a slope may not be sufficient to cause immediate failure, and a varying amount of time will be required to bring the factors into the conditions which could lead to a failure.

If a slope failure has already occurred, subsequent movement along the slip surface will be controlled by the residual strength, no matter what clay type is involved (Skempton, 1964). Skempton and Petley (1967) cite an example (Weald clay) where no movements had occurred for at least 10,000 years, yet tests on material taken from slip surfaces indicated that the strength was at or very near the residual shear strength. Thus it can be concluded that when shear stresses in masses which have previously been sheared or failed are increased, failures can be expected almost immediately.

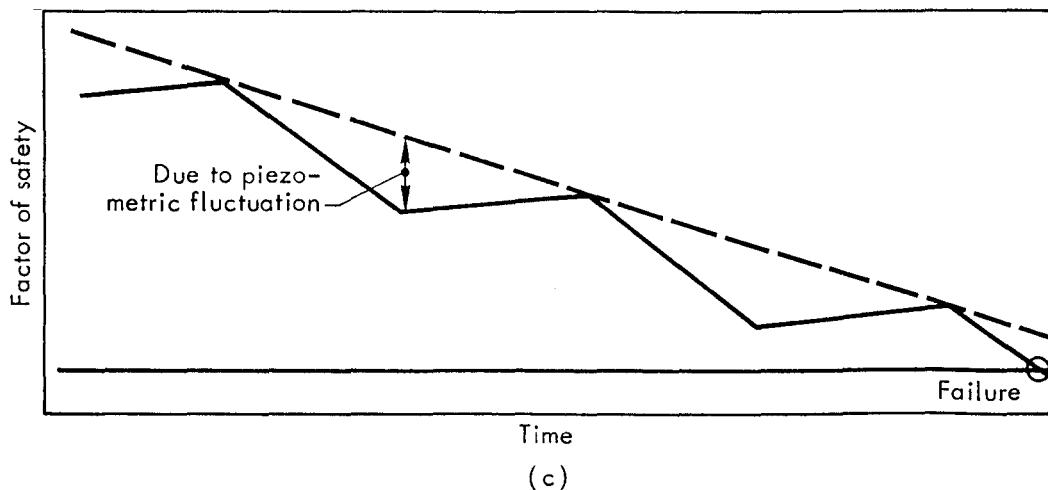
Other examples were cited where failure occurred some time after construction was completed (Table 4). The Jackfield slide (Skempton, 1964) is an example of a failure of a natural slope where natural processes created a slope incompatible with the allowable internal balances of stresses and strength. At the completion of construction of a slope or embankment, a certain gravity loading exists within the material. With time, the internal stresses increase and at the same time the shear resistance decreases. Skempton (1964) indicated that it was possible for



(a)



(b)



(c)

Influence of geologic factors on:

- (a) Shearing stress in a soil mass
- (b) Shearing resistance of the mass
- (c) Factor of safety against failure

Fig. 15. Variation in shear stress, shear resistance, and factor of safety with time (Scott and Brooker, 1968).

failures to occur when the average stress was somewhat larger than the residual

strength of material. From Skempton's analyses of slides in London clay it is

possible to plot the increase in residual factor (i.e., decrease in strength) as a function of time (Morgenstern, 1967). Corroborating data are available from the East and West Culebra slides at the Panama Canal (Hirshfeld, Whitman, and Wolfskill, 1965). Static stability analyses of the Swedish circle type were made of the canal slopes as they existed in January 1912, June 1912, July 1915, and March 1947. Prior to January 1912, the area was free of major slides. The time decay of shear strength for both the London clay and Culebra slide material is illustrated in Fig. 16 (Johnson, 1969). The strengths used for this plot are those determined from circular-arc analyses for a factor of safety of unity at the times indicated above. Collectively, the strength data indicate a general decrease with time. It may be further reasoned that for a given set of conditions, the steeper the slope (i.e., the more closely the gravity loading stresses approach the peak strength), the shorter the time to failure. If the slope were sufficiently flat (i.e., approaching the angle of residual shear strength) and depending on initial conditions, failure

would occur, if at all, at some longer period of time than that represented in the figure.

The role of groundwater (including pore pressures) is very important when considering the time behavior of a slope in clay shale (see Skempton and Hutchinson, 1969). The removal of the load either by excavation or by erosion initiates softening of the material over a long period of time. The reduction of load is followed by lateral expansion with an opening of fissures and an increase in the mass permeability. With the open fissures providing avenues for groundwater movement, softening starts, ultimately leading to conditions in which the average shear strength of the mass is greatly reduced. The reduction in loads initially leads to a reduction in pore pressures within the mass. With the passage of time these pore pressures continually adjust until eventually they are in equilibrium with the steady seepage flow pattern appropriate to the new slope profile. The increase of pore pressures to equilibrium conditions results in a decrease in effective stresses such that failure can occur some time after the change in loading.

Section 3. Discussion of Data Obtained About Clay Shale Slopes Along Upper Missouri River

INTRODUCTION

Several of the geologic formations of the Northern Great Plains are clay shales with varying reputations regarding slope stability. The five geologic units studied in the field were the Marias River formation of the Colorado group, the Claggett formation, the Bearpaw formation, three

formations in the Fort Union group, and the Pierre formation. Of these materials, the Colorado and Fort Union are least troublesome in slopes. Slides are conspicuous features in the Claggett, Bearpaw, and Pierre formations. Since the materials behave somewhat differently in the field, this investigation was guided toward determining measurable differences in such

Table 6. Lithologic description of materials studied

Unit	Approximate thickness (ft)	Description at surface	Source
Fort Union group			
Sentinel Butte formation	1150-1300	As below, except slightly, more coarse-grained overall and containing fewer lignites.	Hearn, et al. (1964)
Tongue River formation		Fine-grained sands, shaly clay, lignites, and some bentonite clays.	
Cannonball formation		Alternating clays, shales, silts, and sands.	
Pierre formation	800	Variable (see Fig. A25, Vol. 2). In general, gray bentonitic shale with scattered marly and concretionary zones.	
Montana group			
Bearpaw formation	1200	Thick unit of medium to dark gray, soft, poorly fissile clay shale. Bentonite disseminated throughout and also occurs in innumerable thin layers. Contains numerous calcareous and clay-ironstone concretions.	Erdmann (1962)
Claggett formation	450-550	Homogeneous sequence of gray to brownish-gray fissile flaky shale and bentonitic shale. Contains several bentonite layers in the lower 100 ft; sandy near top, lower 20 ft may contain scattered pebbles.	Hearn, et al. (1964)
Colorado group			
Marias River formation	800	Medium to dark-gray, silty shale, and moderately fissile siltstones containing scattered thin layers of bentonite and calcareous concretionary zones.	Erdmann (1962)

features as natural slope geometry, strength, mineralogy, plasticity, lithology, and consolidation characteristics. The study related property differences to differences in slope behavior in an effort to better understand the factors that lead to failures.

Much more data were obtained than can be discussed in this volume of the report; only those aspects which seem pertinent to the overall behavior of the five formations studied are treated here. More detailed discussions of particular features are given in Volume 2.

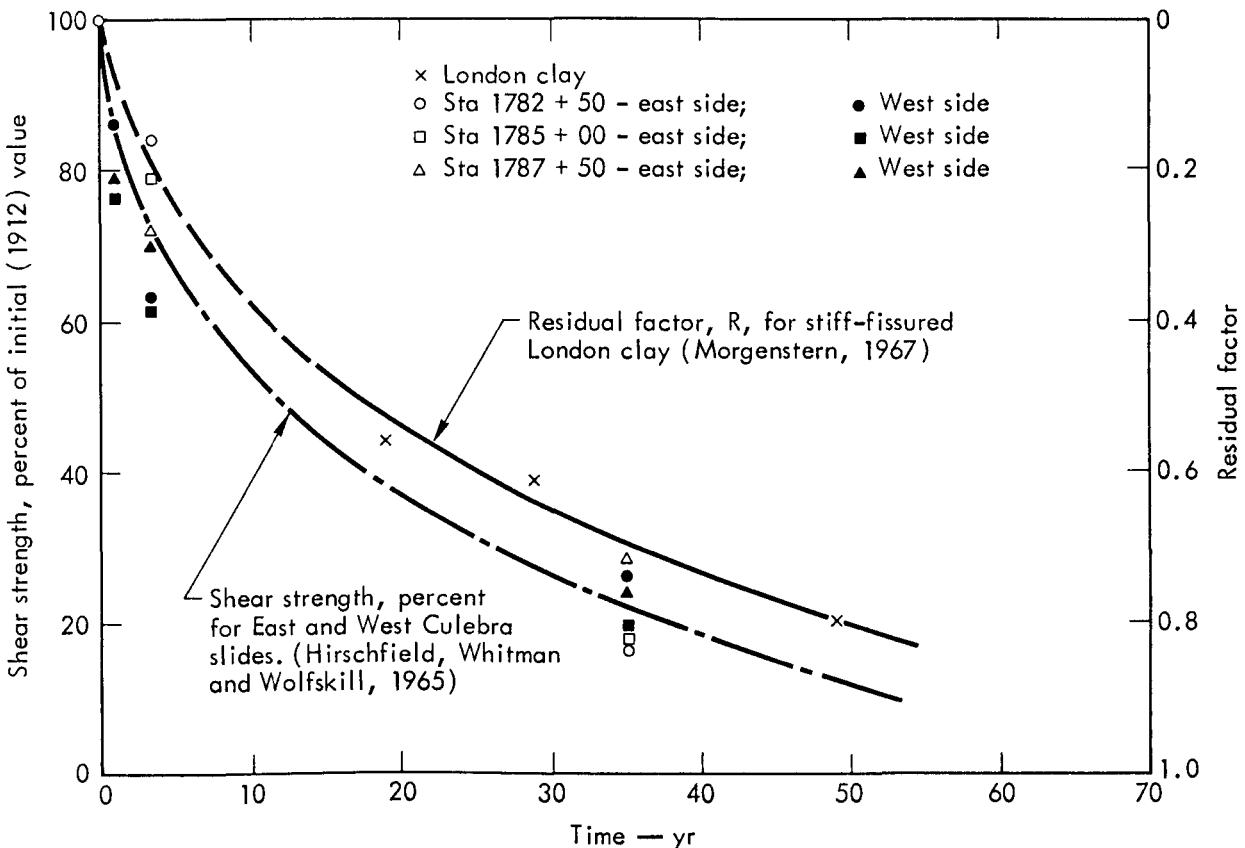


Fig. 16. Rate of progressive failure of London clay and Culebra slides (Johnson, 1969).

GEOLOGIC DATA

Stratigraphy and Environment of Deposition

As shown schematically in Fig. 17, the five materials studied are exposed along a 600-mi reach of the Missouri River from Fort Benton, Montana, to Chamberlain, South Dakota. The Colorado, Claggett, Bearpaw, and Pierre are all marine deposits of Upper Cretaceous age. The Fort Union group, composed of four formations, three nonmarine and one marine, is Paleocene (Lower Tertiary) in age. General stratigraphic relationships across Montana and the Dakotas are given in Fig. 18. Brief lithologic descriptions are given in Table 6.

Colorado Group, Claggett, Bearpaw, and Pierre Formations. The Colorado group is the oldest material studied. At the Fort Benton site, the Colorado group is composed of clay shale and alternating layers of clay shale and cemented siltstone. The Niobrara chalk, which underlies the Pierre formation at Chamberlain, South Dakota (about 60 mi south of Pierre), is correlated stratigraphically with the Colorado shale in Montana (Fig. 18). The Bearpaw and Claggett formations correlate with the Pierre. The lithologies of the Bearpaw, Claggett, and Pierre are similar. All were deposited in a marine environment and consist largely of clay shale with scattered bentonite layers and bentonitic zones. The

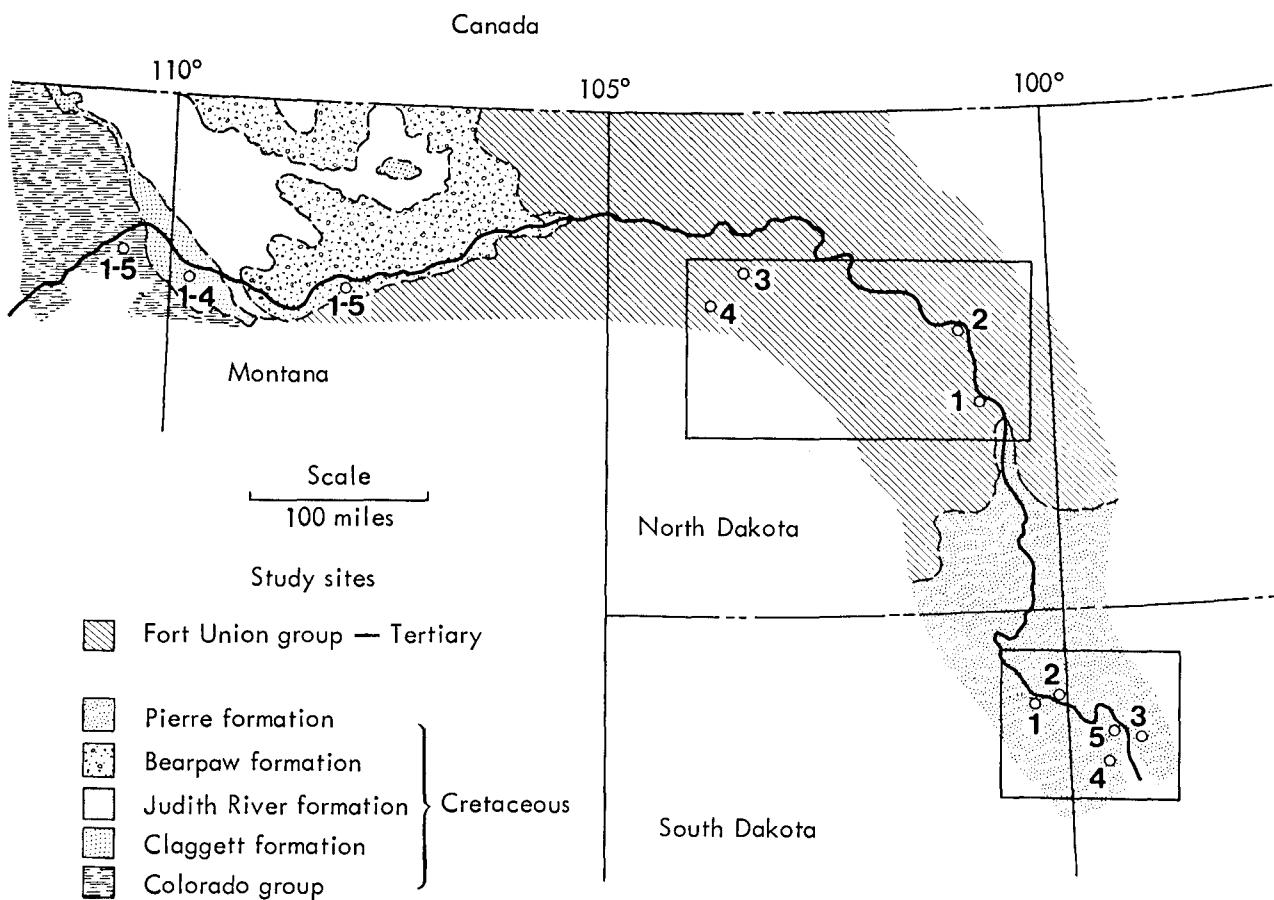


Fig. 17. Generalized geologic map showing study sites for detailed investigation.

Pierre contains some calcareous or marly layers in the lower and upper parts. The lower portion of the Bearpaw formation and both the upper and lower portions of the Claggett formation are sandy. The Judith River sandstone (nonmarine), sandwiched between the Bearpaw and Claggett (Figs. 17 and 18), reflects a temporary retreat of the Upper Cretaceous sea.

Fort Union Group. The Fort Union Group contains, from oldest to youngest, the Ludlow, Cannonball, Tongue River, and Sentinel Butte formations. The Ludlow formation was not investigated. The Cannonball is a marine shale with characteristics similar to the Bearpaw, Claggett,

and Pierre formations. Although the Cannonball is less bentonitic and more coarse-grained than the other formations, it does create slope problems in the Bismarck, North Dakota, area. The Tongue River and Sentinel Butte formations are both nonmarine deposits of clays, silts, sands, and lignite. In detail, the Sentinel Butte is somewhat more coarse-grained overall and contains less lignite than the Tongue River (Laird, 1956). Apparently, a larger portion of the Sentinel Butte was deposited by streams, while the more fine-grained, more lignitic Tongue River was deposited primarily in swamps and lakes. The differences in lithology are reflected in slope behavior; natural slopes in the Sentinel Butte formation are significantly

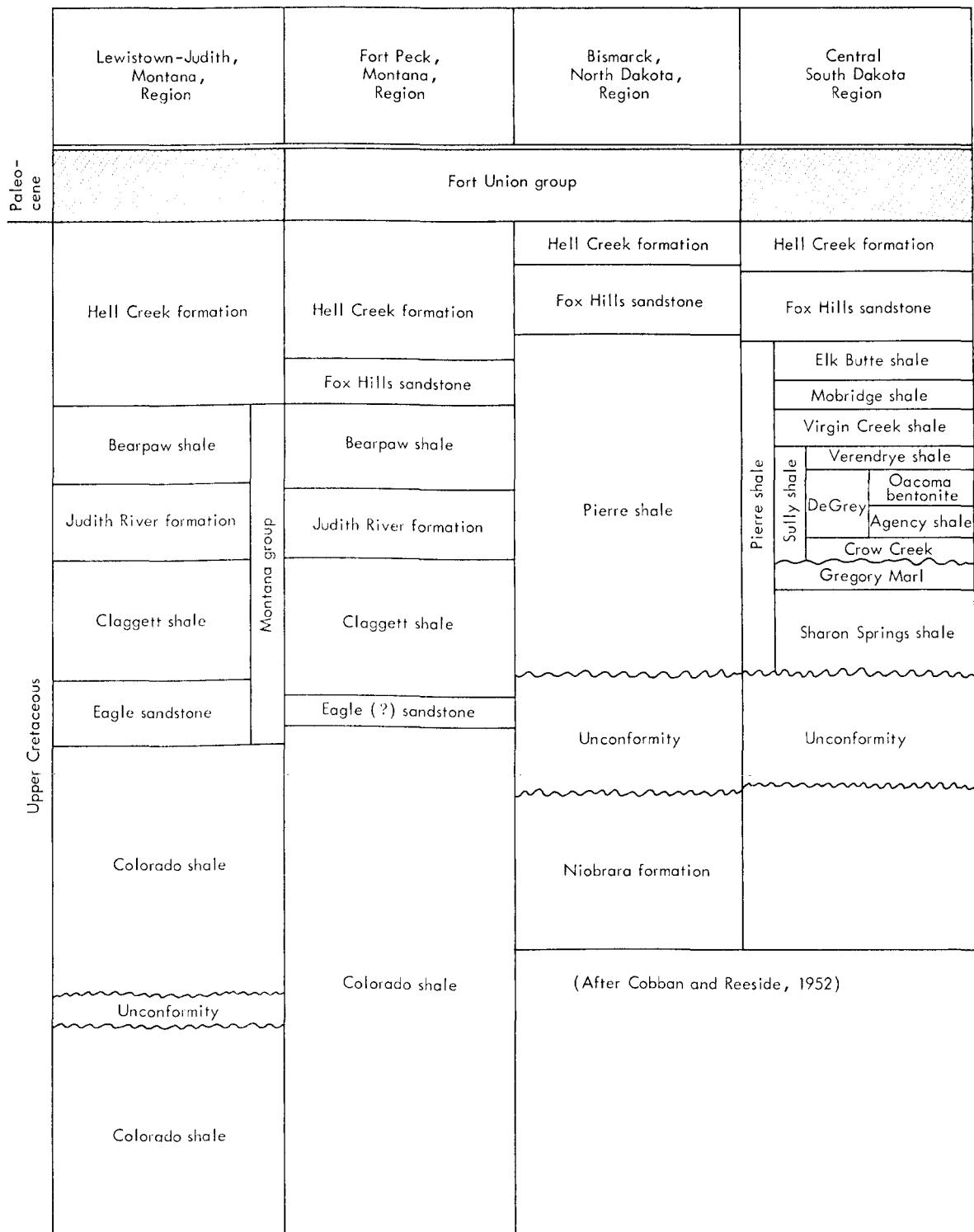


Fig. 18. Stratigraphic correlation chart.

higher and steeper than those in the Tongue River.

Structure

The sites investigated were selected to avoid the complication of large-scale structural features, such as steeply dipping beds or major faulting. Tilting of bedding and disruption of stratigraphic marker horizons at the sites are caused by slope failures, which were extensive in most areas. The five formations exhibited strikingly different slope geometries, which appeared to reflect susceptibility of the materials to failure and mode of failure. Aerial and ground photographs of the slopes are included in Appendix A, Volume 2.

The sites are all within the Williston Basin, a deep, broad depositional basin containing more than 15,000 ft of sediments ranging in age from Cambrian to Tertiary (Laird, 1956). In general, regional dips are gentle toward the center of the basin, except where crustal disturbance has altered the attitudes. Dips in the Colorado group near Fort Benton, Montana, are on the order of a few feet per mile (Lindvall, 1956; 1956a). The Claggett and Bearpaw formations (Erdmann, 1962) at the study sites dip gently southeastward away from the Bearpaw Mountains. The Fort Union group in North Dakota (Laird, 1956), and the Pierre formation at Chamberlain, South Dakota (Warren and Crandell, 1952), are nearly horizontal.

Colorado Group. Slopes in the Colorado group were relatively unfailed, and individual marker beds such as bentonite layers could be traced for hundreds to thousands of feet. Where slope failures

were present, sliding appeared to have occurred along a single plane of circular-to-composite-circular geometry. Individual failed masses were distinct and adjacent materials were unfailed.

Claggett, Bearpaw, and Pierre Formations. Slopes in the Claggett, Bearpaw, and Pierre formations were similar. All the slopes along the Missouri River in these materials were failed. (One slope in the Pierre formation, along the White River, was unfailed.) Failure apparently was coincident with the downcutting of the present Missouri River, and massive slumping may have been the dominant mechanism for valley widening. Erosion modified the slopes in such a way that the very old failures could not be recognized as individual slide masses. However, an excavation into intact material, such as a road cut, usually exposed several "faults" which are portions of the old failure planes. Detailed exploration of the slope failures behind the powerhouse for Oahe Dam (Pierre formation) revealed many of these discontinuities, and the structure and orientation of the zones could be reconstructed only by extensive drilling. A cross section through the left abutment showing the major failed zones is given in Fig. D30, Volume 2. Slopes in the vicinity of Fort Peck Dam were similarly failed (see Fig. D14, Volume 2).

Further movement of the valley walls in more recent time developed a characteristic hummocky topography. These movements appeared restricted to the weathered zone; i.e., approximately the upper 50 ft of material. Active failures were most commonly associated with (1) construction activity where renewed

slumping occurs, at least in part, along pre-existing failure planes, (2) local oversteepening of the slope by the development of lateral drainage on the valley walls or a river meander impinging on the toe of the slope, or (3) filling and pool regulation of reservoirs. These failures may be block slides with planar or composite circular failure surfaces, flow slides, or combinations of these.

Fort Union Group. The distribution and occurrence of slope failures in the Fort Union group were different for the three formations studied. The Cannonball formation, of shallow marine origin, was largely shale, with some sand and silt. Slides were common in the Cannonball along the Missouri River valley, but the slopes did not appear to be failed throughout, nor failed as deeply as in the other marine formations studied (Claggett, Bearpaw, and Pierre). On the west-facing bank of the Missouri River near Bismarck, North Dakota, there were numerous small landslides. Most appeared to be superficial; sliding took place in the weathered zone along a relatively planar surface that nearly paralleled the ground surface.

The Tongue River formation was studied at Garrison Dam, North Dakota, where heights and inclinations of slopes in the vicinity were successfully used to design construction slopes for the project (Lane, 1961). Natural slope data could be used directly as a design tool because the slopes were not failed throughout as were slopes in the Pierre, Bearpaw, and Claggett. There was no intricate network of "faults" like those at Oahe Dam (Knight, 1963), to complicate the assessment. Landslides and unfailed slopes in the Tongue River

formation were studied at the South Unit of Theodore Roosevelt National Memorial Park, North Dakota. There, the incidence of slope failures was strongly influenced by slope exposure. Most of the north-facing slopes were failed, and juniper trees and scrub brush grew in the areas of ponded drainage at the back of the slide masses. South-facing slopes contained markedly fewer slides, and vegetation was mainly limited to scrub brush and grasses. Generally, outcrops of the failed materials were insufficient to estimate accurately the depth of failure or the geometry of the slide surfaces. The surface features of one landslide at Medora, North Dakota, near the entrance to the south unit of the park (Plate A-XVIII, Volume 2), suggested a deep-seated circular slip surface. The beds exposed in the slide mass were dipping into the slope at 20 to 30 deg, while adjacent unfailed materials were horizontal.

The Sentinel Butte formation of the Fort Union group was mapped and studied at the North Unit of Theodore Roosevelt National Memorial Park. There appeared to be more failures in the north-facing slopes, but landslides were common all along the Little Missouri River in that area. The configuration of those failures was different from any other studied. The natural slopes were composite, with a high, relatively steep backslope, sloping at about 20 deg, and an accumulation of slide debris from several slides, with a slope angle of about 3 deg at the toe. The backslopes were unfailed (see Plate A-XVII, Volume 2), and persistent beds such as the Big Blue bentonite could be traced for many miles. The failed masses appeared to have moved downslope as large blocks, sliding along weak zones, usually the Big

Blue bentonite layer. One of these slopes near the park was selected for more detailed study, and one boring was placed at the top of the slope. References to material properties in the Fort Union group in subsequent discussion will refer to the Sentinel Butte formation unless otherwise indicated.

History of Slope Development

The present valley of the Missouri River across Montana and the Dakotas was formed during the last ice age. The age of the slopes, rate of river downcutting, and maximum height and inclination of the slopes are imperfectly known, but some estimates can be made. The glacial history of the Northern Great Plains has been summarized by Lemke, Laird, Tipton, and Lindval (1965). The four continental glacial stages recognized in the United States are, from the oldest, the Nebraskan, Kansan, Illinoian, and Wisconsin. The formation of the Missouri River in the study area began in the Illinoian stage. Prior to Illinoian time, drainage was to the north. The ancestral Missouri River in Montana flowed across an area now occupied by the Milk River, eventually emptying into Hudson Bay. Drainage in North Dakota flowed either northerly to the ancestral Missouri River or east to the Red River which also drained, as it does today, to Hudson Bay. The northernmost streams in South Dakota drained northeasterly to the Red River. Streams in central and southern South Dakota flowed eastward to the south-flowing James River. According to one theory (Warren, 1952) the advancing Illinoian glacier blocked the James River in southeastern South Dakota, perhaps 100,000 or more years

ago, and a new channel was formed in what is now the Missouri River valley. However, White (1964) has suggested that the present channel in southern South Dakota did not form until early Wisconsin time, perhaps 30,000 to 70,000 yr ago. At any rate, the Missouri River valley, from the Chamberlain vicinity in South Dakota to the south, appears somewhat older than the rest of the valley.

No glacial deposits of definite pre-Wisconsin age have been found in either Montana or North Dakota. The Missouri River valley in these states is definitely of Wisconsin age. Six separate glacial advances and retreats have been recognized in the Wisconsin, the first and most extensive blocking the north-flowing streams, principally the Milk River and the Red River. The limits of that advance are shown in Fig. 19, the advance extending slightly beyond the present channel of the Missouri River along most of the valley. The absence of till in the present valley except near Chamberlain, South Dakota, is evidence that the valley was formed after the first ice advance. Downcutting of the present river valley apparently began after the retreat of the first advance, about 25,000 yr ago.

The second ice advance did not reach as far as the first but did extend slightly beyond the present channel of the Missouri River roughly along the reach between Fort Peck and Garrison Dams and in the Fort Benton, Montana, area. The last four advances in the Wisconsin stage did not reach the present channel of the river.

Foundation explorations for bridges and dam sites along the Missouri River and some tributaries revealed that the

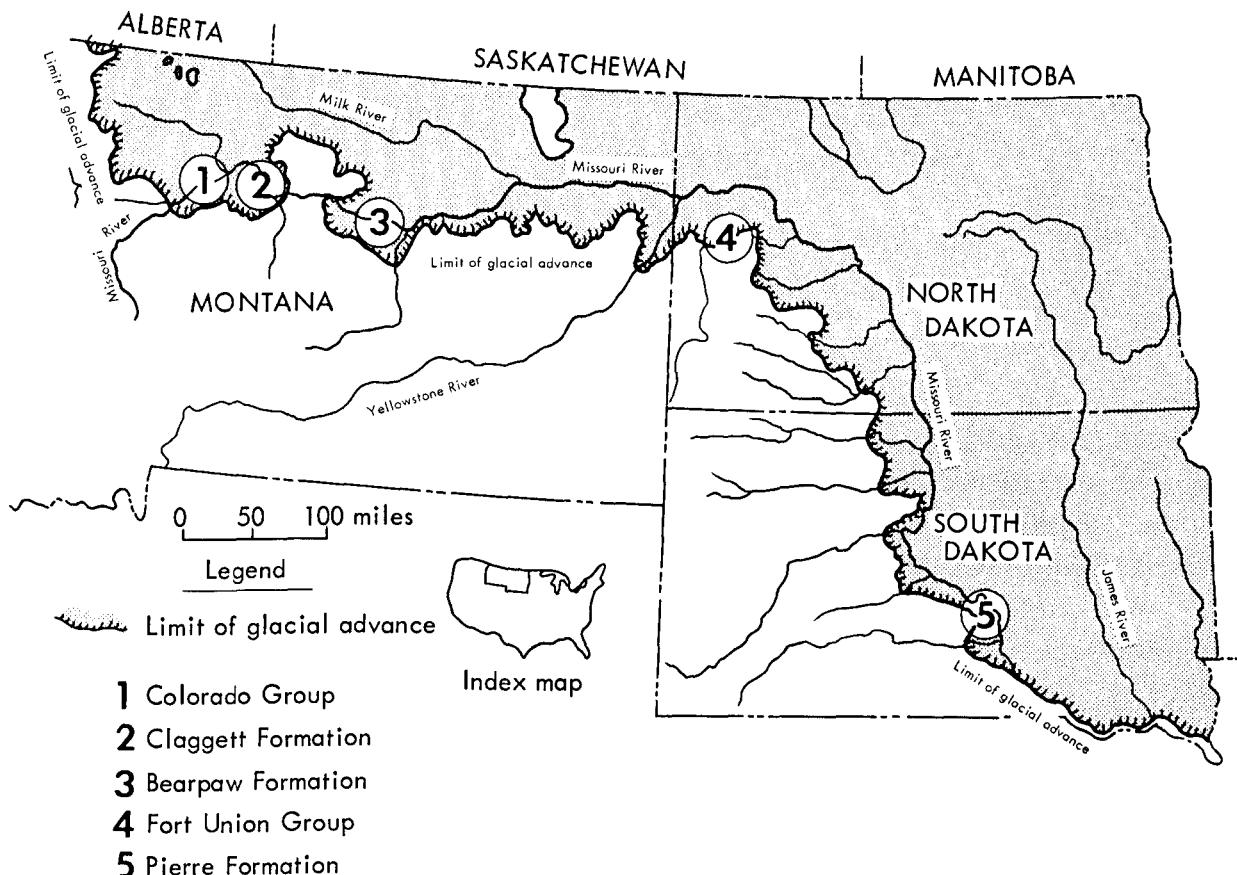


Fig. 19. Limits of first Wisconsin Glacial Advance (Lemke et al., 1965).

river channel has been scoured to depths of 75 to 90 ft below the present bottom elevation and subsequently filled with alluvium. Data on the depth of scour from the five areas closest to sites drilled as part of this study are given in Table 7. They are minimum values, because the thickness of alluvium is that actually encountered in a boring, and drilling was not extensive enough to be certain of the maximum depth.

The Milk River in Montana does not have an alluvium-filled channel.* Since the Milk River roughly parallels the present Missouri River and empties into the Missouri River near Fort Peck, it

seems reasonable that the deep scouring and aggradation along the Missouri River occurred while the Milk River valley was covered by ice. The third and succeeding advances did not reach the Milk River, so downcutting and aggrading were accomplished before the second advance retreated north of the Milk River. Therefore, it appears that all the downcutting occurred after the retreat of the first Wisconsin ice sheet and before the second sheet retreated beyond the Milk River. A few radiocarbon dates have been obtained which allow the time of downcutting to be estimated. Ages reported by Frye and Willman (1963) for glaciation in Illinois can be approximately correlated to the advances described by Lemke et al. (1965) in Montana and the

* Personal communication (F.A. Nickell).

Table 7. Depth of buried channel along Missouri River

Material	Site	Depth of Scour (ft)	Source
Colorado	Fort Benton Damsite	85	U.S. Bureau of Reclamation ^a
Claggett	Illiad Damsite	75	U.S. Bureau of Reclamation ^a
Bearpaw	Rocky Point Damsite	95	U.S. Bureau of Reclamation ^a
Fort Union	Highway 95 Bridge	85	North Dakota Highway Dept. ^a
Pierre	Chamberlain	90	Warren and Crandell, (1952)

^aFile data obtained through courtesy of cited agency.

Dakotas (Fig. 20). It appears that the downcutting and 75 to 95 ft of aggradation were accomplished in the fairly short period of a few thousand years, and that the total relief of the valley is now about the same as it was 20,000 yr ago. However, the inclination of the slopes, particularly in the Claggett, Bearpaw, and Pierre, must have been steeper than at present and the massive slumping that developed the network of "faults" in the valley walls occurred during that time. Stability of high-slopes in those materials beneath the weathered zone now appears largely dependent on the orientation and extent of the network of discontinuities.

The slopes in the Colorado and Fort Union groups are not failed throughout, nor to large distances back into the slopes. The stability of unfailed portions of these groups is more dependent on the long-term strength of intact materials as contrasted with long-term strength along discontinuities. Frequency of slope failures in these materials on an areal basis could probably be related to higher-plasticity zones at critical locations in the slope, but insufficient data are available to evaluate that possibility. Inclination of natural slopes in the Pierre was related to the montmorillonite content of the various

members of the formation by Erskine (1965) (Fig. E12, Volume 2). The Colorado and Fort Union have a wider range in montmorillonite content than the Pierre, and a stronger correlation could be expected.

Consolidation History—Geologic Data

All the materials studied were consolidated under loads in excess of their present overburden load. The importance of overconsolidation in the long-term stability of clay shale slopes was discussed in Section 2. The clay shales along the upper Missouri River were loaded by sediments which have been eroded, and in some cases again loaded by glacial ice. The preconsolidation load imposed by sediments at all the sites was estimated from available geologic and topographic maps by determining the youngest nearby formations present either as isolated erosional remnants or as down-faulted blocks. The thicknesses of the formations that appear to have been removed were projected to the study sites and the load estimated. The results, as summarized in Table 8, are highly speculative, but they do provide a general indication of the geologic preconsolidation load. The load imposed by a layer of material with the

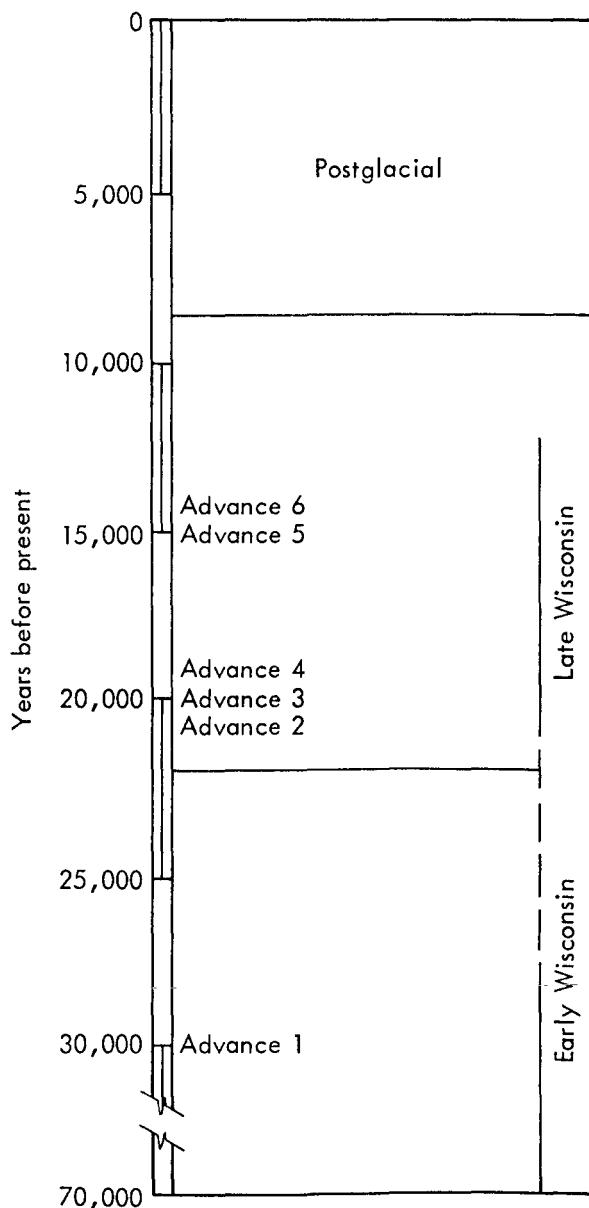


Fig. 20. Absolute time scale for Wisconsin Glacial Stage in Northern Great Plains (Lemke et al., 1965).

density of a clay shale, assuming submerged unit weight, was calculated and compared to the preconsolidation load determined from consolidation test data.

For deeper specimens, the laboratory-derived preconsolidation load is greater than that suggested by the geologic evidence.

However, an assumption that buoyant forces acted to some intermediate depth only, thereby increasing the unit weight, increases the calculated load considerably. Further, Kenney, Moum, and Berre (1967) showed that the preconsolidation pressure from a consolidation test was dependent on the composition of the pore fluid and cementing compounds. For example, the leaching of 0.8% of iron from a sensitive Labrador clay reduced the apparent preconsolidation pressure from 4.6 to 1.9 tons/ ft^2 . The data of Kenney et al., as well as the results of this study, suggested that preconsolidation loads determined from consolidation tests on clay shales were dependent on other factors in addition to the maximum past load. In general, however, it appears that four of the five materials (Colorado, Claggett, Bearpaw, and Pierre) were consolidated by loads of about a hundred tons/ ft^2 or more. The Sentinel Butte formation of the Fort Union group apparently was consolidated by a load of 70 tons/ ft^2 .

The load from glacial ice sheets at the sites drilled apparently was less than the load applied by sediments (Table 9). The thickness of ice at the five sites was estimated by assuming that the face of the glacier was 200 ft high, and the slope of the top of the ice was about 1%. The first ice advance did not leave a large terminal moraine, and the assumption of a 200-ft face is probably somewhat high. The values given are probably reflective of maximum thicknesses. The Colorado group site was covered by two advances; the first was apparently the thickest. The site in the Fort Union group was not glaciated.

Table 8. Comparison of estimated preconsolidation load and calculated preconsolidation load.

Material	Estimated thickness of material removed from top of borings DH-1 (ft)	Estimated preconsolidation load at sample depth (tons/ft ²)	Preconsolidation load calculated from consolidation test results ^a (tons/ft ²)
Colorado	1900	85	120 ^b
Claggett	1800	85	95
Bearpaw	2200	105	>100 ^b
Fort Union	950	35	70
Pierre	600	30	105 ^b

^aAverage from deeper specimens loaded normal to bedding.

^bInterpretation of data is approximate due to poorly defined e-log p curves.

Table 9. Estimated thickness of glacial ice cover

Material	Estimated maximum thickness of ice cover (ft)	Load (tons/ft ²)
Colorado group	1200	40
Claggett formation	500	15
Bearpaw formation	1200	40
Fort Union group	0	0
Pierre formation	700	25

LABORATORY DATA

Consolidation and Rebound

Results of consolidation-swell tests and shrinkage measurements of oriented specimens from the five principal formations are presented and discussed in Volume 2, Appendix C. Consolidation tests were performed on deep samples loaded both normal and parallel to bedding and on at least one shallow sample loaded normal to bedding.

Swell Pressure

The swell pressure was determined in the consolidometer by adding increments

of load to establish the point at which the specimen tended neither to swell nor to consolidate. Although it was not possible to define the swell pressure exactly by this method, the swell pressure could usually be bracketed within fairly narrow limits. The swell pressures and consolidation test results are shown in Figs. C13 through C17 (Volume 2); the data are summarized in Table 10.

The swell pressure measured parallel to bedding was in all cases less than that measured normal to bedding. The ratio of swell pressure parallel to bedding to that normal to bedding was 0.26 to 0.33 for the Colorado group, less than 0.91 for

Table 10. Summary of consolidation and swell test data.

Formation and sample no.	Depth (ft)	Initial moisture (%)	Atterberg limits		Range in swell pressure (tons/ft ²)	Swell pressure ratio ^a	Compression index	Percent thickness change from initial thickness		Time under rebound load (min)	Apparent preconsolidation load from e - log p curve
			LL	PL				Under 350 (tons/ft ²)	Under 0.1 (tons/ft ²)		
Colorado											
DH-1 U-7 ^b	100	19.6	171	24	5.7 to 6.8			18.3	24.8 ^c	11,765	—
DH-1 U-24 ^b	234	6.9	42	20	20.4 to 21.5	0.26 to 0.33		4.7	12.1 ^c	41,000	110-130 ^d
DH-1 U-24 ^e	234	6.5	42	20	5.7 to 6.8			3.1	5.0 ^c	8,530	115-125 ^d
Claggett											
DH-1 U-40 ^b	491	26.2	162	50	29.5 to 31.7		0.09	3.5	10.3 ^c	17,220	90-100 ^d
DH-1 U-50 ^b	519	11.1	104	22	19.3 to 20.4		0.07 ^d	4.9	3.2 ^c	5,715	90-100 ^d
DH-1 U-50 ^e	519	11.4	104	22	Less than 17	Less than 0.91	0.08 ^d	3.2	1.8 ^c	10,035	120-140 ^d
DH-2 U-3 ^b	88	20.6	72	22	3.3 to 4.5		0.16	14.4	3.3	20,215	15-18
Bearpaw											
DH-1 U-46 ^b	401	11.6	81	21	15.9 to 20.4			4.3	6.7 ^c	18,650	>100 ^d
DH-1 U-46 ^e	401	11.6	81	21	7.9 to 15.9	0.39 to 1.0		4.3	5.7 ^c	22,570	>150 ^d
DH-2 U-2 ^b	76	18.3	114	24	4.5 to 6.8		0.10	9.8	0.5	—	20-25
Port Union											
DH-1 U-10 ^b	82	19.0	99	20	4.4 to 5.4		0.26	15.8	1.5	8,580	50-60
DH-1 U-22 ^b	157	23.8	133	22	10.2 to 11.2	0.50 to 0.66	0.34	18.3	21.6	10,000	65-75
DH-1 U-22 ^e	157	24.2	133	22	5.6 to 6.8		0.31	17.8	32.5	8,065	55-60
Pierre											
DH-1 U-2 ^b	52	29.4	162	28	3.4 to 4.5		0.37	20.0	7.8 ^c	11,445	50-65
DH-1 U-20 ^b	216	25.5	148	35	12.5 to 13.6	Less than 0.36	0.20 ^d	9.2	6.1 ^c	10,020	100-110 ^d
DH-1 U-20 ^e	216	26.3	148	35	Less than 4.5		0.23	7.8	7.5	16,680	110-120 ^d

Notes:

^aRatio of swell pressure measured parallel and normal to bedding.

^bSpecimen oriented with consolidation load applied normal to bedding.

^cTest was terminated before sufficient time had elapsed to define completion of swell.

^dInterpretation of data is approximate due to poorly defined e - log p curves.

^eSpecimen oriented with consolidation load applied parallel to bedding.

the Claggett formation, 0.39 to 1.0 for the Bearpaw formation, 0.50 to 0.66 for the Fort Union group, and less than 0.36 for the Pierre formation. The swell pressure parallel to bedding was not well bracketed for the Claggett or Bearpaw, but it appears that the upper limit of the ratio is too high compared to the Pierre and Colorado materials. The swell pressure measured normal to bedding from shallower samples (less than 100 ft) was consistently less than that from the deeper samples (160 to 519 ft). Figure 21 shows a systematic increase of swell pressure with depth for all the materials.

The swell pressure measured normal to bedding is compared in Table 11 to the existing effective overburden pressure. A range in overburden pressure is given assuming the groundwater table to be coincident with the ground surface for the lower value and fifty ft below the ground surface for the upper value. The difference between the effective overburden pressure and the swell pressure (i.e., excess swell pressure) is significantly greater for deep samples than for shallow samples except for the Bearpaw and one specimen from the Claggett. Whether this difference is related to bonding as described by Bjerrum (1967) is not clear. A large decrease in excess swell pressure toward the ground surface could indicate the destruction of diagenetic bonds on unloading, as discussed in Section 2. Such bonds would probably be in the "weak bonds" category. However, the largest decrease in excess swell pressure was determined for the Colorado group and perhaps the Claggett formation; (as indicated by comparison of sample Nos. DH-1 U-40 and DH-2 U-3, Table 11); both

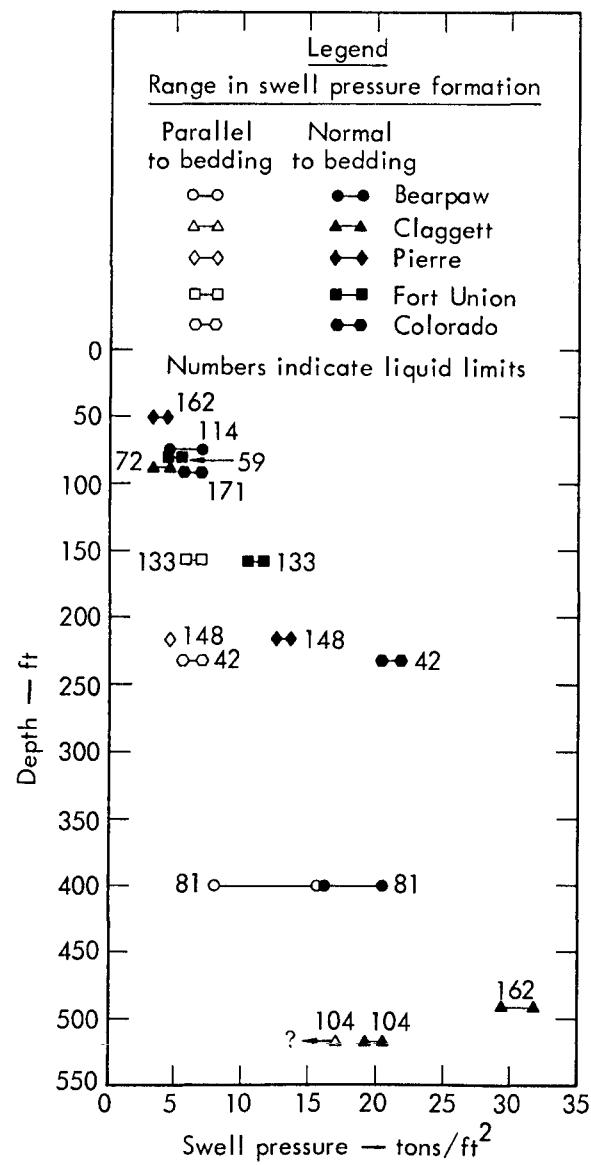


Fig. 21. Swell pressure vs depth, all formations.

materials are thought to contain strong bonds.

In addition to depth dependence, swell pressure may be influenced by plasticity characteristics and the availability of water. Swell pressure was measured on two specimens of the Claggett formation from roughly the same depth (491 and 519 ft) but with significantly different liquid limits (see Table 10). The swell pressure was over 30% greater for the

Table 11. Comparison of overburden pressure and swell pressure normal to bedding

Material & sample no.	Depth (ft)	Effective overburden pressure (tons/ft ²)	Vertical swell pressure (tons/ft ²)	Average excess swell pressure (tons/ft ²)
Colorado				
DH-1 U-7	100	2.5 - 4.0	5.7 - 6.8	3.0
DH-1 U-24	234	9.0 - 10.5	20.4 - 21.5	11.2
Claggett				
DH-1 U-40	491	19 - 20.5	29.5 - 31.7	10.8
DH-1 U-50	519	20 - 21.5	19.3 - 20.4	-0.8
DH-2 U-3	88	1.5 - 3.2	3.3 - 4.5	1.6
Bearpaw				
DH-1 U-46	401	16.0 - 18.0	15.9 - 20.4	1.4
DH-2 U-2	76	3.0 - 4.5	4.5 - 6.8	1.9
Fort Union				
DH-1 U-10	82	2.7 - 4.2	4.4 - 5.4	1.5
DH-1 U-22	157	5.0 - 6.5	10.2 - 11.2	5.0
Pierre				
DH-1 U-2	52	0.8 - 2.3	3.4 - 4.5	2.4
DH-1 U-20	216	7.0 - 8.5	12.5 - 13.6	5.3

specimen with the higher liquid limit. The reduction in excess swell pressure at shallow depths might also reflect the greater availability of water near the surface, so that the materials tended toward equilibrium under a different moisture environment. Such behavior could be independent of bonding in any of the materials. The water content profiles (Volume 2, Appendix B) do show a systematic decrease in water content with depth. Several more data points are needed before the causes of these interesting phenomena can be adequately explained.

Consolidation Tests

The consolidation test results of undisturbed oriented specimens for all five materials studied were plotted as void ratio vs log effective consolidation pressure (Fig. 22). Specimens were loaded to a maximum of 350 tons/ft².

These plots present two interesting features. One is that the straight line or virgin portion of the consolidation curves is nearly identical for specimens oriented from a position normal or parallel to bedding. An exception is the Colorado group, where a specimen loaded parallel to bedding did not consolidate as much as a specimen loaded normal to bedding. In that case, it is believed that the thin cemented layers of siltstone resisted consolidation. The data suggest that the specimens behave isotropically at loads that exceed their maximum past consolidation load.

A second interesting aspect is the relative amount of energy absorbed by the specimens during the consolidation testing. The area under the consolidation curve beyond the swell pressure is related to the work performed on the sample, while the area under the rebound portion of the curve is related in the same way to

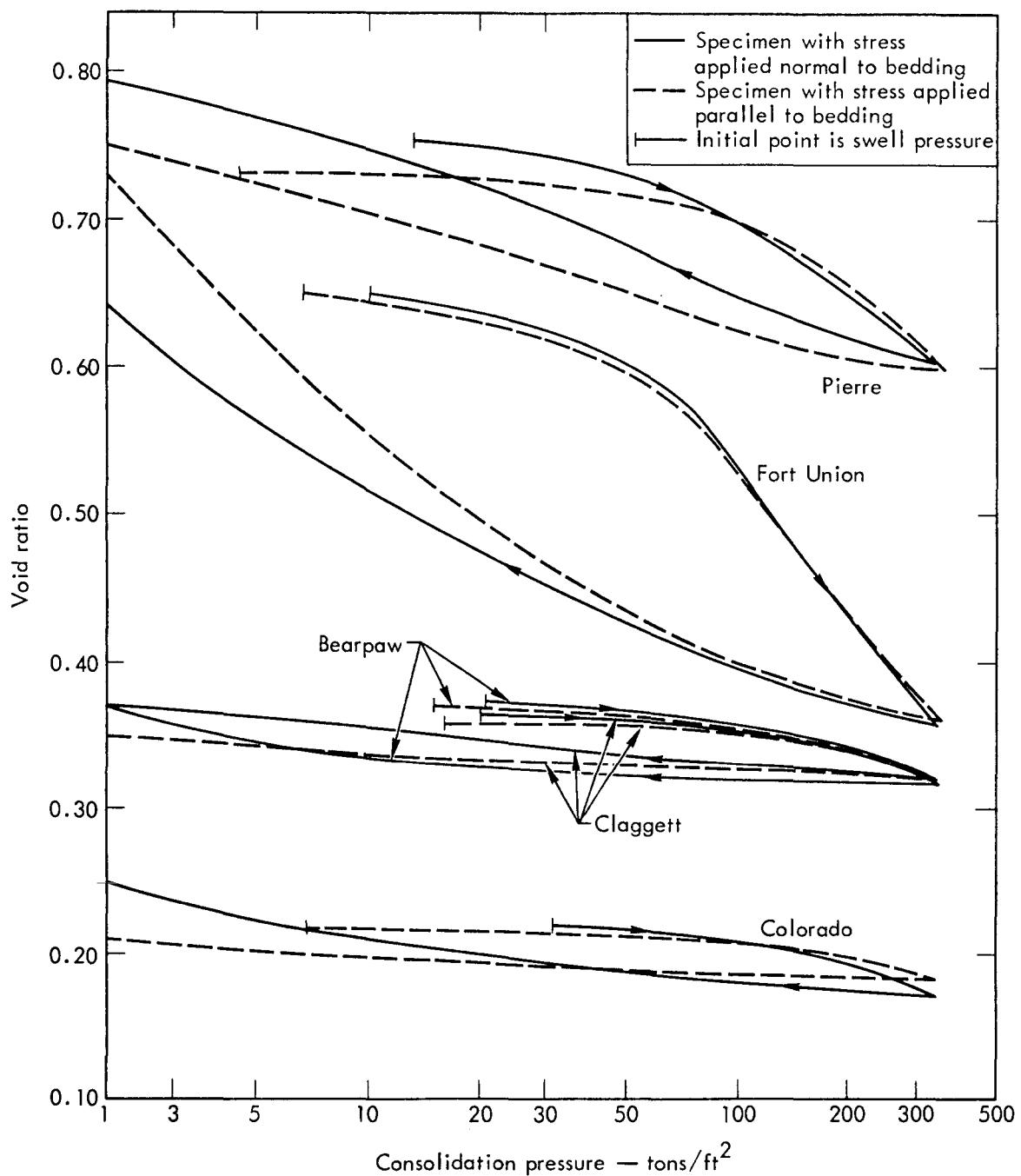


Fig. 22. Comparative consolidation test results for oriented specimens.

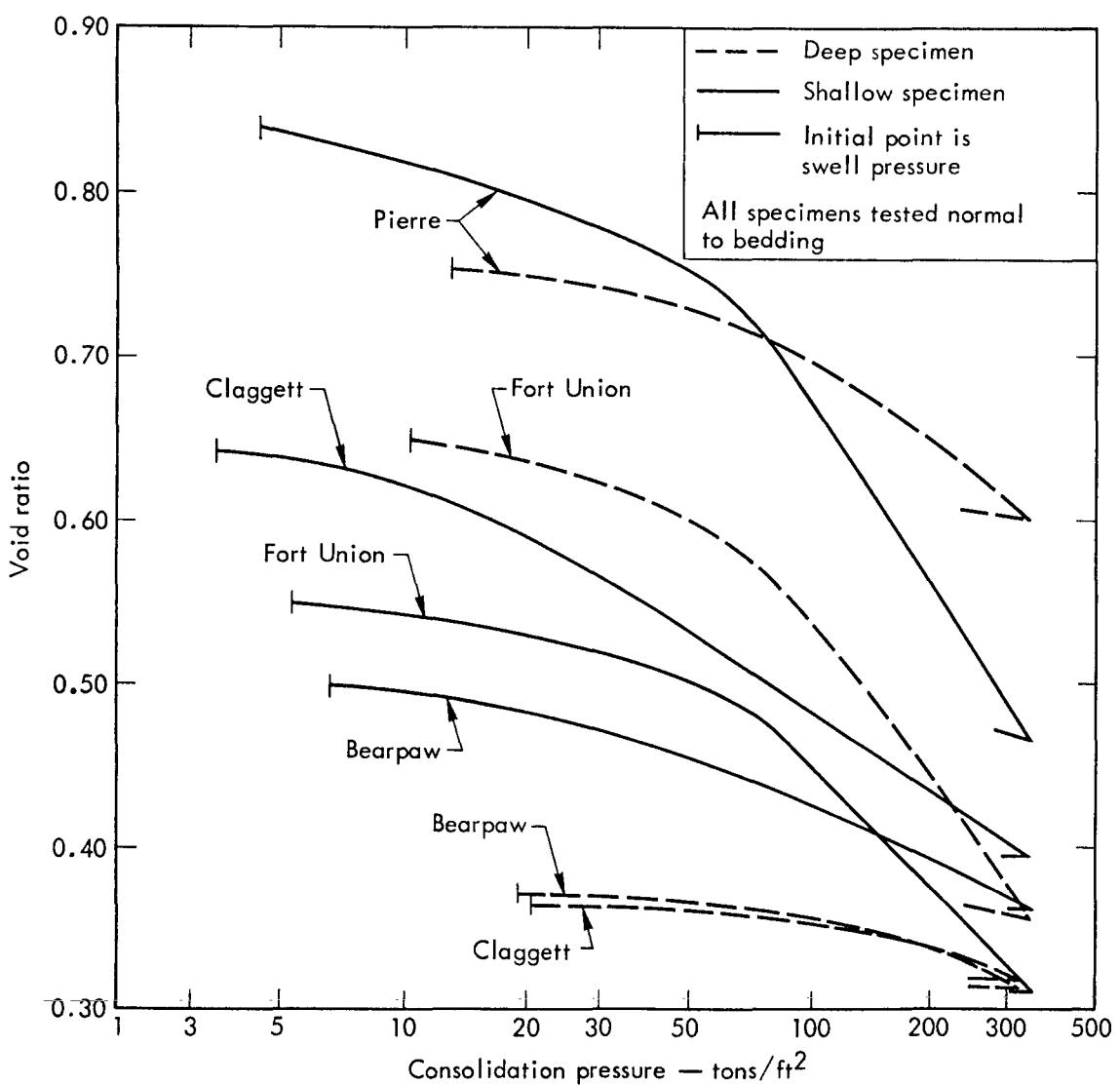
the work done by the specimen in rebounding in the consolidometer. If similar materials are compared it seems reasonable to expect similar characteristics. The Pierre formation specimen had a liquid limit of 148, an initial water content of 26%, and a swell pressure, measured normal to bedding, of between 12.5 and 13.6 tons/ft². The Fort Union specimen had a liquid limit of 133, an initial water content of 24%, and a swell pressure measured normal to bedding of between 10.2 and 11.2 tons/ft². Although the initial void ratio of the Pierre specimen was larger than that of the Fort Union, the former absorbed less energy during consolidation. It appears that there is some property present in the Pierre which resisted consolidation that is not present or operated to a lesser degree in the Fort Union. This property may be termed bonding, but it is not clear whether this is the same type of bonding discussed by Bjerrum (1967) and summarized in Section 2. If the data represent bonding, then the relative strength of the bonds in the five materials can be compared. The Colorado group apparently had the strongest bonds, and considering that this group contains cemented silt layers, the bonds might be classified as strong-to-permanent. The Claggett and Bearpaw curves are very similar and both materials appear strongly bonded. Bonds in the Pierre were weaker than those in the Colorado, Claggett, or Bearpaw, but stronger than those in the Fort Union.

Data from shallower samples in the same materials suggest that in some cases bonding was reduced or destroyed near the ground surface. Figure 23 compares the consolidation curves for deep and

shallow specimens, both loaded normal to bedding. The results from the shallow Pierre, Bearpaw, and Claggett specimens show the effects of rebound and perhaps of the destruction of bonding. The fact that the Fort Union shallow specimen absorbed less energy than the deeper specimen is due in part to the different plasticity characteristics for the two specimens (liquid limit equal, 133 for the deeper vs 59 for the shallower specimen), but it may also be due in part to the absence of bonding in the Fort Union. If there were no bonds (or very weak bonds) formed during natural consolidation, it would be anticipated that the curves would be the same for similar plasticity materials. The shallow specimen of Colorado group materials (not included in Fig. 23, but shown in Fig. C13, Volume 2), consolidated significantly more than the deeper specimen. The minimum void ratio for the shallow specimen was 0.394 compared to 0.166 for the deeper specimen. However, the shallow specimen was a bentonite (liquid limit 171) which did not contain any of the highly bonded layers, and the test results were very poor.

Shrinkage Tests

Shrinkage strains resulting from air-drying from natural water content were measured on specimens cut normal and parallel to bedding. Shrinkage strain test results on oriented specimens from deep borings in the Claggett and Bearpaw formations are shown in Fig. 24. Evidence of increased bonding with depth is suggested by the small difference in strain measured in deep samples compared to large differences near the surface. All the materials (see Figs. C23 through C25, Volume 2) had shrinkage strains parallel to



		Claggett	Bearpaw	Fort Union	Pierre
Shallow	Depth (ft)	88	76	82	51
	Initial water content (%)	21	18	19	29
	Liquid limit	72	114	59	162
	Plastic limit	22	24	20	28
Deep	Depth (ft)	519	401	157	216
	Initial water content (%)	11	12	24	26
	Liquid limit	104	81	133	148
	Plastic limit	22	21	22	35

Fig. 23. Comparative consolidation test results for deep and shallow specimens.

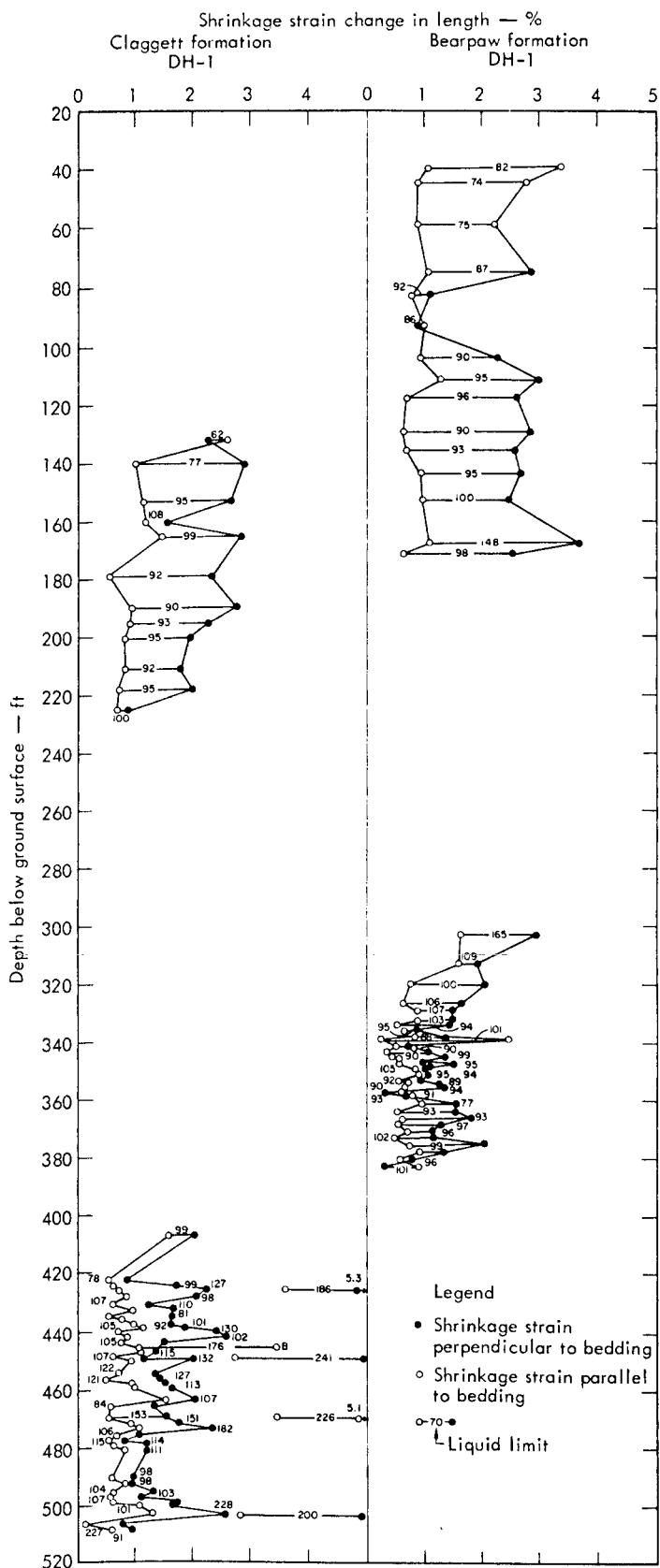


Fig. 24. Shrinkage strain for Claggett and Bearpaw formations.

bedding that were significantly less than those normal to bedding. Also, all except the Fort Union and specimens from a shallow boring in failed Pierre materials showed a perceptible decrease in shrinkage with depth. Again, this appears to be evidence of a lack of strong bonding in the Fort Union group. The materials encountered in boring DH-2 in the Pierre were failed, and bonding formed during natural consolidation might have been destroyed. In general, it appears that large shrinkage strains reflect rebound that occurred in the slopes, and these data are consistent with the consolidation tests performed on deep and shallow specimens.

Residual Strength

Residual strength testing of the five materials was performed on precut surfaces by repeated direct shear. Details of procedure, problems, and data are given in Volume 2, Appendix C. In general, it was found that all the materials could be measured for low residual strengths. In fact, specimens of the Fort Union group recorded both the lowest and the highest strengths measured.

A relationship between the residual shear strength and the liquid limit for the five clay shales studied is shown in Fig. 25. Data from a sixth material, the Dawson formation, have been included because of extensive residual testing performed recently in connection with the design of Chatfield Dam (Volume 2, Appendix D and Fig. 9). The shear strength was reported in terms of the effective friction angle assuming zero cohesion. The well-defined relationship between the liquid limit and the residual

shear strength for the Dawson clay shale probably resulted from the large number of tests conducted and the uniformity in mineralogy of the clay fraction.

The relationship of the liquid limit and the residual shear strength for the other five formations is not as well defined, due to the limited number of tests conducted, but the trend is clearly evident. The Fort Union group results plot parallel to those of the Dawson formation, but the data indicate that the Fort Union is the stronger material for a given liquid limit. These materials are similar, both having been deposited in a nonmarine environment. The Bearpaw, Colorado, and Pierre, all marine deposits, form curves less steep than the Dawson and Fort Union curves and reflect large ranges in liquid limit. Data from the Claggett, also a marine formation, may be an exception. The one sample of Claggett tested with a high liquid limit (172) was cemented with 22% calcite and was not included in the interpretation, leaving a total range of liquid limit available for interpretation of only about 30%. Although a line through the four data points for Claggett may not be representative of the overall formation, the results are more like those from the other marine formations than those from the Dawson and the Fort Union.

It is puzzling that known weak clay shales such as the Pierre, Bearpaw, and Claggett indicate higher strengths than the Dawson and the Fort Union clay shales, which generally do not present slope problems. It appears that low residual shear strengths can be measured in the laboratory for all the clay shales tested; however, the field slope behavior is related to the particular materials in

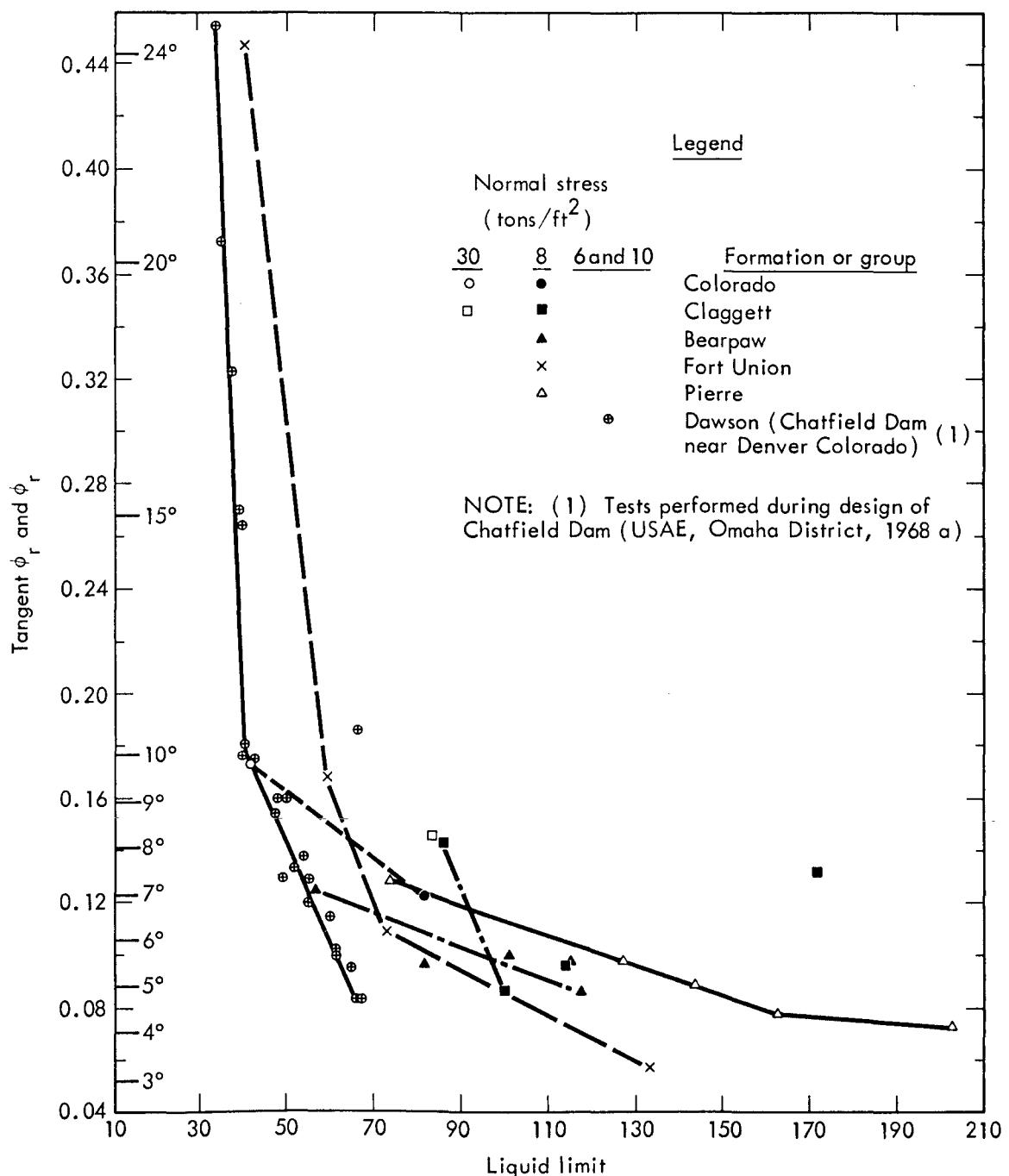


Fig. 25. Summary of residual direct shear test data.

which a potential slide zone is located and to associated seepage conditions. The continental deposits contain numerous layers of stronger and more permeable materials, such as sandstone, siltstone, and lignite, which add materially to overall slope stability. The marine deposits

tend to be uniformly composed of lower-strength materials. The Colorado shale is a generally stable marine deposit with a laboratory-measured residual friction angle of about 7 deg in the higher-plasticity material. However, the Colorado appears strengthened by its numerous silt layers.

Cementation and Slickensides

Cementation was studied by (1) visually noting the reaction of air-dry material to dilute hydrochloric acid, (2) insoluble residue tests, and (3) slaking tests in ethylene glycol and water. The slaking tests were carefully monitored to note rate and degree of specimen breakdown. Detailed results are given in Fig. C3 of Volume 2. Zones of cemented material were identified in all five formations. The zones of cementation in the Bearpaw and Claggett were so localized that they are hardly noteworthy and probably have little influence on overall formation stability. Slight cementation, based on acid reaction, was found in the upper 111 ft of the Fort Union group and in an 80-ft zone of the Pierre formation. Except for parts of the Colorado group, all the materials broke down completely in water when slaked from the air-dry state. In the only exception, the thin silty layers in the Colorado group did not slake even after six cycles of wetting and drying. The clay shale portions of the Colorado group slaked completely in one cycle. Obviously, if the cementing agents bond the particles strongly, the potential mass stability is enhanced. However, tests to investigate the effects of very slight cementation by minerals other than calcium carbonate are lacking, and such tests need to be developed.

Slickensides and gouge zones were noted by the geologist in the field; similar occurrences were noted in the laboratory examination. A slickenside classification and description of slickensided zones are given in Volume 2, Appendix C. Very small shiny surfaces could be detected only by carefully dissecting the core and studying each tiny fragment. In the laboratory,

only a small section of core could be examined in this way because of the requirement to preserve specimens for other tests. The tiny slickensides were generally not detected in the field, suggesting either that they were too small to be observed or that slickensides developed during storage through stress relief. A summary of slickensides and gouge zones is presented below.

Colorado Group. No slickensides were found in the Colorado group laboratory specimens. Throughgoing slickensides were found in the field boring samples at depths of 76 and 147 to 149 ft. In the latter zone, the moisture content increased from 6 to 17%. The bedding below the lower zone was tilted at 3 to 7 deg, while the bedding above was nearly horizontal. Bedding exposed in the slope was also horizontal.

Claggett Formation. Two borings were drilled in the Claggett formation. The deeper boring at the top of the slope encountered 148 ft of Judith River sandstone and then Claggett shale to a total depth of 532 ft. The lower part of the Judith River and the upper 50 ft of Claggett (i.e., a total depth of 200 ft) contained several zones of discontinuous-to-continuous slickensides. Throughgoing slickensides were also found at depths of 444, 489 to 494, and 524 to 527 ft in association with bentonite layers. A shallower boring drilled to a depth of 204 ft at approximately mid-slope contained numerous slickenside zones. Slickensides were developed to some degree in every sample examined in the laboratory. Numerous zones of gouge and tilted bedding up to near vertical were found throughout the boring.

Bearpaw Formation. Two borings also were drilled in the Bearpaw formation. The deeper boring, located at the top of the slope, contained scattered slickensides throughout the 405 ft sampled, with a significant gouge zone at a depth of 264 ft. The mid-slope boring (to 195 ft) contained zones of slickensides somewhat more intense than those in the upper boring. Numerous slickensides and broken materials were identified in the Bearpaw just above the contact with the underlying Judith River sandstone.

Fort Union Group. A few slickensides were found in the field associated with lignite beds. No slickensides were found in the laboratory specimens.

Pierre Formation. Two borings were drilled in the Pierre. Scattered slickensides were noted throughout both borings. There was a marked increase in moisture content, with some slickensides, just above the contact with the underlying Niobrara chalk.

ANALYSIS OF SLOPES

Slope Charts

The data collected from the present study can be presented in a form similar to slope charts developed at selected engineering projects (Appendix D), and as clay shale slopes in other geographical areas (Appendix E). The technique involves the plotting of observed slope height vs slope inclination (expressed in degrees or as slope cotangent). Slope charts have proved to be valuable tools for the assessment and the design of many project slopes (see, for example, Lane,

1961, and MacDonald, 1942); however, in such cases the data collected for a specific application are from a local area and constitute a consistent set of information applicable to other slopes in the immediate area. When the data are used to compare field observations from different formations, or from within a formation but in cases in which the slopes are influenced by different factors, some caution is necessary in drawing conclusions concerning the degree of similarity (or dissimilarity).

Points on a slope chart are identified as representing either failed or unfailed slopes. If a series of slopes are made with varying heights and inclinations, those that exceed a critical height will fail and reduce the inclination; such slope adjustment will in most cases occur without a substantial reduction in height. Thus a line may be constructed on a slope chart to indicate an upper limit for the height of stable observed slopes. Points representing failed slopes will move horizontally from their unfailed position on the chart toward the limiting line. Thus for slopes created by the down-cutting of a river, such as those in the formations under present study, failure occurs at a time when the slope height reaches a value inconsistent with the shear strength of the material, point (a) Fig. 26. After failure the inclination reduces to point (b) if the strength is composed of frictional and cohesive components, and to point (c) if the strength is represented by friction alone. From point (b) or (c) the relation between slope height and inclination moves respectively to point (d) or (e) as the height is increased. The actual path taken as slope height increases through the

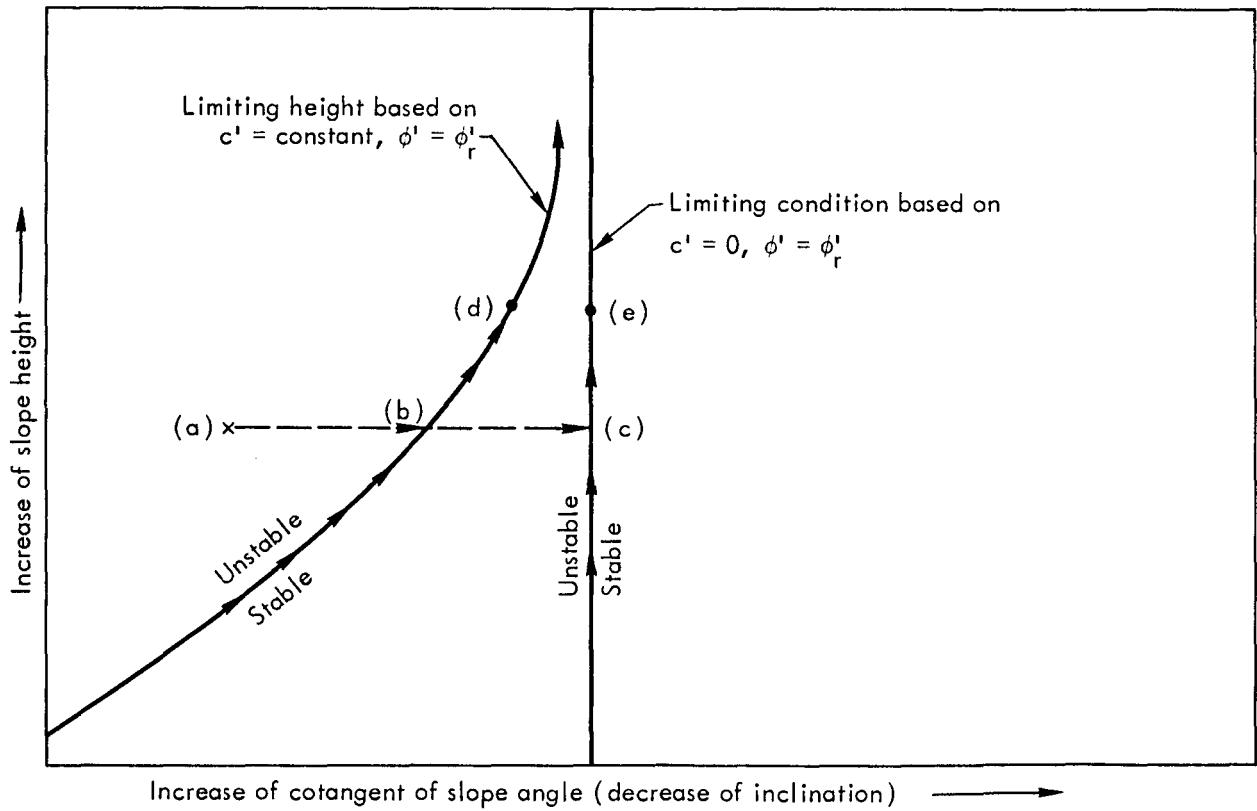


Fig. 26. Theoretical slope chart for development of a clay shale slope.

downcutting of a river is complex, reflecting variations in strength that may occur with time and space, reductions in strength from peak to residual, groundwater variations, and stratigraphy.

In using data collected on slope charts, a careful distinction should be made of the time scale represented by the data. Skempton and Hutchinson (1969) showed that observed slopes in London clay demonstrated a general tendency for the inclination to decrease with increasing orders of magnitude of time. Thus what may be considered an acceptable slope inclination for immediate excavation can be somewhat steeper than that acceptable for long-term excavation from an engineering standpoint. In turn, the long-term excavation from an engineering standpoint can be somewhat steeper in contrast to

long-term in the geologic sense. Data collected as part of this study (Volume 2, Appendix A), and as discussed subsequently represent slope heights and inclinations resulting from geologic time; the results indicate that the slope inclinations are comparable to the laboratory residual angles of friction. Excavated slopes at engineering projects (Volume 2, Appendix D), have been designed on the basis of strength parameters which yield steeper inclinations for a given slope height than would be indicated by the inclination of geologically older slopes. Peak strength parameters were not determined as part of this study. However, in the case histories discussed in Appendix D, peak shear strengths from laboratory tests were not used for slope design; instead, design strength parameters were determined

from observed slope behavior in the immediate project areas.

For reasons explained in Section 2, a slope in clay shale does not behave in a manner similar to one in a normally consolidated clay. However, empirically derived slope charts in both types of material have the same general shape (i.e., a concave-upward curve), indicating increasingly higher slopes for decreasing slope inclinations. Curves constructed from several previous studies are shown in Fig. 27.

In normally consolidated clays, a slope chart prepared from given strength parameters c' and ϕ' indicates a general concave-upward shape as shown in Fig. 28 (Taylor, 1948). Such data indicate that as the inclination of the slope increases (decreasing cotangent) the critical slope height (factor of safety equal to unity) decreases. For reference on Fig. 28, the critical height for a purely cohesive material (ϕ' equals zero) depends entirely upon the value of the cohesion and is independent

of the slope angle, as indicated by the horizontal line. Similarly, for a purely frictional material (c' equals zero) there exists no unique height-slope angle relationship, as indicated by the vertical line. Steady seepage lowers the critical height for a given slope angle. A rigid base raises the critical height for a given slope angle.

As noted previously, the slopes in the Claggett, the Bearpaw, and the Pierre formations are similar. Residual friction angles for the three formations were found to vary from 4.1 to 8.3 deg (cotangents, 14 to 7). Slope charts indicating the heights and inclinations of slopes in the five study formations are shown in Figs. 29 through 33 (see also Tables A1 through A5, Volume 2). The thickness of the shale varied from 430 to 500 ft for the Claggett formation, 400 to 600 ft for the Bearpaw formation (two slopes 140 ft), and 200 to 490 ft for the Pierre formation. For slopes composed entirely of shale the range in slope angles compares to the

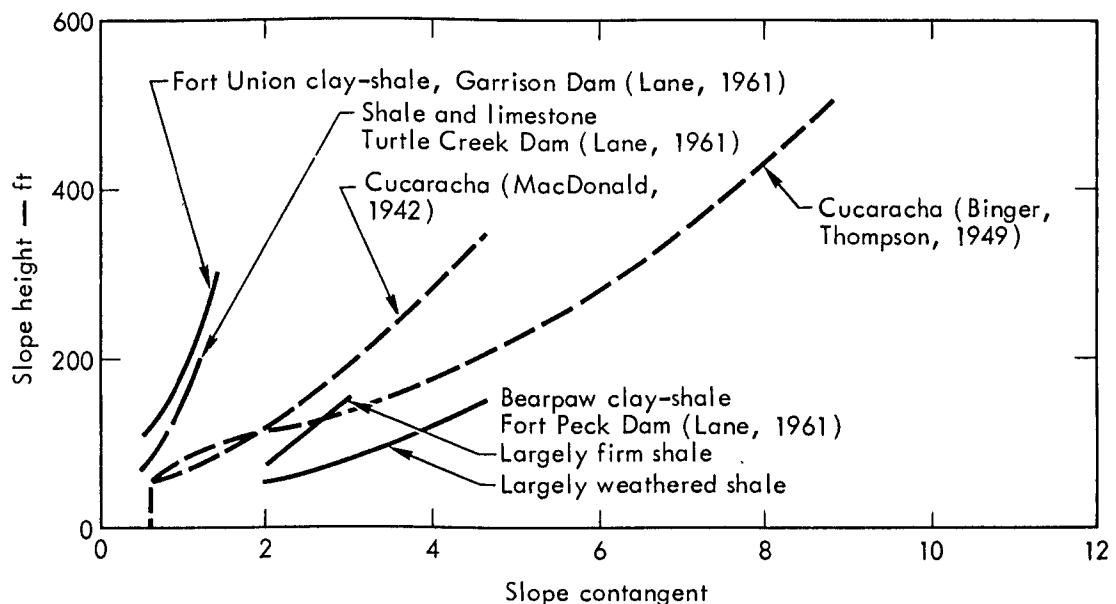


Fig. 27. Slope design curves for several projects constructed in clay shale.

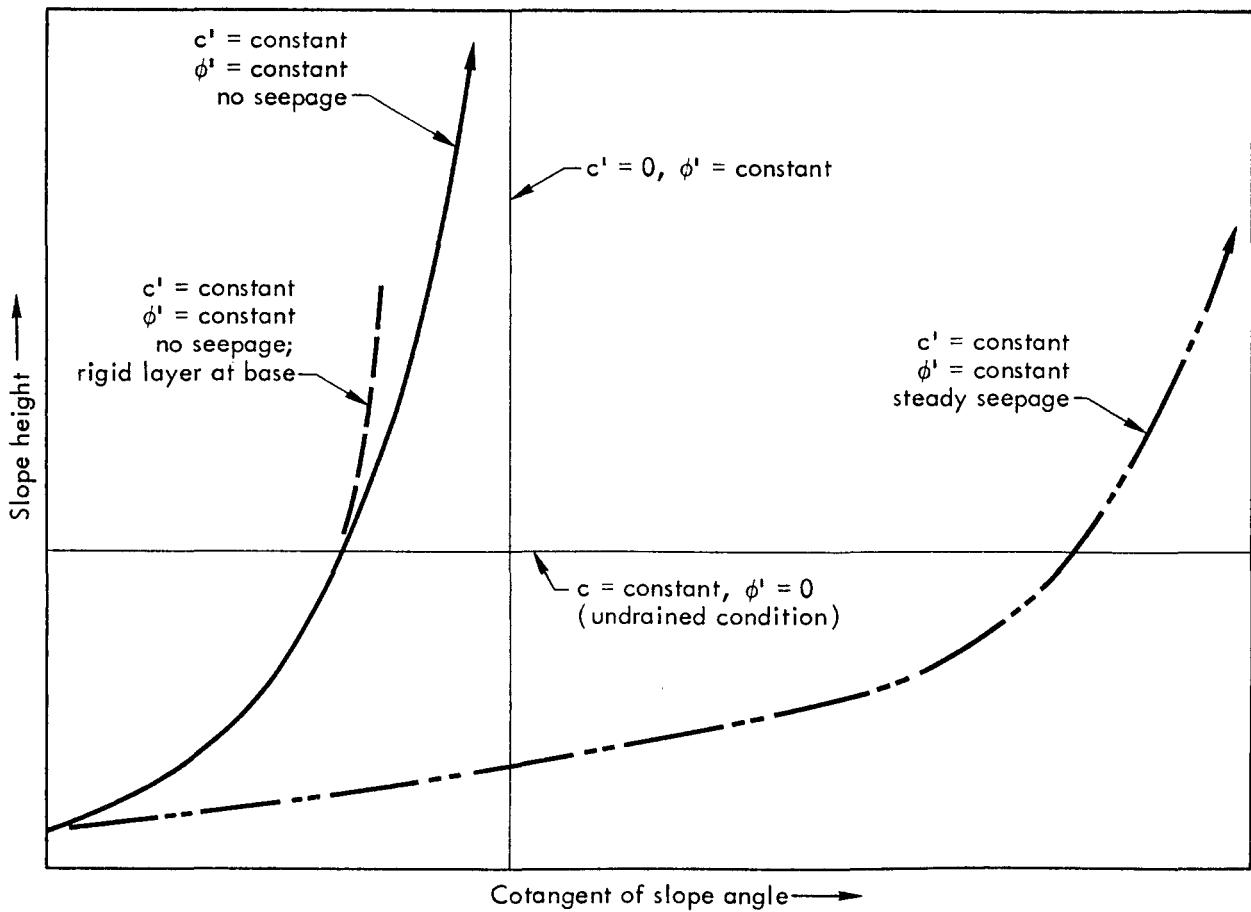


Fig. 28. Theoretical slope chart showing effects of seepage.

range in residual angle of internal friction. A rigid underlying base, such as the Judith River sandstone at some of the Bearpaw sites, appears to have a stabilizing effect and tends to create slightly steeper slopes than those without such a base. The rigid base was shown previously to increase the slope inclination for a given height in a normally consolidated clay. Additionally, for the Bearpaw slopes the Judith River may reflect a more favorable drainage condition. Although the Pierre formation was underlain by Niobrara chalk at some sites, the range in slope inclinations was comparable to the ranges for slope inclinations in shale only. The Niobrara chalk possibly did

not offer the same drainage potential as the Judith River sandstone. Slopes in the Bearpaw which were capped with the Fox Hills sandstone were also found to have steeper inclinations than those in shale only.

Some slopes in the Pierre were capped with Pleistocene deposits but did not indicate any appreciable increase in inclination. The difference may possibly lie in the relatively higher strengths of the materials above and below the Bearpaw and the smaller amount of strength contributed by the Pleistocene deposits above the Pierre slopes. Also, the slopes in the Pierre are older than those in the other materials by perhaps up to 20,000 yr.

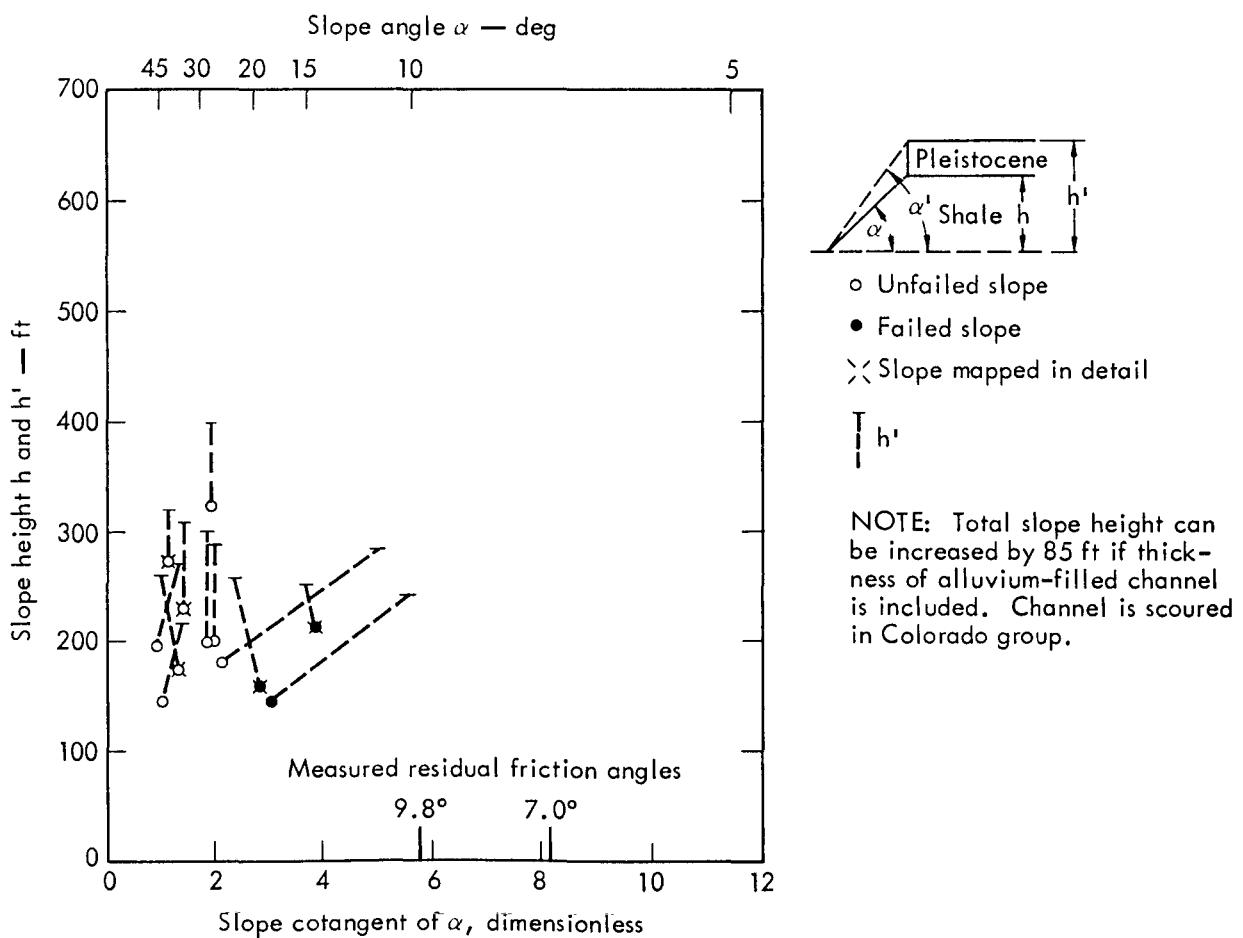


Fig. 29. Height and inclination of slopes in Colorado group.

Erosion may have reduced the inclination of those slopes to the extent that the effect of reinforcing members is not obvious. Although the Claggett formation is overlain by the Judith River sandstone and underlain by the Eagle sandstone, a situation similar to that which gave steeper slopes in the Bearpaw, the slopes in the Claggett formation are at inclinations comparable to the residual angle of internal friction. This observation is very curious in the light of the striking similarities between the Bearpaw and Claggett formations as determined from the boring information and laboratory tests.

The design slopes for two major projects in the Bearpaw and Pierre forma-

tions were based in part on slope charts constructed from slopes in the respective project areas. These curves are shown on the appropriate slope height and inclination plots, Figs. 31 and 33, for comparison to the data collected in the present study. For the Bearpaw formation two curves of limited height were constructed for the Fort Peck Dam design, one for firm material and one for weathered. The curves were comparable to data collected in the present study for shorter slope segments which were in a failed condition. The construction of a cut slope in a mass which has already experienced failure would be expected to create unbalanced shear stresses which

would cause the slope to adjust itself by flattening. The observation of slopes (Appendix D) did indicate that slope adjustments have occurred at various times. The same general observations are applicable to the Oahe Dam in the Pierre formation where slope failures occurred on both abutments during construction. Movement was mainly along the old failure surfaces which had developed during down-cutting of the river.

Slopes in the Colorado group stand at relatively steep angles when compared to the residual angle of internal friction, Fig. 29. Residual friction angles ranged from 9.8 to 7.0 deg (cotangents 5.8 to 8.1);

stable slopes were found ranging in height from 140 to 320 ft with slope cotangents ranging from 1 to 2. Failed slopes were reported standing at slope cotangents of 3 to 4. The average strength along failure planes must have been increased from that obtained from the residual friction angle by the influence of the numerous interbeds of cemented silt.

Slopes in three formations of the Fort Union group stand at heights varying from 200 to 600 ft at angles comparable to the residual angle of internal friction. However, the residual angle of internal friction has such a wide variation (24.1 to 3.3 deg) that the comparison to slope

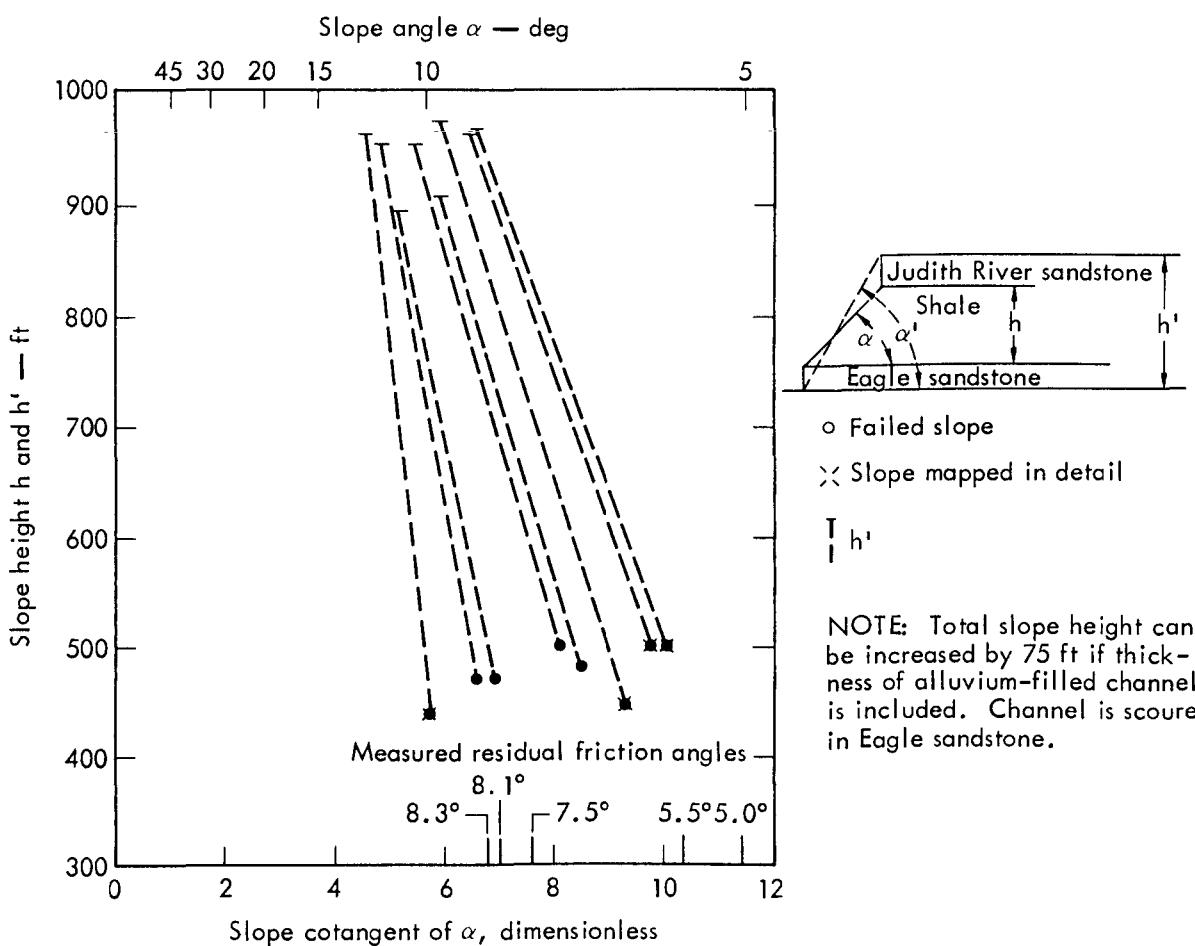


Fig. 30. Height and inclination of slopes in Claggett formation.

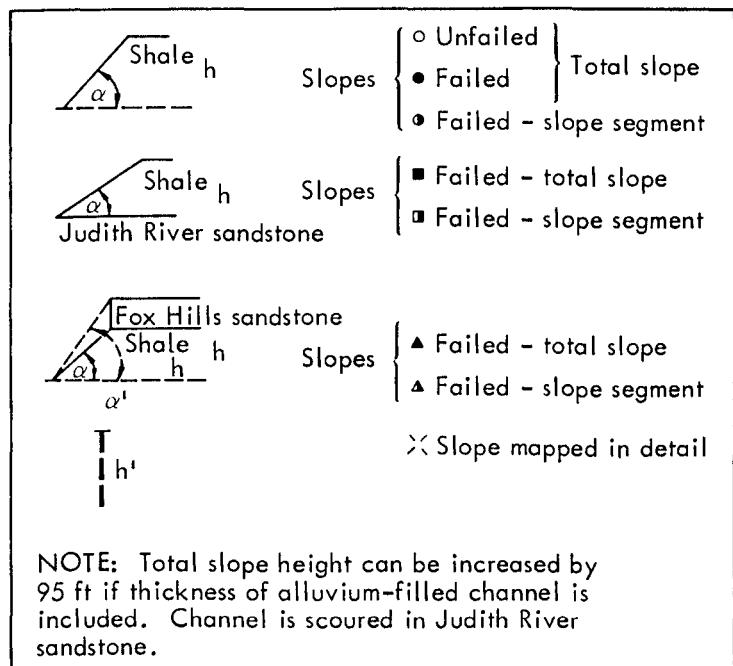
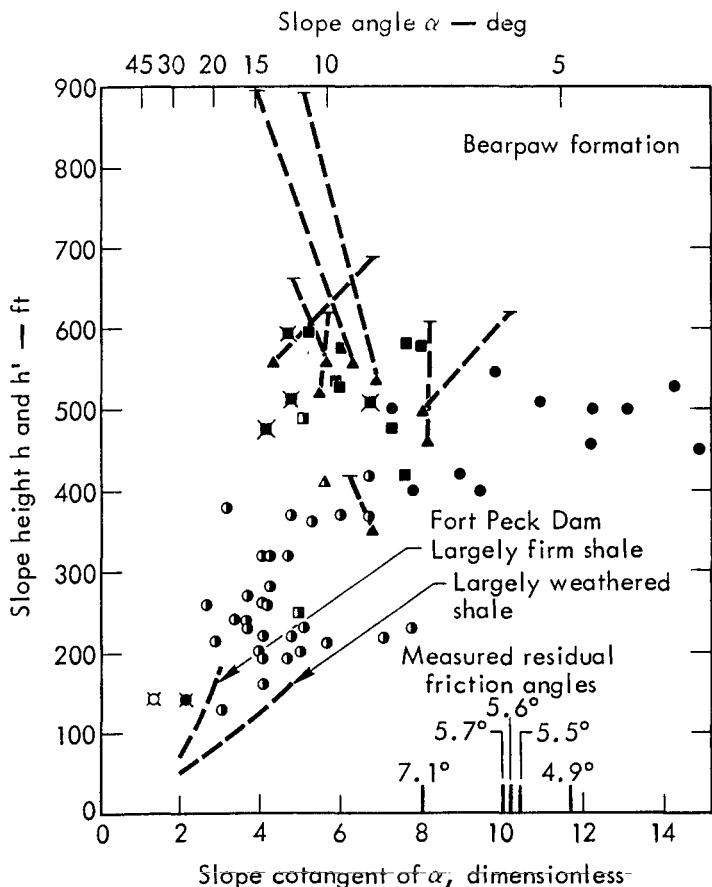


Fig. 31. Height and inclination of slopes in Bearpaw formation.

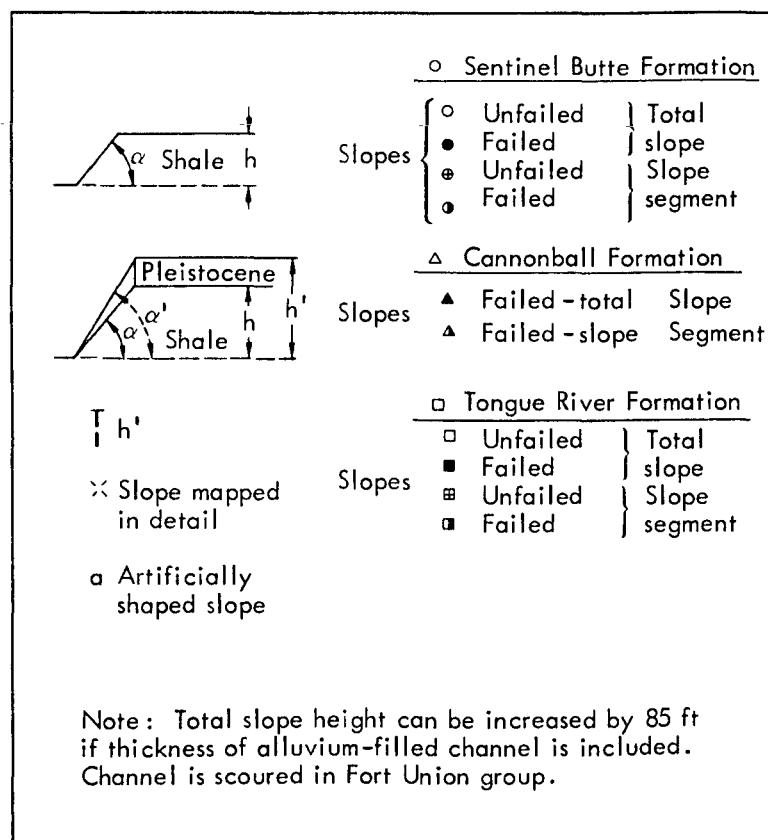
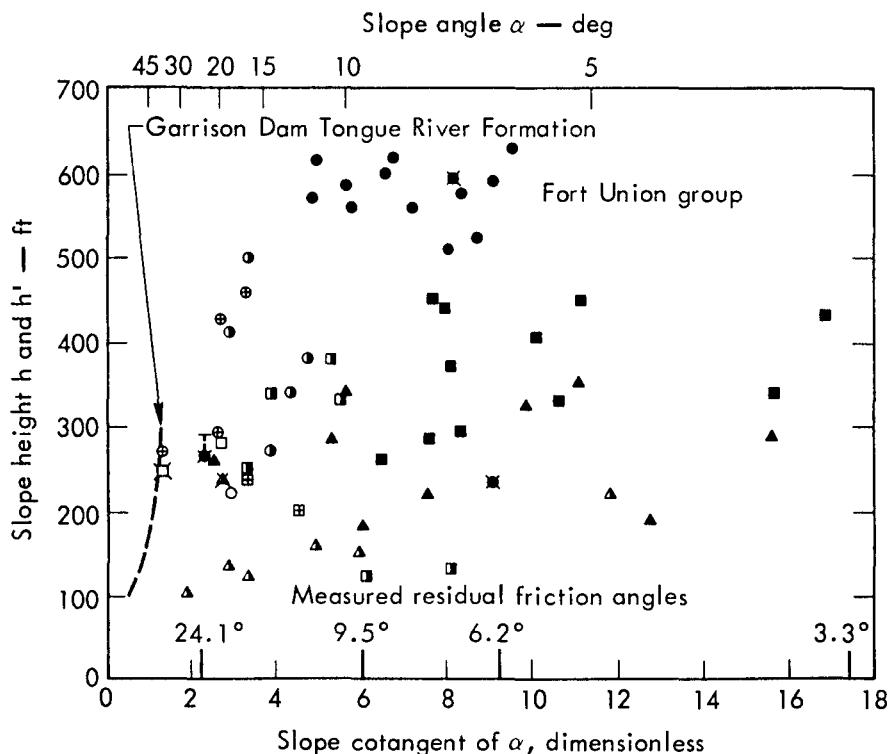


Fig. 32. Height and inclination of slopes in Fort Union group.

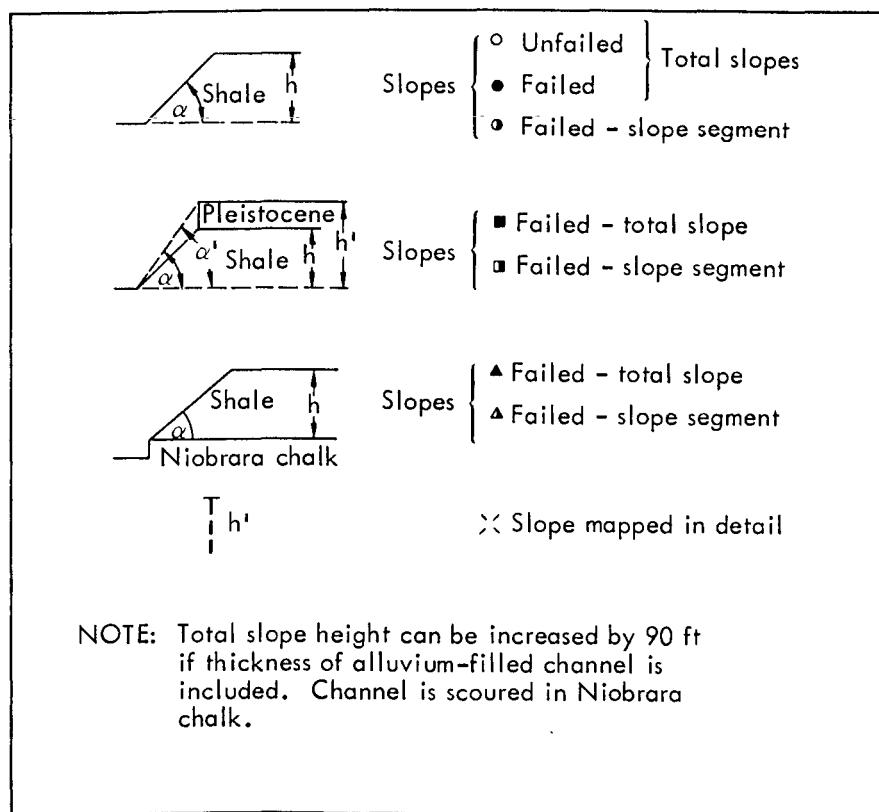
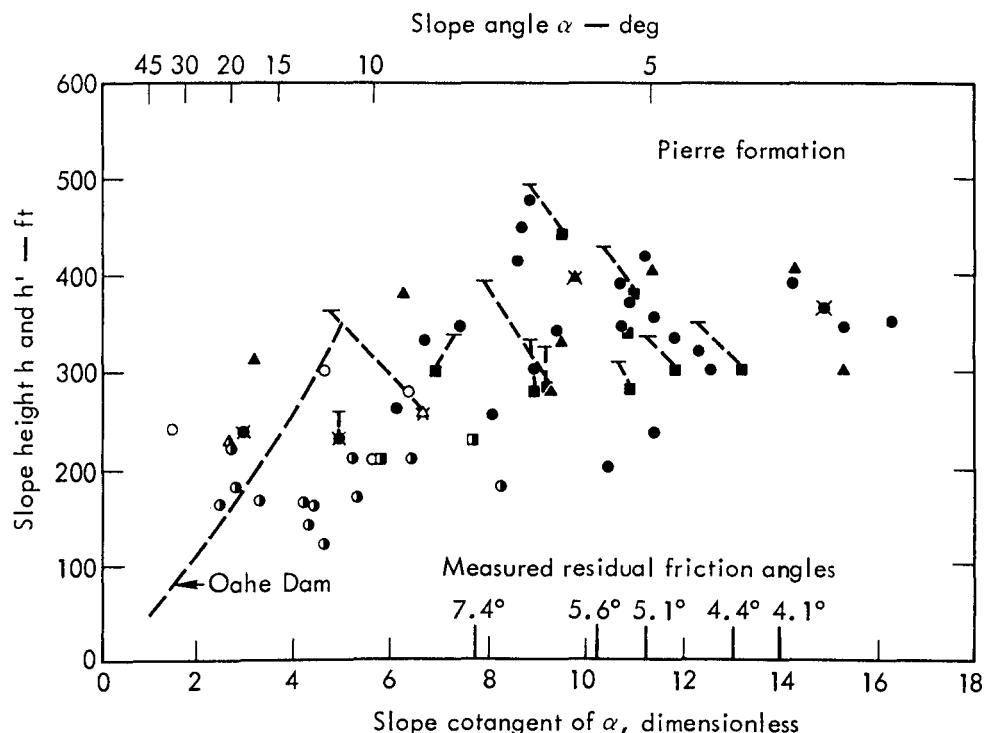


Fig. 33. Height and inclination of slopes in Pierre formation.

inclination has little significance. When the data were compared to each other, it was found that slopes lying mostly in the Sentinel Butte formation are steeper as well as higher, as a group, than slopes in the other formations. This behavior is thought to be a reflection of overall more coarse-grained lithology in the Sentinel Butte.

The selection of design slopes for Garrison Dam in the Tongue River formation of the Fort Union group was based in part on slope charts constructed from slopes in the project area. The curve was plotted on the slope chart, Fig. 32, and compares favorably with data collected in the present study for slopes and slope segments located within the Tongue River formation. Project slopes have been observed to be stable, with no failure over a 22-yr performance history. Slopes in this material are not failed throughout, as are slopes in the Bearpaw, Claggett, and Pierre formations. The slope design curve at Garrison Dam was constructed on the basis of the strength of the material and the seepage conditions at the site. For cases such as this, the slope chart has proven to be a useful and reliable tool.

Stability Analyses of Slopes

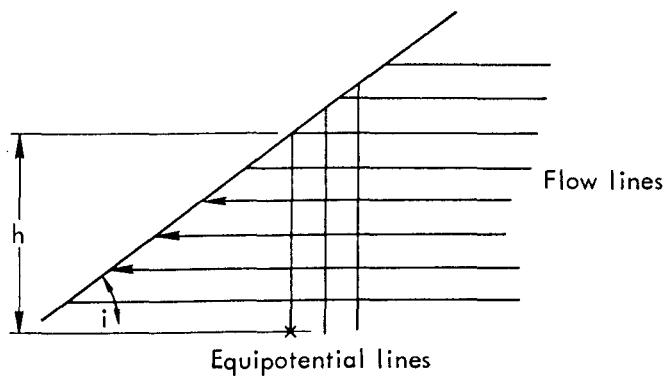
Analyses of failed slopes in overconsolidated clays and clay shales have been performed many times; for example, Skempton (1964), Sinclair et al., (1966), Hutchinson (1969), and Volume 2, Appendix D, of this report. In general, these analyses are possible after detailed studies of sites identifying failure surfaces and groundwater conditions. When factors of safety have been computed with a residual

friction angle, the factors of safety have been shown to be near unity; similarly computed average strengths along a failure surface assuming a factor of safety of 1.0 have been shown to correspond closely to that factor of safety given by the residual strength.

In this study an attempt was made to perform stability analyses of all failed slopes. In general, the information obtained was of such a nature that analysis was possible only by making gross assumptions as to the position of the failure surface and groundwater conditions. The analyses considered an active wedge with a variable base angle driving a passive wedge with a horizontal base; the boundary between the active and passive wedges was assumed to be vertical. As limiting conditions, seepage was assumed to be parallel to the outer slope, horizontal, or vertical (Fig. 34). The vertical condition is identical to a condition of saturation with no seepage; the extreme flatness of the slopes makes any difference between horizontal and parallel seepage very small. A parameter study of the boundary locations of the wedges indicated that for all practical purposes the wedge analysis and the infinite slope solution for the flat slopes give the same required friction angle as shown in Fig. 35 for a 1 on 10 slope.

The method of analysis is illustrated for the Claggett formation, site 2, Section G-G' in Fig. 36. First, a true-scale section was prepared. From boring information, a 0.3-ft gouge zone was noted at the contact between the overlying Judith River sandstone and the Claggett shale; the fact that this contact was some 140 ft below the observed in-place contact

Case: Horizontal seepage



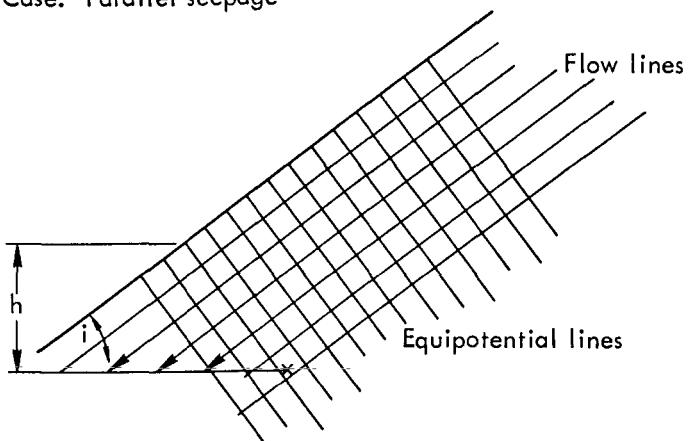
$$h @ x = h$$

$$u @ x = \gamma_w h$$

Infinite slope solution:

$$FS = \left[\frac{\gamma_t \cos^2 i - \gamma_w}{\gamma_t \sin i \cos i} \right] \tan \phi$$

Case: Parallel seepage



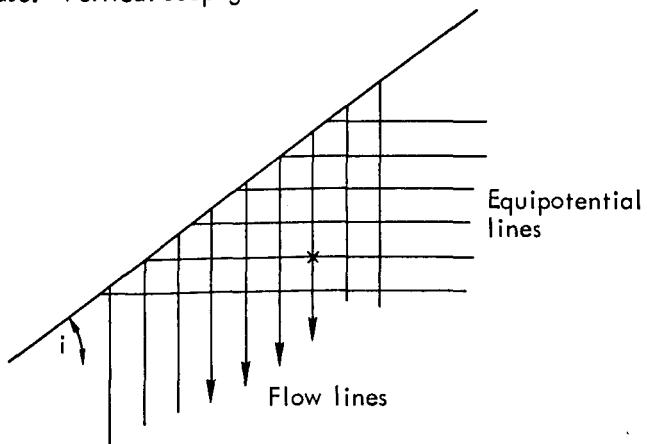
$$h @ x = h$$

$$u @ x = \gamma_w h$$

Infinite slope solution:

$$FS = \frac{\gamma_b}{\gamma_t} \frac{\tan \phi}{\tan i} = \frac{\gamma_t - \gamma_w}{\gamma_t} \frac{\tan \phi}{\tan i}$$

Case: Vertical seepage or no flow-saturated condition



$$h @ x = 0$$

$$u @ x = 0$$

Infinite slope solution:

$$FS = \frac{\tan \phi}{\tan i}$$

Fig. 34. Assumed seepage conditions in slope analyses.

indicated the possible base of an active sliding wedge (see Plate A-VII, Volume 2). The lower boring indicated fault zones at

several depths with the lowermost zone noted near the Claggett-Eagle sandstone contact; thus the base of the passive wedge

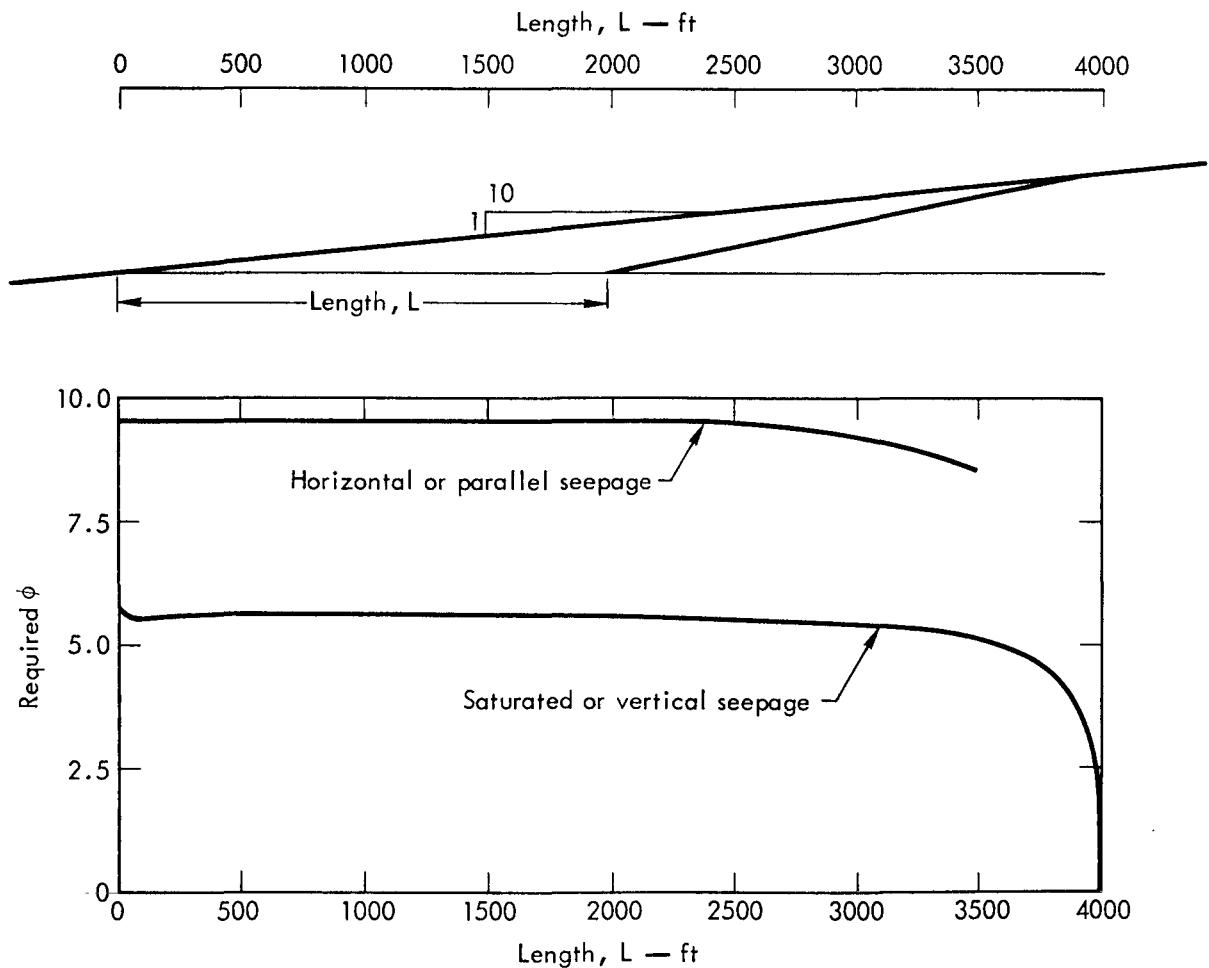


Fig. 35. Effect of boundary location on required angle of internal friction.

was assumed to coincide with that contact. The boundary between the active wedge and the passive wedge was assumed to be vertical. The required angle of internal friction was calculated using the assumed geometry for different assumed values of cohesion and a factor of safety of unity as shown in Fig. 36. The calculations were performed with the seepage conditions assumed parallel to the slope and vertical (or saturated—no flow conditions). For the relatively flat slopes, the differences in calculated strengths were negligible for assumed horizontal seepage and assumed parallel seepage. The difference between the required friction angle for cohesion

assumed equal to zero and the required friction angle computed from an infinite slope analysis for flat slopes was minimal for the flatter slopes. The actual difference was determined by the nonuniformity of the slope. This observation, as verified by different assumed active failure planes, Fig. 36, for site 1, Section F-F', indicates that the difference in backfigured strength parameters is not too dependent on the actual position of the failure surface.

By far the greater difference in backfigured strength parameters is generated by the assumed seepage conditions.

Section	Surface	Assumed seepage condition	Average required ϕ' for FS = 1 ($c' = \text{lb}/\text{ft}^2$)				Assumed * ϕ'_r	Average required ϕ' on inclined surfaces for FS = 1 ($c' = \text{lb}/\text{ft}^2$)		
			$c' = 0$	$c' = 500$	$c' = 1000$	$c' = 2000$		$c' = 0$	$c' = 500$	$c' = 1000$
			11.1	8.5	5.9	0.5	—	—	—	—
F-F', Site 1	A	Parallel	6.5	4.9	3.4	0.3	—	—	—	—
	A	None	10.6	8.7	6.8	3.0	—	—	—	—
	B	Parallel	6.1	5.1	4.0	1.7	—	—	—	—
	B	None	10.4	8.6	6.9	3.3	—	—	—	—
	C	Parallel	6.1	5.0	4.0	1.9	—	—	—	—
	—	Parallel	—	—	—	—	7.0	16.5	—	11.0
	G-G', Site 2	Parallel	7.9	6.3	4.7	1.4	—	—	—	—
—	—	None	4.8	3.8	2.8	0.8	—	—	—	—
	—	Parallel	—	—	—	—	7.0	12.4	—	8.1

* Assumed residual friction angle on horizontal surface ($c'_r = 0$)

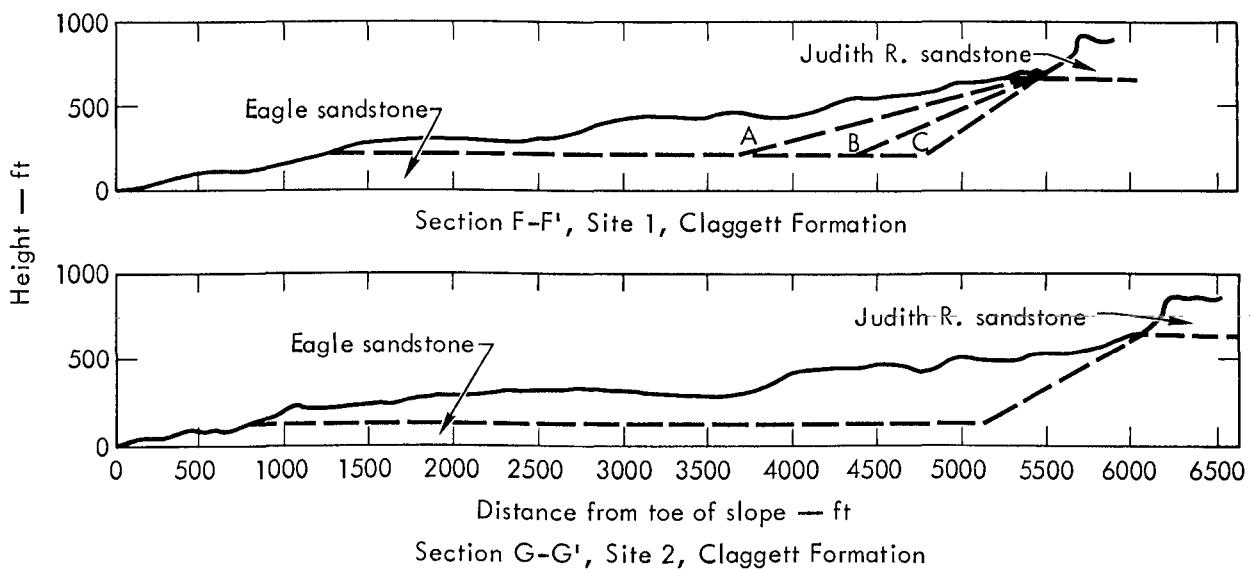


Fig. 36. Example of slope analyses for two slopes in Claggett formation.

In some slopes higher strength parameters may be obtained along planes of failure through interbedded materials. Also, the stability analyses as described above indicated larger required strength parameters than were believed possible from laboratory tests. Consequently, a second type of analysis was made in which the strength along the base of the passive wedge was assumed to be controlled by the residual friction angle alone, and the

required strength parameters were determined for the planes participating in cross-bed shear for an assumed factor of safety of unity.

The results of the stability analyses for the failed slopes are shown in Table 12. In addition to these analyses, detailed and extensive stability calculations were performed in the design and analysis of several project slopes (Appendix D). Statements of the design strengths for the

Table 12. Results of stability analyses of failed slopes

Group or formation	Site and section	Range in measured ϕ'_r	Average slope inclination (deg)	Assumed seepage condition ^a	Average required ϕ' for FS=1 ($c' \text{ lb/ft}^2$)				Average required ϕ' on boundaries of active wedge, FS=1, ($c' \text{ in lb/ft}^2$)				Infinite slope solution ϕ'_r	
					$c' = 0$	=500	=1000	=2000	Assumed ϕ'_r on passive wedge	$c' = 0$	=500	=1000		
Colorado group	1, A,A'	9.8°-7.0°	19.7	Horizontal	34.7	18.7								
				Parallel	29.2	13.6								
Marias River formation	5, E,E'		14.5	Vertical	17.0	11.7	6.1		7.5	38.4	14.9			
				Horizontal	31.5	18.6								
				Parallel	30.3	17.6								
				Vertical	18.8	14.5	10.0	6.4	7.5	31.8	22.9			
Claggett formation	1, F,F'	8.3°-5.0°	10.2	Parallel	10.4	8.6	6.9	3.3	7.0	16.5	11.0	18.1		
	2, G,G'		5.7	Vertical	6.1	5.0	4.0	1.9					10.2	
				Parallel	7.9	6.3	4.7	1.4	7.0	12.4	8.1	10.3		
	3, H,H'		5.9	Vertical	4.8	3.8	2.8	0.8					5.7	
				Parallel	9.3	7.3	5.3	1.3	7.0	13.0	8.9	10.6		
	4, I,I'		10.0	Vertical	5.9	3.4	0.8						5.9	
				Parallel	18.2	12.4	6.2						17.6	
				Vertical	10.6	7.1	3.5		7.0	13.9	10.1	10.0		
Bearpaw formation	1, K,K'	7.1°-4.0°	24.0	Horizontal	17.1									
				Parallel	16.3								38.8	
				Vertical	9.7								24.0	
	2, L,L'		12.3	Parallel	13.5	10.9	7.9	2.0	5.4	22.1	19.2			
				Vertical	9.1	8.3	5.2	1.3	5.4	25.1		21.5		
	3, M,M'		8.3	Parallel	14.1	11.0	7.9	1.5	5.4	27.1	20.7		12.3	
				Vertical	8.2	6.4	4.6	0.9					8.3	
	4, N,N'		11.8	Parallel	18.0	14.8	9.5						20.7	
				Vertical	9.1	6.8	4.6		5.4	14.1	8.1		11.8	
	5, O,O'		13.3	Parallel	20.9	16.2	11.8	1.6	5.4	16.7	10.3		23.2	
				Vertical	11.0	9.2	6.5	0.9	5.4				13.3	
Design strength for Fort Peck Dam (Lane, 1961 and Appendix D)														
Weathered clay shale and bentonite $\phi' = 10.5^\circ$ $c = 400 \text{ lb/ft}^2$ - (Backfigured from embankment slide on foundation)														
Weathered clay shale and bentonite $\phi' = 10^\circ$ $c = 400 \text{ lb/ft}^2$ - (From project slope chart)														
Firm shale and bentonite $\phi' = 20^\circ$ $c = 1600 \text{ lb/ft}^2$ - (From project slope chart)														
Fort Union group														
Cannonball	1, P,P'	24.1°-3.3°	20.3	Vertical	14.1								20.3	
Sentinel	3, R,R'		7.0	Parallel	5.2								12.5	
Butte				Vertical	3.0								7.0	
Tongue River	4, S,S'		23.3	Horizontal	33.4		25.4	14.4					37.8	
				Parallel	31.1		23.2	12.8					23.3	
				Vertical	19.6		13.6	6.8						
Design strength for Garrison Dam (Lane, 1961 and Appendix D)														
Tongue River formation $\phi' = 20^\circ$ $c = 1400 \text{ lb/ft}^2$ (From project slope chart and confirmed by laboratory tests)														
Pierre formation	1, U,U'	7.4°-4.1°	11.5	Parallel	14.1	7.9	4.6		-6.5	27.2	10.4	20.4		
				Vertical	8.4	0.9							11.5	
	2, V,V'		3.8	Parallel	6.4	1.1			6.5	6.0	0.6	6.8		
				Vertical	3.7	0.7							3.8	
	3, W,W'		8.5	Parallel	10.1	2.0			6.5	21.9	10.9	15.1		
				Vertical	6.6	1.3							8.5	
	4, X,X'		18.2	Parallel	20.6	17.5	14.1		6.5	25.4	19.4	30.4		
				Vertical	14.1	17.9	9.4						18.2	
	5, Y,Y'		5.8	Parallel	4.2	1.4			6.5				10.4	
				Vertical	3.2	1.1							5.8	
Design strength for Oahe Dam (Lane, 1961 and Appendix D)														
Clay shale with faults and bentonites $\phi' = 12^\circ$ $c = 400 \text{ lb/ft}^2$ (From design experience)														

^aVertical seepage assumption same as assuming saturated material with no seepage.

major projects in the formations under study are also indicated in the table.

Colorado Group. The failed slopes in the Colorado group were short, and infinite slope analysis was not considered applicable. Observations at the slopes led to the assumption of a basal slide along a thin bentonite seam; the base of the active wedge was assumed to be coincident with the back scarp of the slide mass. Little information is available on groundwater conditions, but surface springs or seeps were noted in the slopes.

Horizontal interbeds at the site and numerous joints, particularly in the upper 150 ft, could provide favorable drainage conditions. Analyses under the assumption of vertical seepage gave a required average residual friction angle of 17 to 19 deg with no cohesion acting, and 12 to 16 deg with 500-lb/ft² cohesion acting. For the assumption of a residual friction angle of 7.5 deg (measured range 9.8 to 7.0 deg) acting on the base of the passive wedge, a required friction angle of 32 to 38 deg was computed (cohesion equals zero) for the crossbed strength. The

required friction angle was reduced to 15 to 23 deg by assuming that the cohesion along the crossbed shear plane could be as high as 500 to 1000 lb/ft².

Claggett Formation. The Claggett slopes were all long and relatively flat; differences in computed friction angles by infinite slope or wedge methods were not significant. Because of the extremely flat slopes little difference in required strength was found by assuming horizontal or parallel seepage patterns. Ponding and springs observed on the south side of the river indicated that the seepage conditions could be extreme for these slopes; on the north side these surface expressions were absent. Required friction angles of 8 to 10 deg were computed for the south slopes under adverse seepage, and 11 deg for the north slope (site 4, section I-I') for favorable seepage. These values compare very favorably with measured residual friction angles of 8.3 to 5.0 deg.

Bearpaw Formation. It was believed that slopes in the Bearpaw had a more favorable drainage condition than the Claggett slopes. This condition was reflected in part by the slightly steeper slopes in the Bearpaw. Surface expressions of seepage were also absent at the sites studied. For the most favorable seepage condition, required residual friction angles ranged from 8 to 12 deg; for unfavorable seepage the required friction angle varied from 11 to 16 deg. These data agree very well with the adopted strength parameters at Fort Peck Dam of 400 lb/ft² for the cohesion and 10 to 10.5 deg for the friction angle.

Fort Union Group. Data previously presented for the Fort Union group indicated wide ranges in measured residual angles and observed slope heights and inclinations. Back-calculated strength parameters reflected similar wide ranges, and are therefore not very conclusive. For example, one slope (site 1, Section P-P') located within the Cannonball formation indicated a back-figured residual friction angle of 14 deg. A second slope (site 3, Section R-R'), located in the Sentinel Butte formation, indicated a back-figured residual friction angle of 3 to 5 deg, depending on the assumed seepage conditions. The one failed slope in the Tongue River formation (site 4, Section S-S') indicated an average residual friction angle varying from 20 to 30 deg. The required friction angle, assuming an average cohesion of 1400 lb/ft² (not shown in Table 12), was 19 deg for the most unfavorable seepage conditions. These latter strength parameters compare to strength parameters of 20 deg and 1400 lb/ft² used for the design of Garrison Dam in the Tongue River formation.

Pierre Formation. Slopes studied in the Pierre formation exhibited surface expressions of seepage, indicating that unfavorable conditions probably existed at the slopes. Incised gulleys at site 2 (Section V-V') and site 5 (Section Y-Y') indicated that extensive erosion had taken place to reduce the slope inclinations. Significantly, the required residual friction angle for parallel seepage was about 4 to 6 deg, which was within the range of values for the measured residual friction angle (7.4 to 4.1 deg). It was believed that site 3 (Section W-W') had the failure base

near the Pierre-Niobrara contact; the required friction angle was 10 deg for seepage parallel to the slope. Sites 1 and 4 (Sections U-U' and X-X', respectively) are located primarily within the Verendrye and Gregory members of the formation. The design strength parameters for Oahe Dam (located mostly in the same members) were 12 deg and 400 lb/ft². Calculated friction angles for an assumed cohesion of 500 lb/ft² were 8 and 18 deg.

DIFFERENCES IN PHYSICAL PROPERTIES

The principal differences in slope behavior in the Pierre, Bearpaw, and Claggett on the one hand and the Colorado and Fort Union on the other are related to structural defects, principally old failure planes and soft seams. The fundamental differences that led to the formation of the defects remain unexplained. The absences of old failure planes away from the river valley strongly suggest that the structural defects are gravity-induced and not the result of deep-seated tectonic activity.

In general, the two more stable materials, the Colorado and Fort Union, represent the extremes of measured properties in the five materials studied. However, a gross comparison of all the properties measured for the five materials reveals considerable overlap. The plasticity, moisture content, consolidation characteristics, unconfined compressive strength, petrography, and residual strength are slightly different for each formation. There is almost as large a range in measured properties within a formation as between formations, and the gross differences in physical behavior

cannot be directly related to any specific test or series of tests.

The Colorado and Fort Union groups possess cementation and lack homogeneity of lithology. The Colorado consists principally of shale with scattered bentonite layers but also contains numerous very thin layers of siltstone. The potential significance of the siltstone layers was overlooked during initial testing, so that most of the tests were performed on the clay shale. Slaking tests performed on the silty layers did not cause any significant breakdown even after six cycles of wetting and drying. Scattered slickensides in boring samples suggest that movement has occurred in the Colorado slopes. However, individual beds can be traced intact along continuous outcrops for hundreds to thousands of feet. It is suggested that movements have occurred along shale or bentonite layers and that the movement was nearly horizontal. The siltstone layers, while not an imposing feature in the core samples, apparently greatly enhanced the stability of the slopes by increasing the cross-bed shear strength and possibly by decreasing lateral expansion in the formation.

The lack of material homogeneity is obvious in the Sentinel Butte formation of the Fort Union group. Less than half the materials in the borings were classified as clay shale; the remainder were lignite, silty sand, and sandstone. These materials provide better drainage and higher cross-bed shear strength. The high cross-bed strength apparently restricts the development of progressive failures back into the slope. Additionally, the results of slaking and insoluble residue tests suggest very slight

cementation in the upper portion of the slopes.

On the other hand, the Pierre, Bearpaw, and Claggett formations appear to be relatively homogeneous sequences of clay shale

with scattered layers of bentonite. No significant zones of strong cementation were found. For gross comparison, the cross-bed strength may not be greatly different from the strength along the bedding.

Section 4. Assessment of Important Characteristics of Clay Shale

IMPORTANT CHARACTERISTICS

This section contains a discussion of the more significant characteristics of clay shale with respect to slope behavior in natural and cratered slopes. Differences in the natural slope configurations of the five materials studied during this investigation are reflected in the laboratory test data and geologic descriptions of the mapped slopes. However, as discussed in the previous section, these differences for the most part are subtle.

The most significant differences among the high natural slopes in the five formations studied appear to be caused by structural defects in the clay shales. The defects were mainly in the form of old failure planes, but intense slickensides would probably produce similar results. The natural slopes in the Colorado and Fort Union groups were not failed throughout, as were slopes in the Claggett, Bearpaw, and Pierre formations. Slopes in predominantly unfailed materials, as at Garrison Dam were successfully designed using slope charts with proper seepage assumptions. Slope charts developed for the Pierre at Oahe Dam and the Bearpaw at Fort Peck Dam were not as reliable. Slope failures occurred at both projects

and were associated with movement along discontinuities and softer seams, usually bentonite layers. A slope chart developed from natural slopes but containing only failed slopes cannot be directly utilized for design purposes. In such cases, it is necessary to determine the distribution, attitude, and strength of the weak members, as well as the seepage conditions. High slopes in these materials are generally inclined at angles within their range of measured residual friction angles. Natural slope inclinations in the Bearpaw, where capped or underlain by a more stable sandstone formation, were somewhat steeper than their range in residual friction angles. However, those slopes without a stronger member in the section were inclined as flat as or flatter than the residual friction angle.

The strength or stability of those slopes in which a network of discontinuities or closely spaced slickensides exists can apparently be best analyzed by back-figuring a failed slope. This back-figuring allows two parameters, the cohesion and the effective angle of internal friction, to be varied and studied.

Shear strength parameters acting at the time of a failure in a slope can be

determined successfully by reconstructing the slope geometry and groundwater conditions in a back-figuring process. The exercise of determining shear strength parameters by back-figuring from failed conditions at a given location has certain advantages over the familiar determination from slope charts. The actual position of a failure surface can be utilized, groundwater conditions can be included, and finally the preslide geometry, in the case of excavated slopes, can be used in the analysis. At the same time, several disadvantages are offered. The method currently available utilizes a limiting equilibrium between a required strength and shear stresses caused by gravity loading. One factor shown in Section 2 to be important to clay shale behavior is initially high horizontal stresses, and these cannot be included in a limit-type analysis. Second, the back-figured strength parameters represent only average conditions and do not adequately reflect spatial variation in stresses, nor do they locate overstressed regions within the mass. Finally, analytical methods may be applied in many situations only with the aid of several assumptions. For example, in Section 3 of this report, the results of analyses for the slopes along the Missouri River were shown with gross assumptions made regarding actual groundwater conditions and the locations of failure surfaces.

With the disadvantages cited, it is still beneficial to compute shear strength parameters in as many slopes as possible in a given area. Such analyses, particularly if performed on excavated slopes in shales, give insight as to the effect of time in decreasing the applicable shear

strength of a material. An illustration of this use is summarized by Johnson (1969); also, see Fig. 16. In an area relatively free of slickensides or previously failed material, it may be appropriate to design on the basis of the fraction of the shear strength at failure by comparing the projected life of the structure to the time of failure of the excavated slopes. In many cases, particularly in material containing many slickensides and other evidence of preconstruction failures, analysis of failed slopes will illustrate strength parameters comparable to residual strength. In such cases, the analyses serve only to verify the precariousness of any slope construction in the area, which is also readily recognized by cursory examination of the local topography. For these situations, only rather simple analyses, such as the infinite slope method, are usually sufficient to illustrate the existing conditions. In some clay shale formations, such as the Colorado group, the stratigraphy has a pronounced effect upon the stability of slopes within the formation. In the Colorado group, back-figured strength values gave some estimate of strength values applicable to cross-bed shear. For design purposes, this information can be of vital importance in estimating stable construction slopes.

ASSESSMENT OF LONG-TERM STABILITY OF NUCLEAR CRATER SLOPES IN CLAY SHALE

General

A nuclear or high explosive cratering detonation in rock or other cohesive material produces a crater with certain predictable characteristics. A cross section of one-half of a row crater, normal

to the longitudinal axis, is illustrated in Fig. 37, with dimensions scaled in feet divided by explosive yield (of each single charge; in kilotons) to the one-over-3.4 power. Experience has indicated that such a scaling relation gives a good approximation of final crater dimensions for explosives buried at optimum depth in a variety of materials.

The materials comprising the crater are conveniently divided into three zones (Fisher, 1968); the fallback and ejecta zone, the rupture zone, and the zone of intact material. The fallback consists of materials that have experienced significant disarrangement and displacement and have come to rest within the crater; the ejecta consists of material thrown out above and beyond the apparent crater. Both ejecta and fallback have characteristics similar to rubble or dumped rock. The rupture zone, extending outward from the true crater, has experienced crushing and fracturing. In this zone there are significant displacements but the material

has not been internally disarranged, in contrast to the fallback and ejecta. Rupture-zone materials are similar to highly jointed or fractured rock. Beyond the rupture zone the materials are not significantly altered from their preshot state and are termed intact material, although pre-existing natural joints or fractures are present.

The assessment of crater slope stability usually involves an estimation of the strength and seepage characteristics of the various crater zones and an assessment of the response of the materials to weathering processes. Specific data from cratering experiments are sparse, but collectively they suggest that the initial crater configuration is stable. Outside of minor sloughing of fallback materials, there have been no slope failures in any of the experimental craters produced by nuclear or high explosives to date. However, the experimental crater slopes in clay shale have shown evidence of distress, as discussed below.

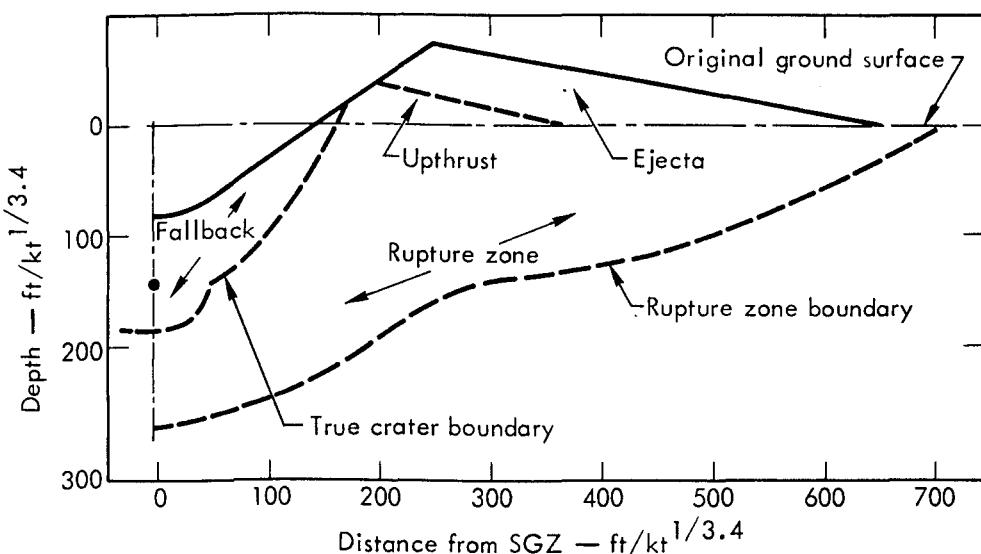


Fig. 37. Profile perpendicular to longitudinal axis of row crater in hard work.

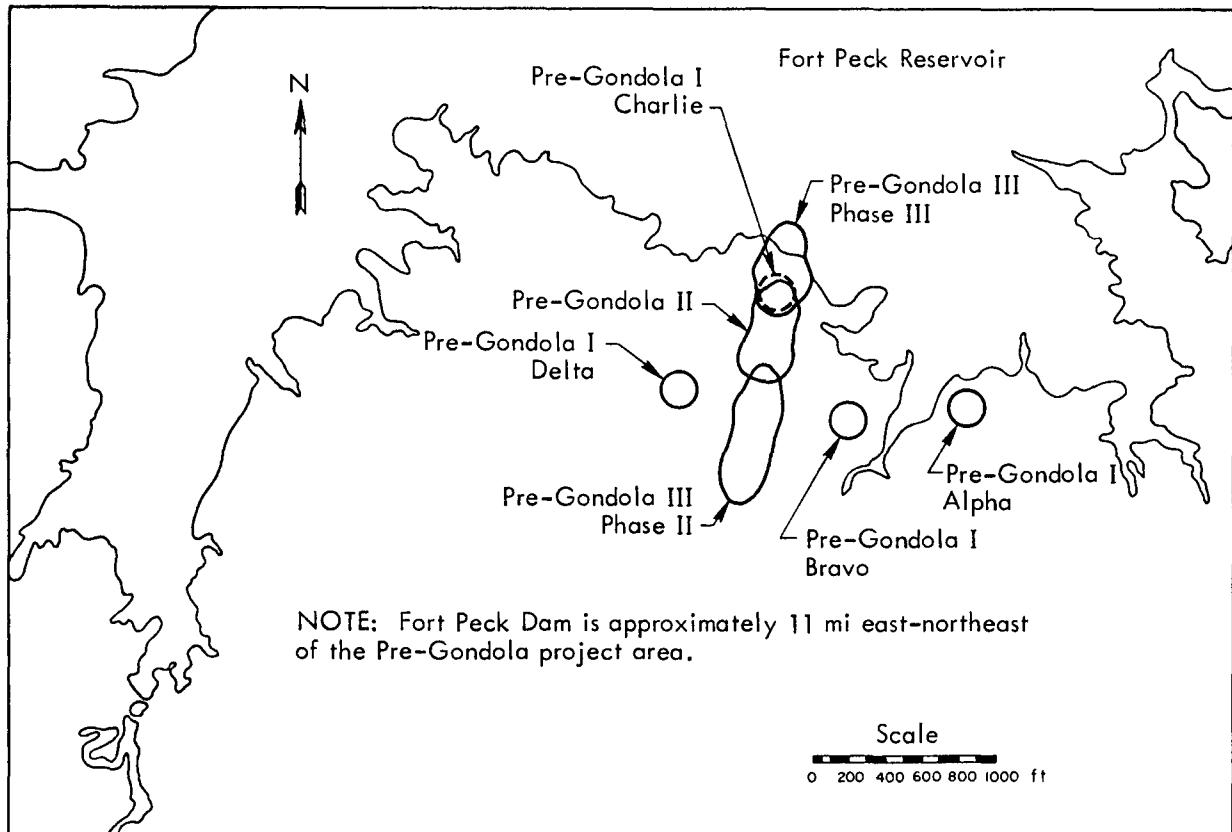


Fig. 38. Location of Pre-Gondola crater, Montana.

Crater Slopes in Clay Shale

Cratering experience in clay shale is limited to the Pre-Gondola series of high explosive cratering experiments conducted by NCG in the Bearpaw formation at Fort Peck, Montana (Fig. 38, Table 13). The experimental program was started in June 1966, and observations have been continued to the present (1970). Included in the test program were a 1000-lb site-calibration series, four single cratering detonations each of 20 tons yield (Pre-Gondola I), a 140-ton row crater (Pre-Gondola II), a 210-ton connecting row experiment (Pre-Gondola III Phase II) and a 70-ton row experiment connecting the previously excavated row crater to the Fort Peck Reservoir (Pre-Gondola III Phase III). A cross section through the

Pre-Gondola III Phase II connecting row crater, showing the crater dimensions and the distribution of ejecta, fallback, up-thrust zone, and rupture zone, is presented in Fig. 39. The maximum total slope heights of the Pre-Gondola craters are on the order of 75 ft. It is exceedingly difficult to project data from such slopes to planned projects that may be several hundred feet high, but several observations have been made which are pertinent.

Routine laboratory tests including water content, density, unconfined compression, and triaxial shear tests on preshot and postshot samples from Pre-Gondola II indicated that no significant changes were induced in the intact clay shale fragments as a result of the cratering event (Fisher et al., 1969). However,

Table 13. Pre-Gondola crater data^a

Event	Approximate total yield (tons)	Depth of burst (ft)	Average lip crest radius (ft)	Total slope height (ft)	Overall inclination (deg)	Date
Pre-Gondola I						
Alpha	1 charge, 20	52.7	100.4	46.0	25	1 Nov 66
Bravo	1 charge, 20	46.3	102.1	43.2	23	25 Oct 66
Charlie	1 charge, 20	42.5	101.8	47.1	25	28 Oct 66
Delta	1 charge, 20	56.9	94.5	38.2	22	4 Nov 66
Pre-Gondola II						
Row charge	5 charge, 137	53.1 ^b	141.5 ^b	73.6 ^b	27	28 Jun 67
Pre-Gondola III						
Phase II connecting row	7 charge, 210	53 ^b	152 ^b	78 ^b	27	30 Oct 68
Phase III row charge connecting to reservoir	5 charge, 70	36 ^b	120 ^b	58 ^b	29	6 Oct 69

^a Not all events in the Pre-Gondola series are included.

^b Average

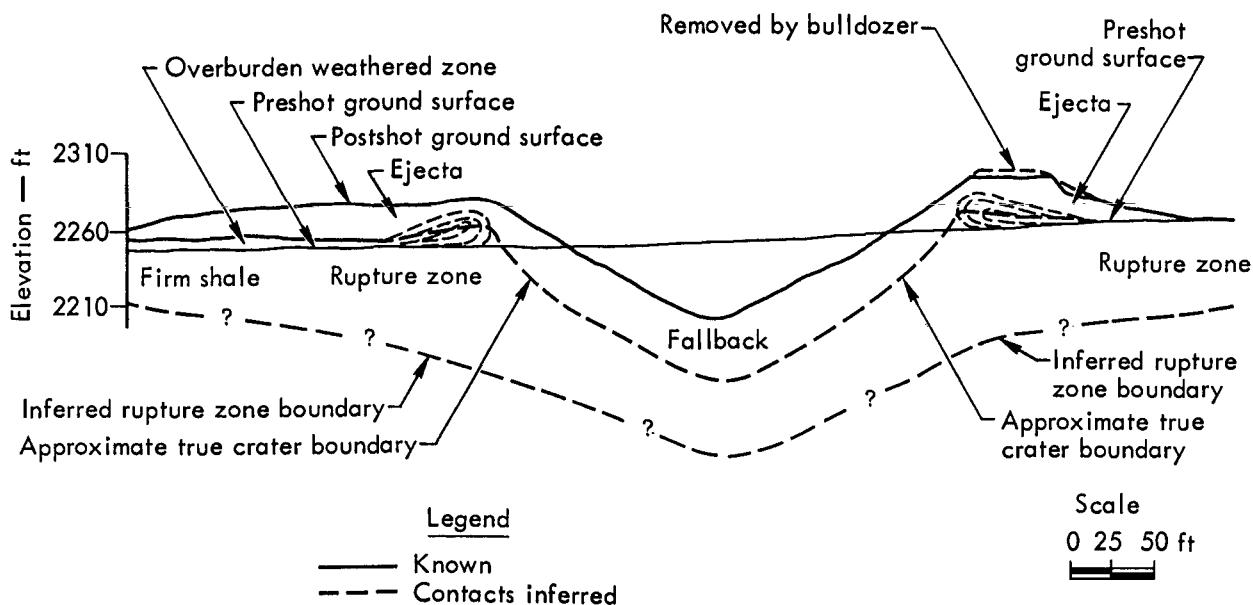


Fig. 39. Postshot cross section through Pre-Gondola III Phase II crater.

as would be expected, a large number of new fractures were developed; and although no large-scale field shear tests were performed, the mass strength of the cratered shale was probably reduced.

The orientation, frequency, and distribution of blast-induced fractures cannot be accurately predicted. Furthermore, the amount of displacement caused by the event along pre-existing and blast-induced

fractures cannot be estimated for available data. Skempton and Petley (1967) reported that displacements of about 3 mm along natural joints are sufficient to reduce the strength to the residual value. The Pre-Gondola crater slopes are standing more steeply than the residual frictional strength of the Bearpaw shale, but the slopes are only 4 yr old or less and not particularly high. Cracking did occur at the top of the Pre-Gondola I (Bravo) crater, where the total slope height was 43 ft, after some fallback materials were removed for gradation measurement and the slope inclination was somewhat steeper than the 23 deg reported in Table 13 (Fig. 40).

Figure 41 is a view of the Pre-Gondola III Phase II row crater, with the

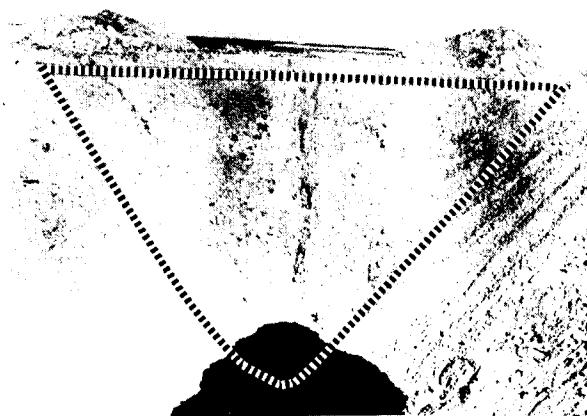


Fig. 40. Pre-Gondola I (Bravo) crater, September 1967, showing steep slopes.



Fig. 41. Pre-Gondola III Phase II crater, September 1969.



Fig. 42. Pre-Gondola row crater, June 1970, showing extensive cracks.

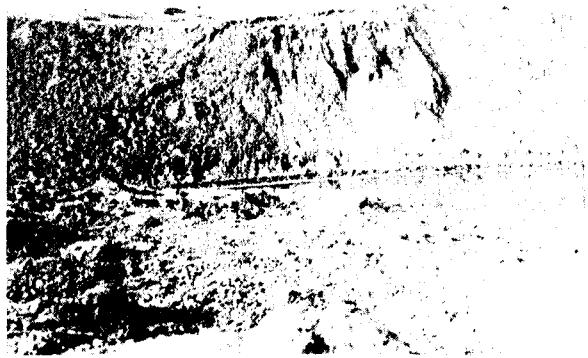


Fig. 43. Pre-Gondola row crater, June 1970, showing surficial sliding.

Pre-Gondola II row crater in the background, about 1 yr after the Pre-Gondola III detonation and over 2 yr after the Pre-Gondola II event. The materials at the surface of both craters have slaked to a depth of about 1 ft, forming an im-

pervious blanket. The water in the craters was from surface runoff. Note the rills on the crater slopes and the evaporation marks around the edges of the pools. The fragments seen on the surface crumble when disturbed.

The slopes in the Pre-Gondola II and Pre-Gondola III row craters have undergone progressive movements, although no massive slope failures have occurred. The movements are evidenced by extensive cracks which have noticeably widened since first they were observed just below the crater crest on the east side of the channel created during Pre-Gondola III Phase III (Fig. 42). The number of cracks has since significantly increased; cracks have increased in width from less than an inch to 2 to 6 in.

Observations indicate that near the channel entrance extensive surficial sliding is the prominent type of failure (Fig. 43). The eroding and wearing back of the toe by wave action have oversteepened and weakened the slope, and may be causing some of the observed motion. Failure is evidenced by slides that terminate in talus cones at the toe of the slope. The slope in the upper part shows a definite step-like form (Fig. 44).

Movements of the crater slopes were monitored by surveying the positions of surface pins. Data indicate that movements are occurring, but do not delineate the seat of movement. Movements are accentuated near the channel entrance, with only small cracks along inner portions of the channel, Fig. 45. Observations have shown that while cracks have developed, most movement has been evidenced by surficial sliding. This slightly unstable condition has not seemed to impair the



Fig. 44. Pre-Gondola row crater, September 1970, showing step-like surface.



Fig. 45. Pre-Gondola row crater, June 1970, showing inner end of channel.

navigational prism to any discernible degree.

The effects of seepage must also be evaluated as a time-dependent phenomenon. Piezometers were installed to record groundwater levels both before and following the Pre-Gondola III Phase II experiment (Fig. 46). The preshot groundwater level was approximately 35 ft below the ground surface (Fig. 47). Postshot meas-

urements (Fig. 48) indicated a drastic depression of the groundwater level to more than 70 ft below the preshot level and more than 80 ft below the water level in the Fort Peck Reservoir. The depression of the water surface results from the increased permeability created by the large number of open fractures developed in the rupture zone and to some extent by the reduction in total load in the zone. The steady rise in the water surface indicates that the fractures remained open and that water migrated through the rupture zone. The groundwater level rose approximately 20 ft from June 1969, to September 1969 (Fig. 48). The slaking of surface material and the overburden pressures apparently had little short-term effect on the permeability of the shale at the Pre-Gondola III site.

The Pre-Gondola III Phase III experiment, which connected the Pre-Gondola III Phase II row crater with the Fort Peck Reservoir, produced a marked and rapid rise in the groundwater level below the Pre-Gondola III Phase II crater (Fig. 49). The groundwater level reached a stabilized position near the reservoir level in approximately 1 mo. Continued monitoring of the piezometers to 31 May 1970 indicated a steady groundwater level with fluctuations corresponding to changes in the reservoir water level.

With time, the initially stable, relatively steep slopes have experienced some slope adjustments. Thus slopes intended to be stable for a reasonable engineering project life of perhaps fifty years would have to be somewhat flatter than those now existing. The optimum slope angle cannot be predicted on the basis of this one set of experiments. Additional tests at

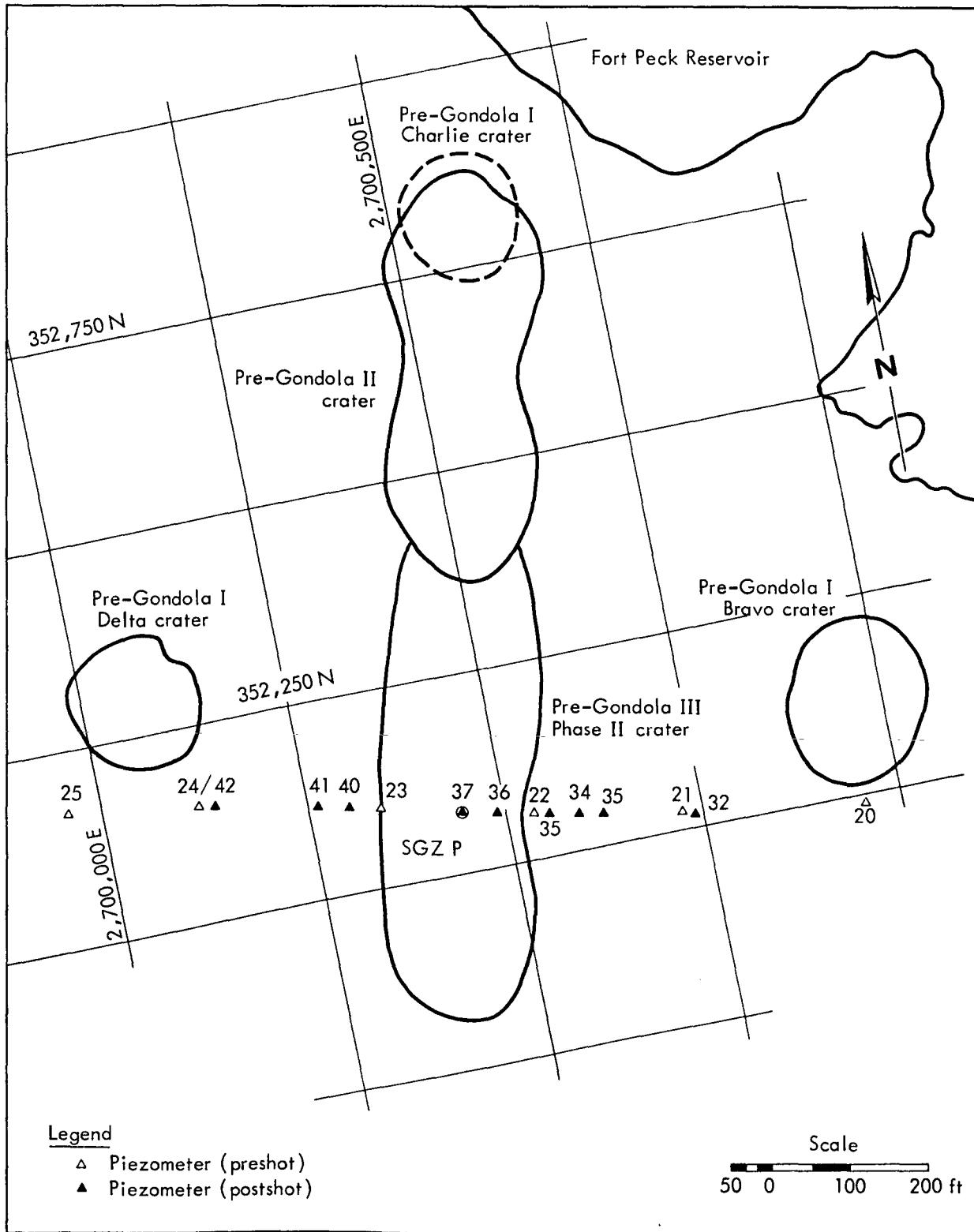


Fig. 46. Pre-Gondola III Phase II postshot piezometer locations.

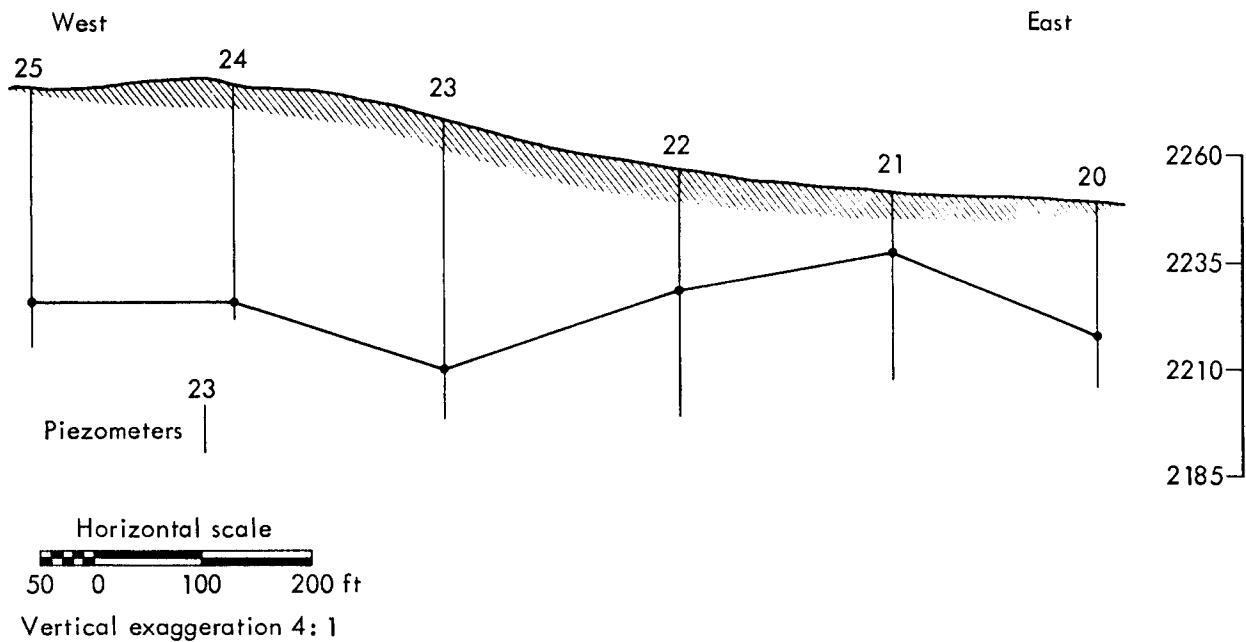


Fig. 47. Pre-Gondola III Phase II preshot groundwater level.

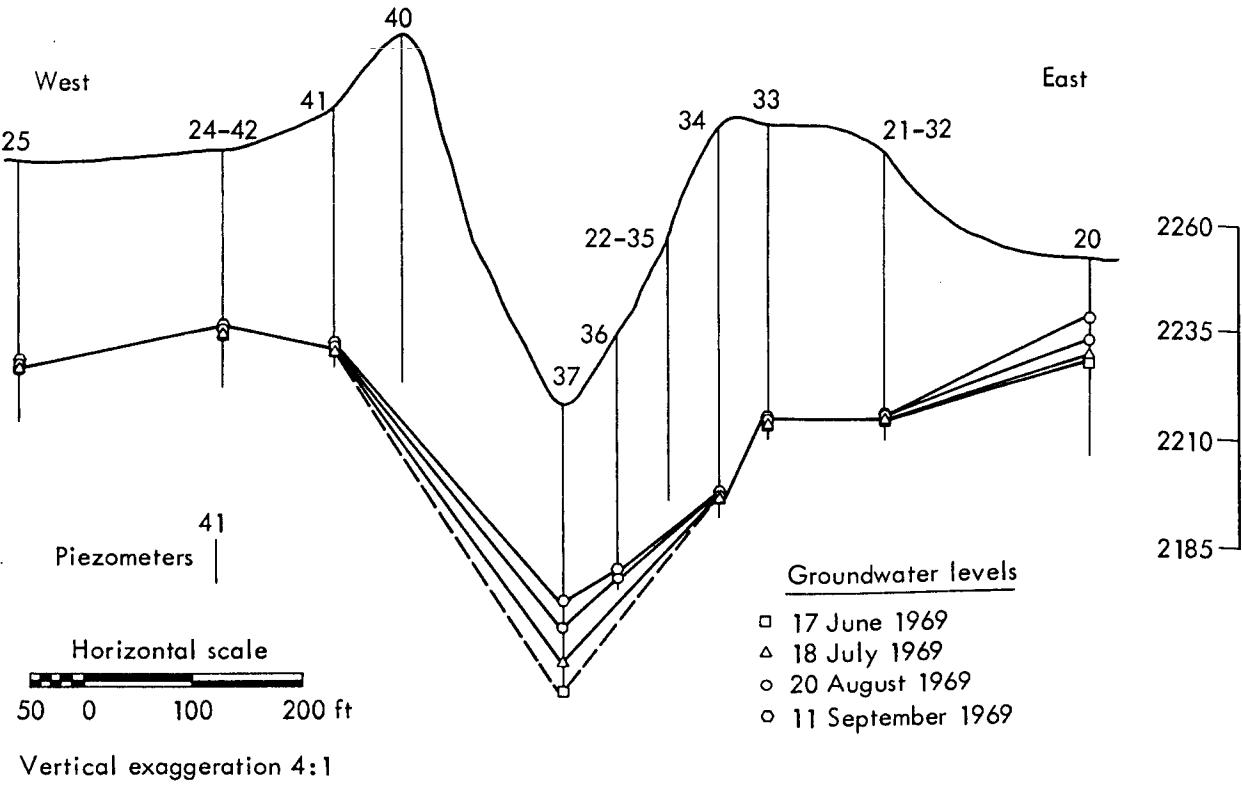


Fig. 48. Pre-Gondola III Phase II postshot groundwater level.

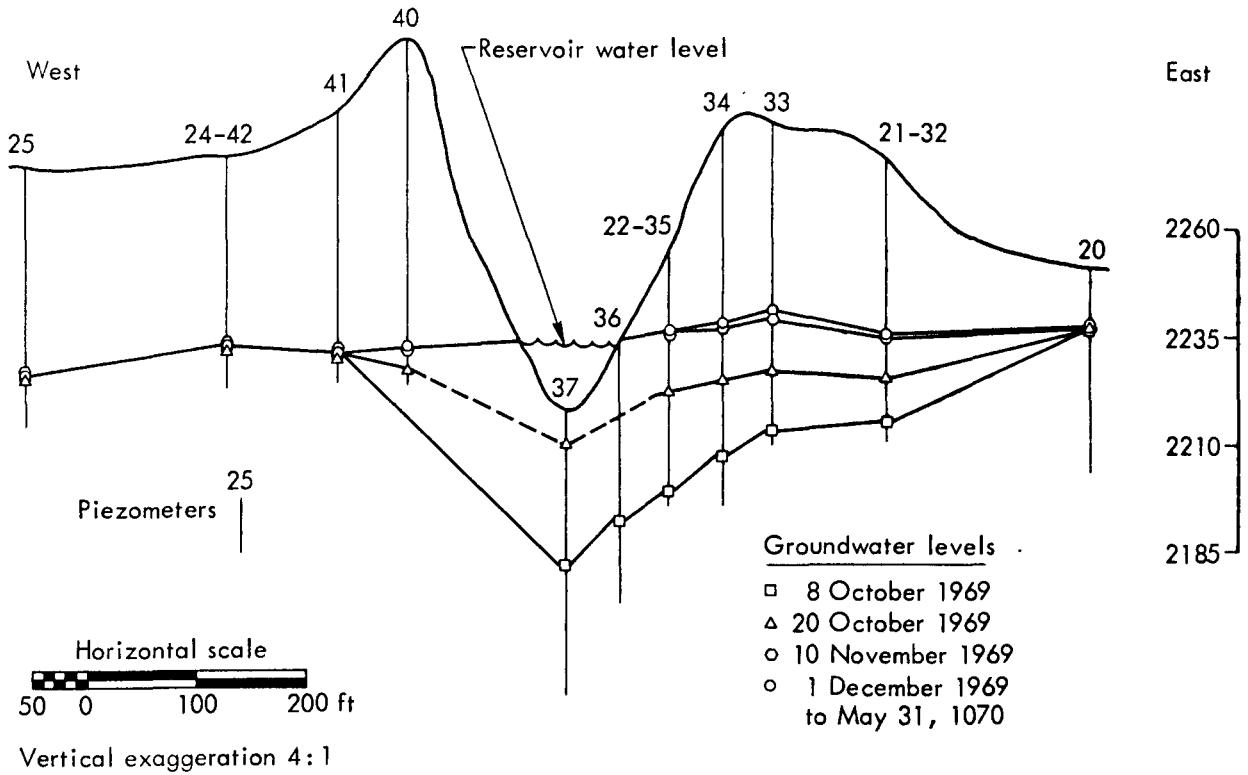


Fig. 49. Pre-Gondola III Phase II groundwater levels following Phase III detonation.

varying depths and additional time to allow the resulting slopes to respond are needed to assess the stability characteristics. Until such results are available, the

design of slopes for explosive excavations in clay shale will be strongly influenced by experiences with natural and conventionally excavated slopes.

Section 5. Summary and Conclusions

The degree of overconsolidation and the lithology, both reflections of geologic history, are the principal features determining the slope behavior of clay shales, which have long been associated with slope instability and construction problems. The overconsolidation of clay shales is related to troublesome rebound phenomena, such as swelling, high residual horizontal stresses, and the development of fissures. Important features of the lithology are the constituent minerals (particularly the clay minerals), the mechanical composition

(particularly the clay-size fraction), the nature of the bonding of the particles, the presence or absence of any cementing agent, and the degree of homogeneity of the material. Homogeneous marine-deposited clay shales are particularly susceptible to sliding failure.

The local geologic structure and water conditions are also important stability factors. The same material may be stable in an arid environment but marginal or unstable in a slightly wetter climate. The presence of unfavorably oriented weak

layers, such as bentonite beds, may present serious stability hazards. On the other hand, the presence of favorably oriented slightly stronger beds, such as cemented silts or sands, either interbedded within the clay shale or at its base or top, may allow steeper stable slopes than would otherwise be possible.

Time is another important factor in clay shale behavior. Slopes may remain standing for extended periods but eventually fail through a complex interaction of progressive failure and reduction in effective shear strength, thus yielding lower overall slope inclinations. Slopes that have previously failed may be found standing at low or marginal factors of safety; any attempt to steepen such slopes, as by excavation, reactivates the material and leads to renewed movements.

The natural slopes studied as the major part of this investigation in five clay shale units in the upper Missouri Basin showed the effects of all these factors in varying degrees. In the Claggett, Bearpaw, and Pierre formations (all marine-deposited clay shales of Late Cretaceous age), slopes were commonly failed throughout, and while overall slope heights amounted to several hundred feet, inclinations were typically very low (5 to 10 deg). Excavations into these materials have usually exposed old failure surfaces, on which renewed sliding occurred. Slickensides were especially common in these three materials. Cementation was minimal, with the exception of parts of the Pierre. Slope inclinations in these three materials are comparable to their residual angles of internal friction. Where the Judith River sandstone formed a rigid base, certain slopes in the Bear-

paw stood slightly steeper than elsewhere.

Slope failures in the Colorado group, also a marine deposit of Late Cretaceous age, were relatively uncommon, and where they occurred, slide masses were discrete and moved along single failure surfaces. Cementation was fairly well developed in the Colorado group, and probably as a result thereof, slope inclinations in the unit commonly were steeper than the residual angle of internal friction of the materials. Slickensides were present but not common.

The Paleocene (lowermost Tertiary) Fort Union group, composed of one marine and two continentally deposited formations, generally stood less steeply and showed more failures than the Colorado group, but stood steeper with fewer failures than the Claggett, Bearpaw, and Pierre formations. Notably, there were wide intraformational as well as the interformational differences within each of the five units. Bentonitic layers with high plasticity characteristics, high swelling potential, and low residual shear strength were found in all of the units.

Extensive laboratory testing documented in detail such properties as Atterberg limits, natural water contents, preconsolidation pressures, swelling and shrinkage behavior, and residual shear strengths. Variations in properties were determined for the different materials, but the properties of the five formations showed considerable overlap. Since there was almost as large a range in measured properties within a formation as between formations, it appears that only gross differences in slope behavior can be related to any specific test or series of tests.

In line with the above test observations, it was found that both in major engineering projects in the upper Missouri Basin and in other clay shale areas generally, slopes were customarily designed on an empirical basis. The principal factors of concern were local site geologic and hydrologic conditions, and the chief bases for judgment were existing nearby slopes in the same materials. Moreover, design slope charts and curves have been most successfully used where local structural and seepage conditions have been taken into account. Laboratory strength determinations have played an integral, but subordinate, part. Conservatism in slope design has generally been vindicated, since time-dependent phenomena have caused some slopes that stood apparently stable for varying periods

of time to eventually experience some distress.

The only experimental crater slopes in clay shale (at the Pre-Gondola project of the Nuclear Cratering Group at Fort Peck, Montana) have been subject to observation for only four years or less and show signs of potential future distress, although no massive failures have yet occurred. It appears that in the future cratered slopes for engineering purposes must be designed largely on the basis of experience with conventionally excavated slopes, and that the design must be conservative. No experience to date indicates that it is necessary to design slopes as flat as the residual angle of internal friction of the materials, although such an inclination would be the lower limit to which slopes might be designed.

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Printed in USA. Available from the National Technical Information Center, National Bureau of Standards, U. S. Department of Commerce, Springfield, Virginia 22151
Price: Printed Copy \$3.00; Microfiche \$0.65.

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Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY (Corporate author)	2a. REPORT SECURITY CLASSIFICATION UNCCLASSIFIED
U.S. Army Engineer Nuclear Cratering Group Lawrence Radiation Laboratory Livermore, California 94550	2b. GROUP
3. REPORT TITLE	

EMPIRICAL STUDY OF BEHAVIOR OF CLAY SHALE SLOPES

4. DESCRIPTIVE NOTES (Type of report and inclusive dates)

Final Report

5. AUTHOR(S) (First name, middle initial, last name)

R.W. Fleming, G.S. Spencer, and D.C. Banks

6. REPORT DATE	7a. TOTAL NO. OF PAGES	7b. NO. OF REFS
December 1970	Vol 1 - 101	
8a. CONTRACT OR GRANT NO.	8b. ORIGINATOR'S REPORT NUMBER(S)	
	NCG Technical Report No. 15	
b. PROJECT NO.	9b. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)	
c.		
d.	None	

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Distribution of this report is unlimited

11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY
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13. ABSTRACT

This study was undertaken to determine the factors that lead to instability in clay shale to provide a basis for assessing the probable long-term stability of high crater slopes in clay shales.

The chief features contributing to the engineering behavior of clay shales are degree of overconsolidation and lithology, both of which reflect geologic history. Local geologic structure and hydrologic conditions affect individual slopes. Weak layers may present slope hazards, while conversely a few stronger layers may materially strengthen an entire clay shale slope. Time-dependent phenomena are important in clay shale slopes, which may fail after standing apparently stable for many years.

Intensive studies were conducted of natural slopes in five clay shale units in the upper Missouri Basin. The Claggett, Bearpaw, and Pierre formations, all marine-deposited shales of Late Cretaceous age, showed extensive slope failures; high slopes generally stood at overall inclinations of only 5 to 10 degrees, which are comparable to the residual angles of internal friction of the materials. The Cretaceous Colorado group and Lower Tertiary Fort Union group showed steeper slopes with fewer failures. Intensive laboratory testing of physical properties of all materials indicated that only gross differences in slope behavior can be related to any specific test or series of tests.

Engineering practice in clay shale materials is customarily based on an empirical approach, attention being given to local site conditions and to the observed behavior of existing nearby slopes. Limited observations of experimental crater slopes in clay shale are available. The conclusion is reached that (continued)

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Nuclear Construction Engineering Nuclear Cratering Nuclear Construction Technology Slope Stability Clay Shale						

13. Abstract (continued)

future design crater slopes must be based on experiences with conventionally excavated slopes. Furthermore, the design of cratered slopes must be conservative, although no experience indicates that the slope inclination needs to be as flat as the residual angle of internal friction of the site materials.

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