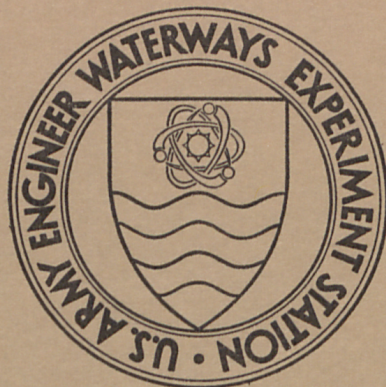


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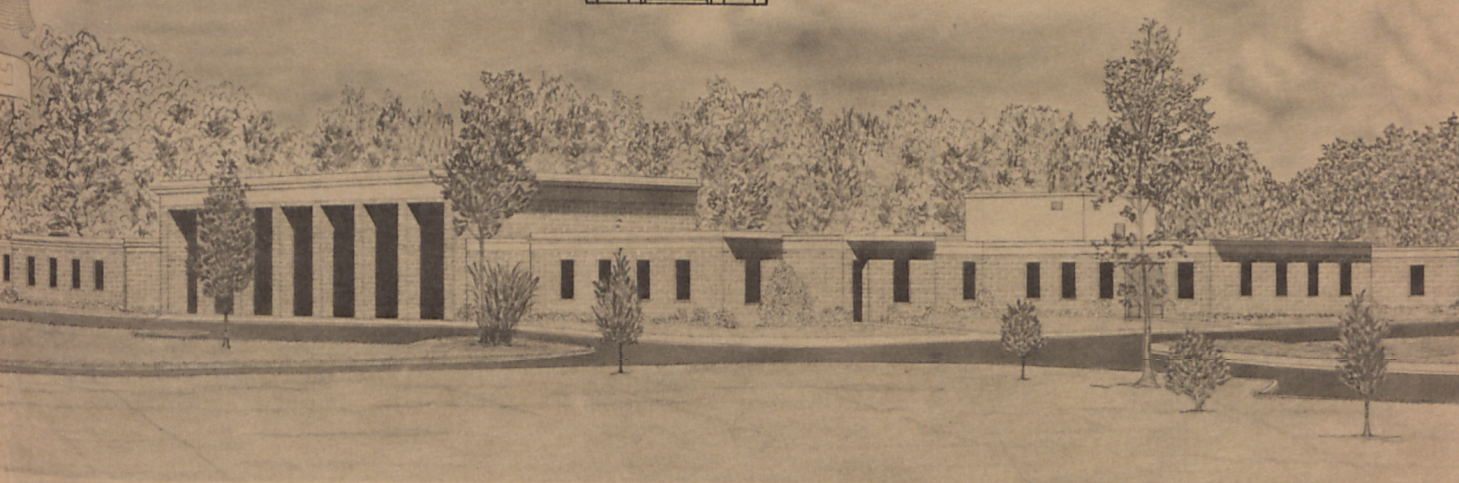


MISCELLANEOUS PAPER S-71-22

STUDY OF CLAY SHALE SLOPES

by

D. C. Banks



October 1971

Conducted by U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

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FOREWORD

This paper was presented at the Thirteenth Symposium on Rock Mechanics: Stability of Rock Slopes, which was held from 30 August through 1 September 1971 at the University of Illinois, Champaign-Urbana, Illinois. The paper was presented by Mr. D. C. Banks, Rock Mechanics Section, Soils Division, U. S. Army Engineer Waterways Experiment Station (WES), and comprises a synopsis of studies, performed in part at WES, of stability of clay shale slopes along the Missouri River Valley and the Panama Canal.

The work described was conducted and this paper prepared under the general supervision of Messrs. W. C. Sherman, Jr., Chief, Soil and Rock Mechanics Branch, and J. P. Sale, Chief, Soils Division. Director of WES was COL Ernest D. Peixotto, CE; Technical Director was Mr. F. R. Brown.

STUDY OF CLAY SHALE SLOPES

By Don C. Banks¹

ABSTRACT

The paper gives information on the stability of clay shale slopes from studies of the geology and history of sliding, determination of physical properties from laboratory tests, and stability analyses of the slopes. Slopes in five different formations along the upper Missouri River were studied. The most expedient manner to view the stability characteristics of these slopes was through empirical slope charts. Design experience in the study area had used similar approaches with success when local site geologic and hydrological conditions were considered. In the Panama study, detailed records were available to reconstruct the sliding history and allow limit type analyses to be performed. Both approaches can be utilized to determine design slopes by giving proper recognition to the limitations of each.

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STUDY OF CLAY SHALE SLOPES

By Don C. Banks¹

INTRODUCTION

The U. S. Army Engineer Waterways Experiment Station (WES) has recently participated in two studies relating to the stability of clay shale slopes. The first study, sponsored by the U. S. Army Engineer Nuclear Cratering Group (NCG), concentrated on slopes found in five geologic formations or groups along the upper Missouri River valley (Fleming, Spencer, and Banks, 1970). The purposes of the study were to define characteristics typifying a potentially troublesome clay shale, to determine factors leading to instability, and to provide a basis for assessment of the long-term stability of high crater slopes in clay shale. The second study, sponsored by the Panama Canal Company (PCC) and, during the first year, by the Atlantic-Pacific Interoceanic Canal Study Commission and, during the remaining four years, by the Office, Chief of Engineers (OCE), is a continuing study to assemble information about slope stability along the present Panama Canal that will assist operation and maintenance and to develop new information that will provide a basis for more rational, economical, and reliable slope design for a new canal. During the first year of the study, the data collected and analyzed dealt mainly with the East and West Culebra slides and the Model Slope (Lutton and Banks, 1970).

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Previous studies by others, dealing with identification of clay shale materials, testing techniques for index and physical properties, delineation of in situ conditions and determination of factors affecting clay shale behavior, have indicated the complex nature of problems in clay shale. From studies, such as discussed in this paper, the various parameters affecting slope design are brought into focus to allow the designer to make predictions of slope behavior in other situations.

SIGNIFICANT RESULTS FROM THE EMPIRICAL STUDY OF SLOPES ALONG THE UPPER MISSOURI RIVER VALLEY

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 of the Fort Union group, and the Pierre formation, Fig. 1. The study consisted of collecting data from field mapping and office studies of aerial photographs, review of literature referring to the various geologic units, boring and sampling of selected sites, laboratory tests of selected samples, case history studies of the major dam projects in the area, and a survey of clay shale slope behavior from other geographical areas.

A brief lithological description of the five geologic units studied is shown in Table 1. From field studies, detailed topographic and surface stratigraphy were determined; some information was obtained on material characteristics from shallow samples. Additional slope data were collected from aerial photographs.

Slope Behavior

In the Claggett, Bearpaw, and Pierre formations (all marine-deposited clay shales of Late Cretaceous age), all slopes exhibited indications of failure;

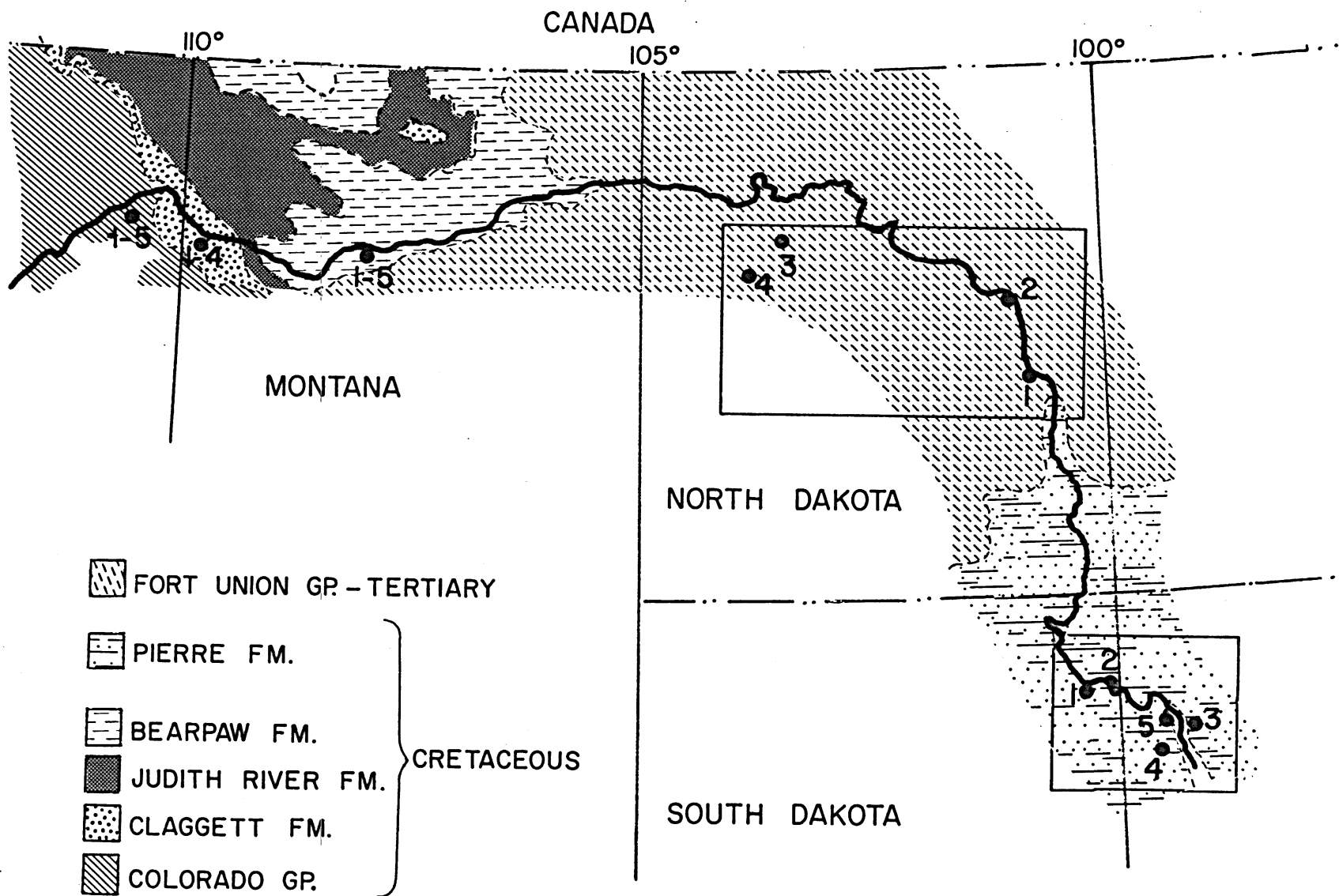


Fig. 1. Generalized geologic map showing study sites for detailed investigation.

Table 1. 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) |

and while overall slope heights amounted to several hundred feet, inclinations were typically very low (5 to 10 deg), for example, Fig. 2. 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 Bearpaw stood slightly steeper than elsewhere. Slope failures in the Colorado group, also a marine deposit of Late Cretaceous age, were relatively uncommon. Where they occurred, sliding appeared to involve a single failure, for example Fig. 3. 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 nonmarine 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 geologic units studied. Bentonitic layers with high plasticity characteristics, high swelling potential, and low residual shear strength were found in all of the units.

Laboratory Tests

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

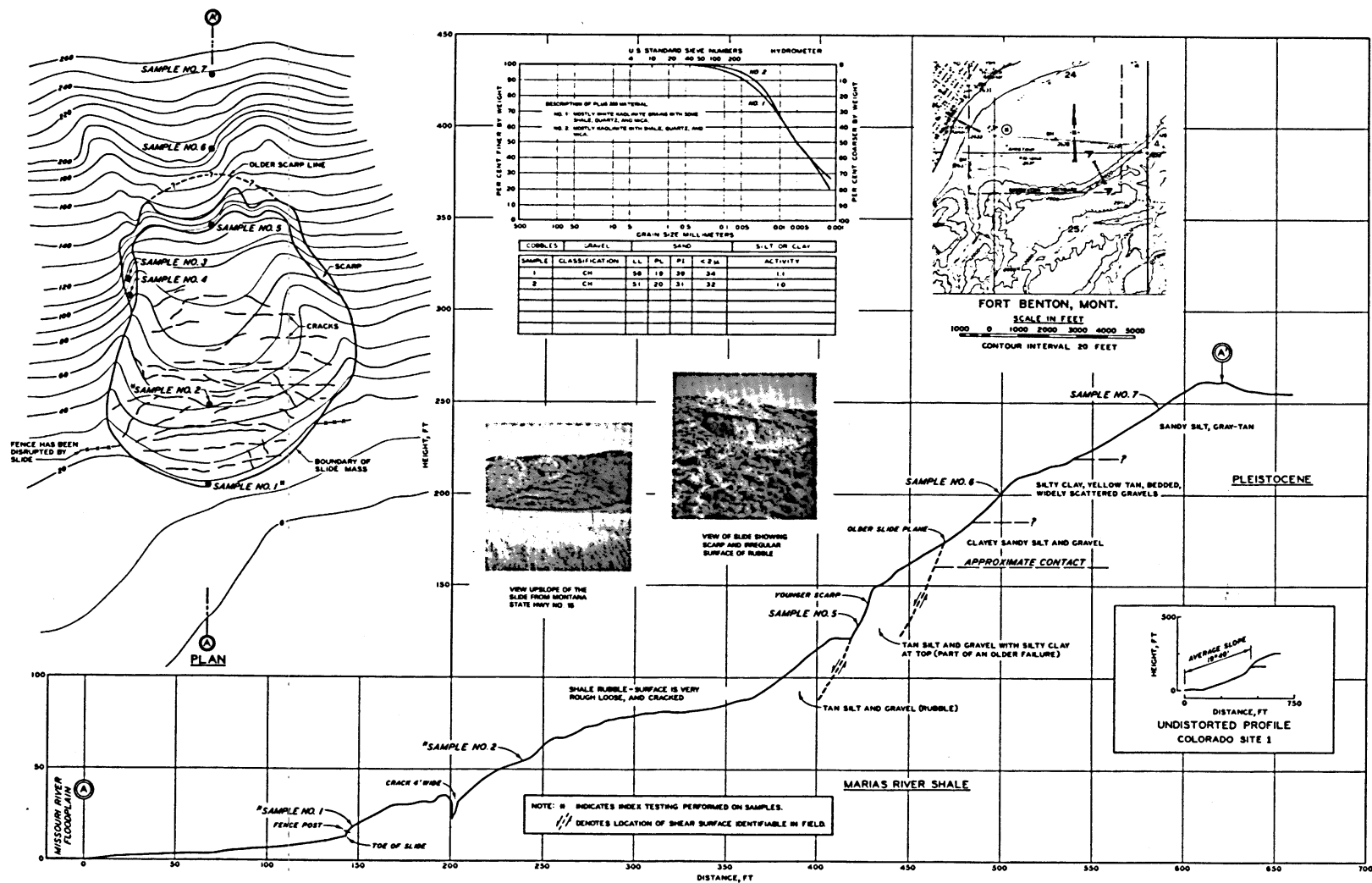


Fig. 3. Colorado site 1, geologic section A-A' and plan.

showed considerable overlap. The tests indicated a highly anisotropic character to the swell-shrinkage behavior of the clay shales. In particular, the swell pressure measured parallel to bedding was less than the swell pressure perpendicular to bedding; additionally, shrinkage of specimens was less than that measured parallel to bedding than perpendicular. The effects of surface weathering and erosion were illustrated by the decrease in shrinkage with depth and an increase in the swell pressure. The effect of depth was also noted from consolidation tests in that shallow specimens exhibited steeper consolidation curves than the deeper. Since there was almost as large a range in measured properties within a formation as between formations, it appeared that only gross differences in slope behavior could, in this study, be related to any specific test or series of tests. This conclusion was especially true since the slopes under consideration were all natural, geologically old slopes, and had therefore been subjected to the various factors discussed previously for many thousands of years. Since the overall slopes were generally failed, the residual strength of the materials was important, Fig. 4.

Results of Study

The empirical approach of plotting slope inclinations versus height for the various slopes studied within a given formation was a logical method of summarizing the data obtained. Furthermore, case studies of the major dam projects in the upper Missouri Basin and in other clay shale areas generally indicated that 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,

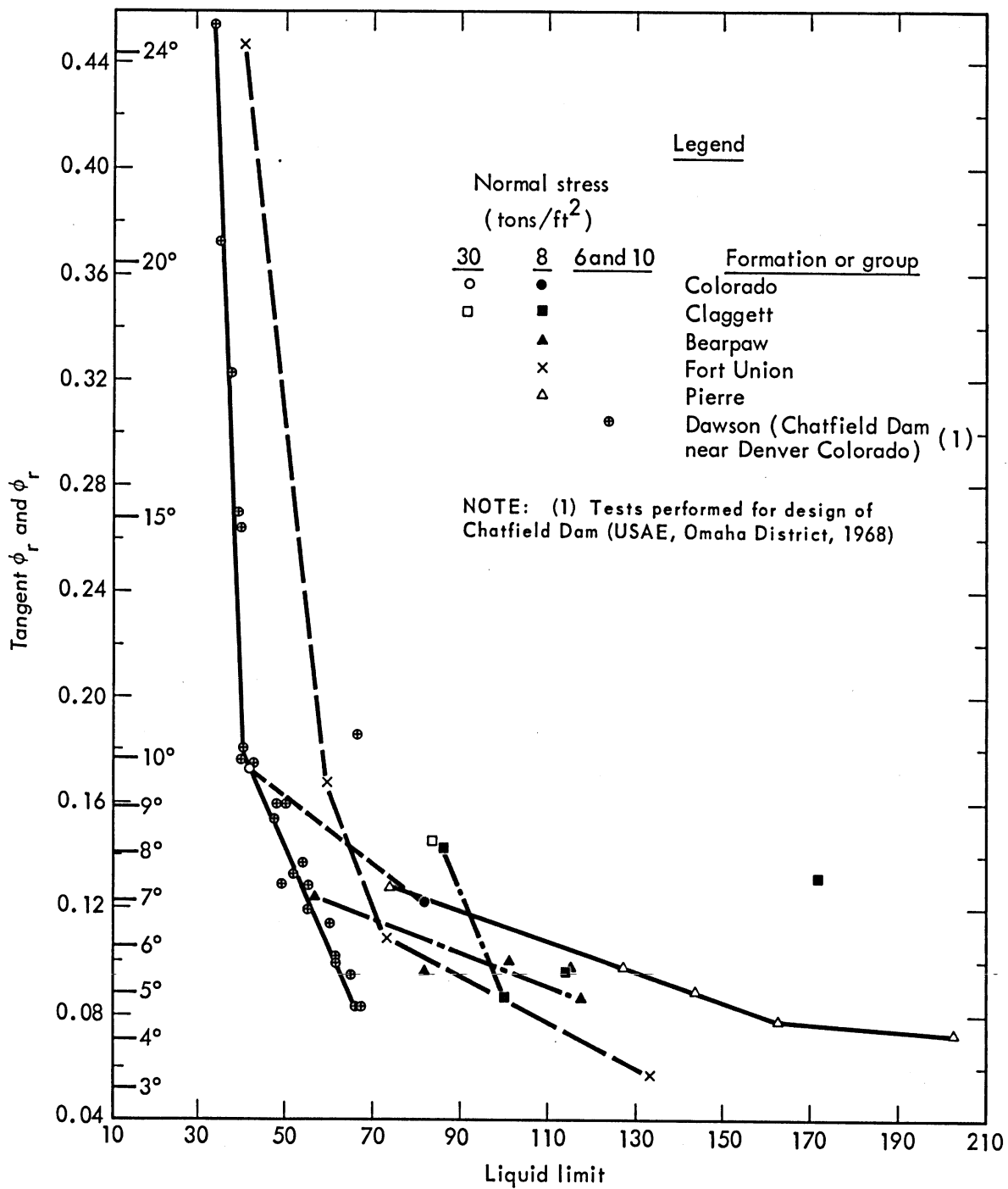


Fig. 4. Summary of residual direct shear test data.

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 slope charts developed for the five units studied are shown in Figs. 5 through 9. The slopes in the Claggett, Bearpaw, and Pierre formations are similar. For slopes composed entirely of shale, the range in slope angles compares to the 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 cause slightly steeper slopes than those without such a base. Slopes in the Bearpaw which were capped with Fox Hills sandstone were also found to have steeper inclinations. Slopes in the Colorado group stand at relatively steep angles when compared to the residual friction angle. The average strength along failure surfaces must be increased from that obtained from the residual friction angle by the influence of numerous interbeds of cemented silt. Slopes in the Fort Union group stand at variable heights and inclination, depending on the formation involved. Slopes in the Sentinel Butte formation are steeper, as a group, than slopes in other formations. This behavior was thought to reflect a coarser-grained lithology in the Sentinel Butte.

Since the basic purpose of this study was to provide an assessment of the long-term stability of high crater slopes, a review of experiences was made. The only experimental crater slopes in clay shale (the Pre-Gondola project of the Nuclear Cratering Group at Fort Peck, Montana) have been subject to observation for only four years or less. The slopes varying in height from 40 to 80 ft and in inclination from 22 to 29 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

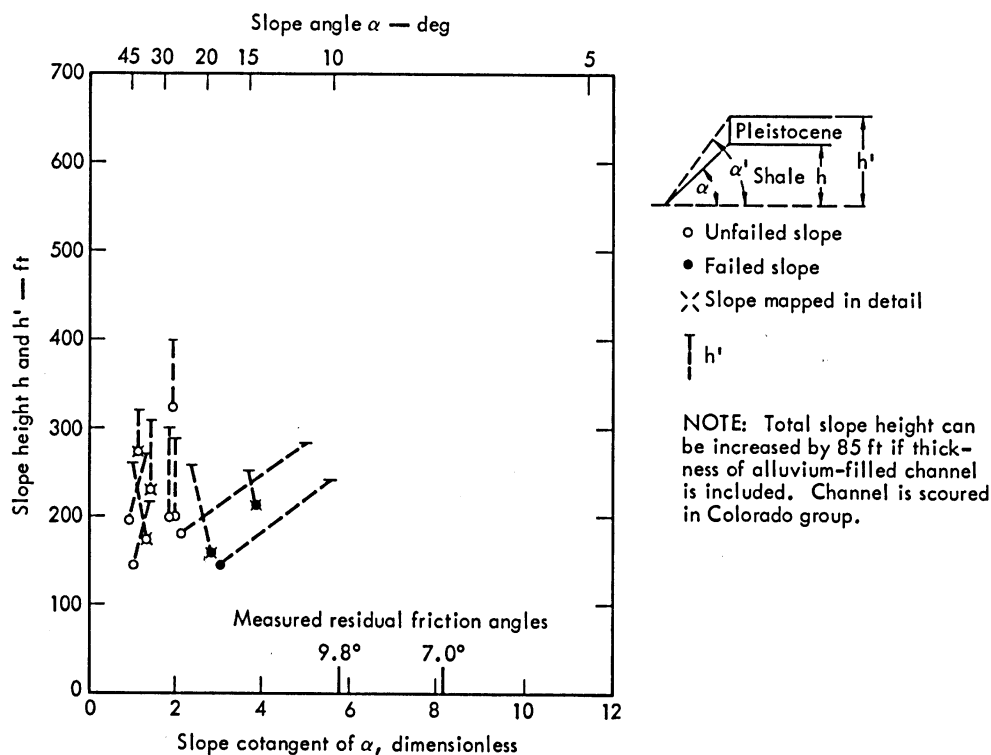


Fig. 5. Height and inclination of slopes in Colorado group.

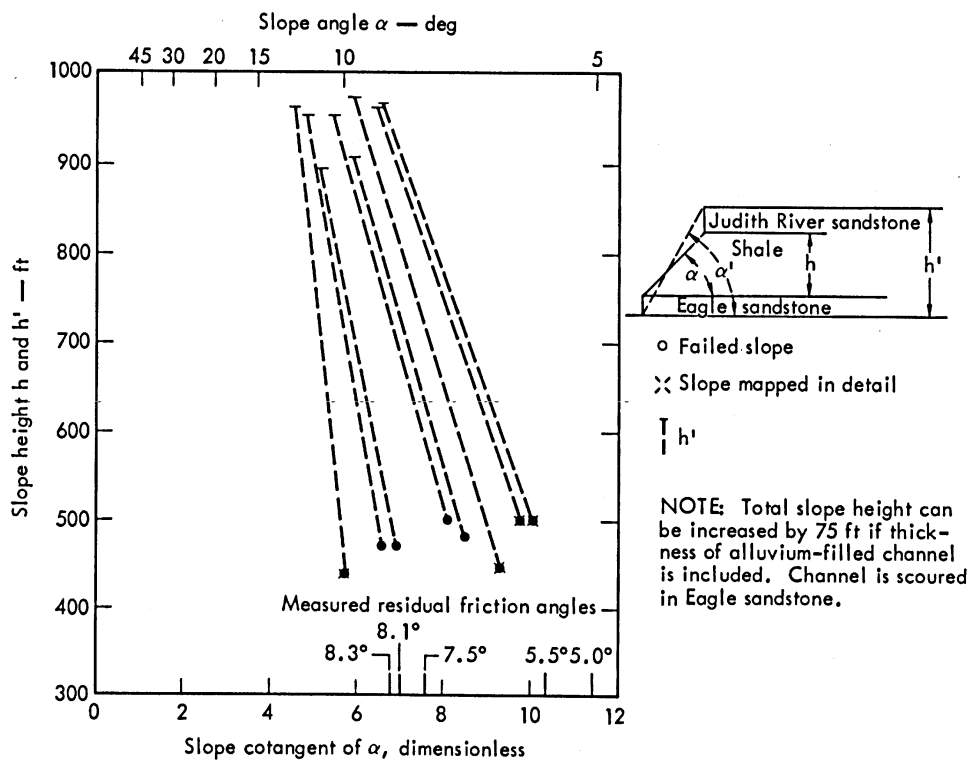


Fig. 6. Height and inclination of slopes in Claggett formation.

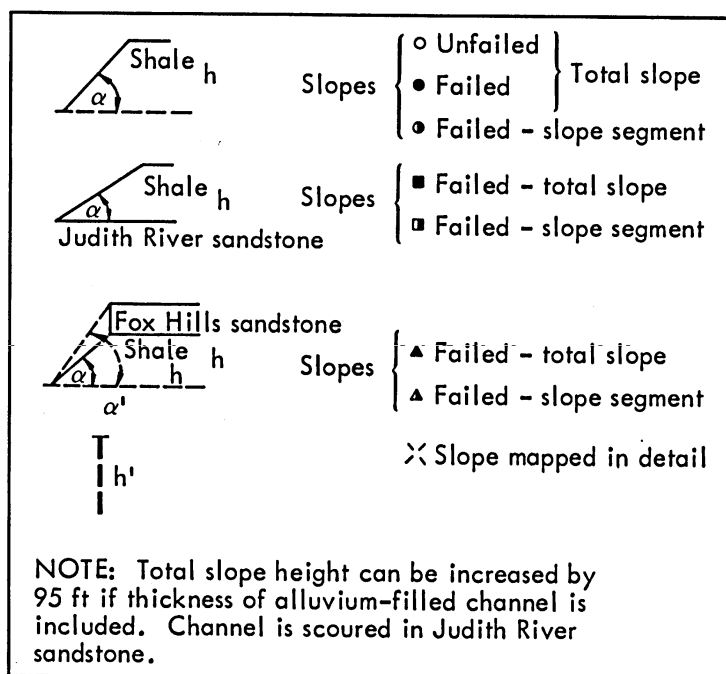
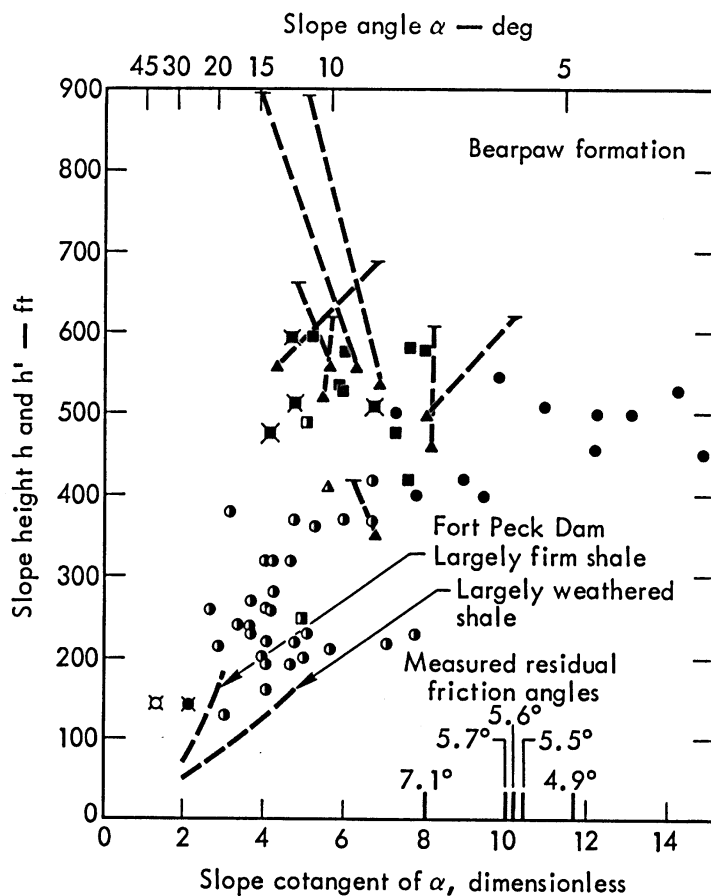


Fig. 7. Height and inclination of slopes in Bearpaw formation.

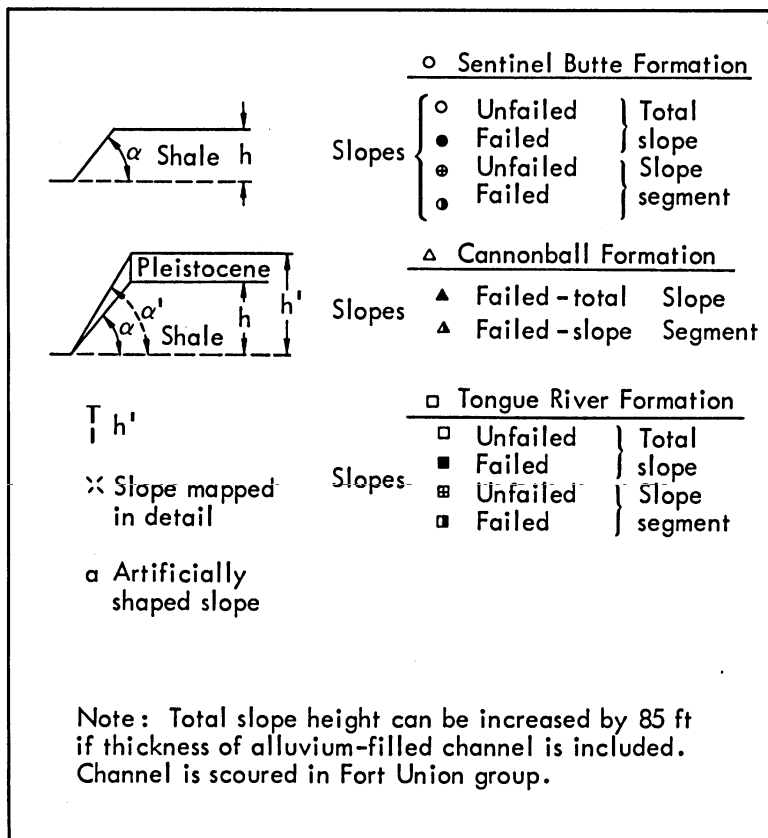
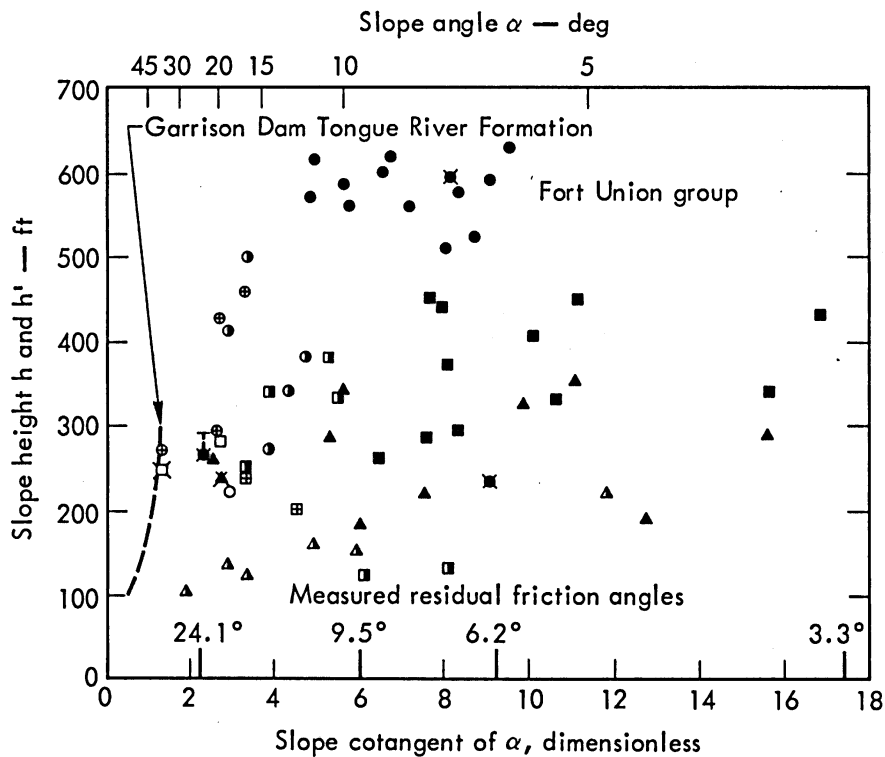


Fig. 8. Height and inclination of slopes in Fort Union group.

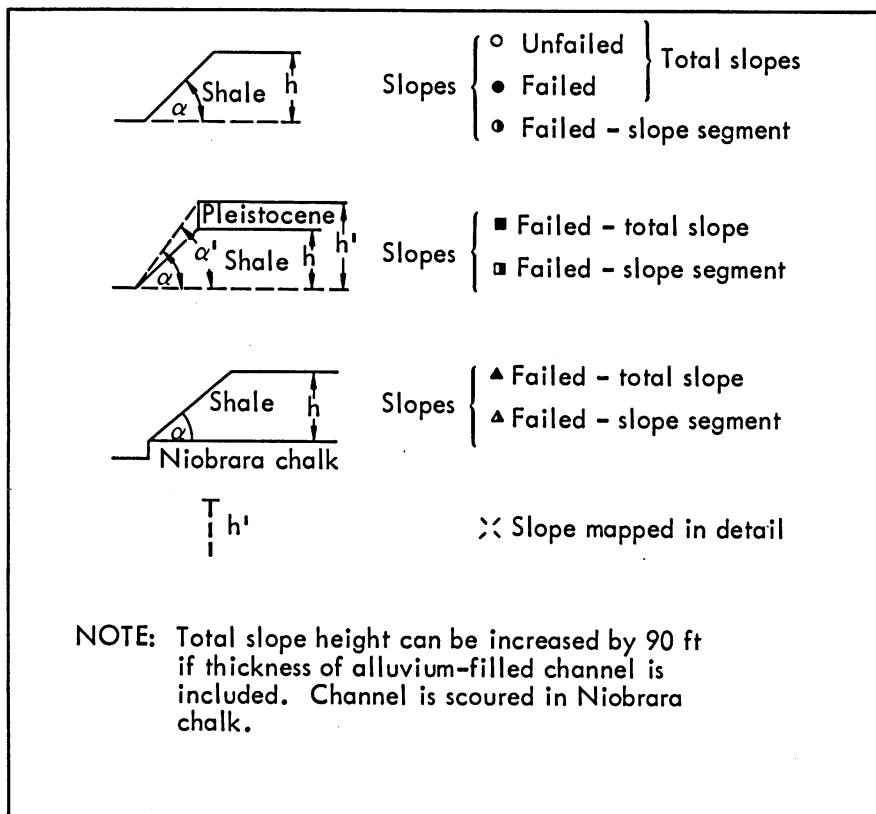
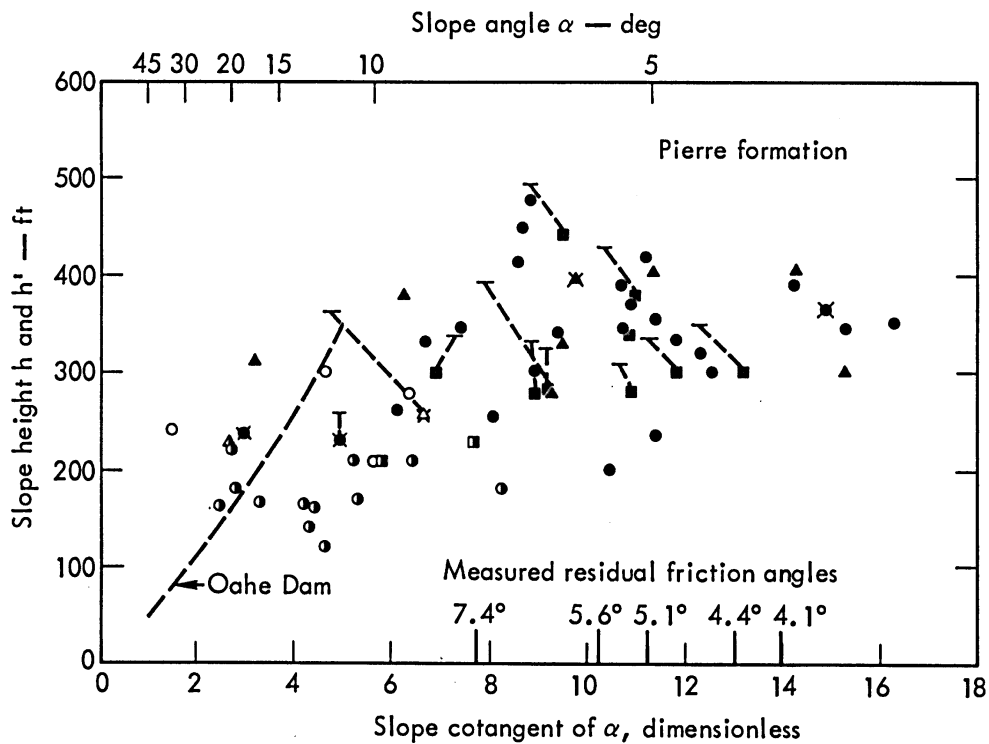


Fig. 9. Height and inclination of slopes in Pierre formation.

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 ultimately adjust.

RESULTS OF STUDIES OF CLAY SHALE SLOPES ALONG THE PANAMA CANAL

The Panama Canal studies differed from the previously described work in that the slopes were constructed as part of an engineering project and data and experience records existed from which the behavior of the slopes could be reconstructed. The factors previously discussed were of course operative to varying degrees, but the failures were induced as a direct result of the construction activities. The study is a continuing project which will cover the entire 12-mile Gaillard Cut through the Isthmian Divide. The present paper covers the results of the first year's effort and concentrates on the East and West Culebra slides, and the Model Slope, Fig. 10.

Geology

The Isthmian Divide area is underlain by relatively soft Tertiary pyroclastic strata and localized masses of resistant basalt and breccia. These contrasting rocks are distinctly expressed as broad rolling topography and conical hills, respectively. A system of north-south faults and subordinate east-west fractures divides the area and is responsible for localized basaltic intrusions. North-south faults delineate a graben structure crossing the canal obliquely at the East Culebra and West Culebra slides. Bedding dips adversely toward the canal at least locally, contributing to instability in both banks. Favorable dips in the Model Slope, i.e., nearly horizontal or into the slope, may account in part for the slope's stability. The Cucaracha formation,

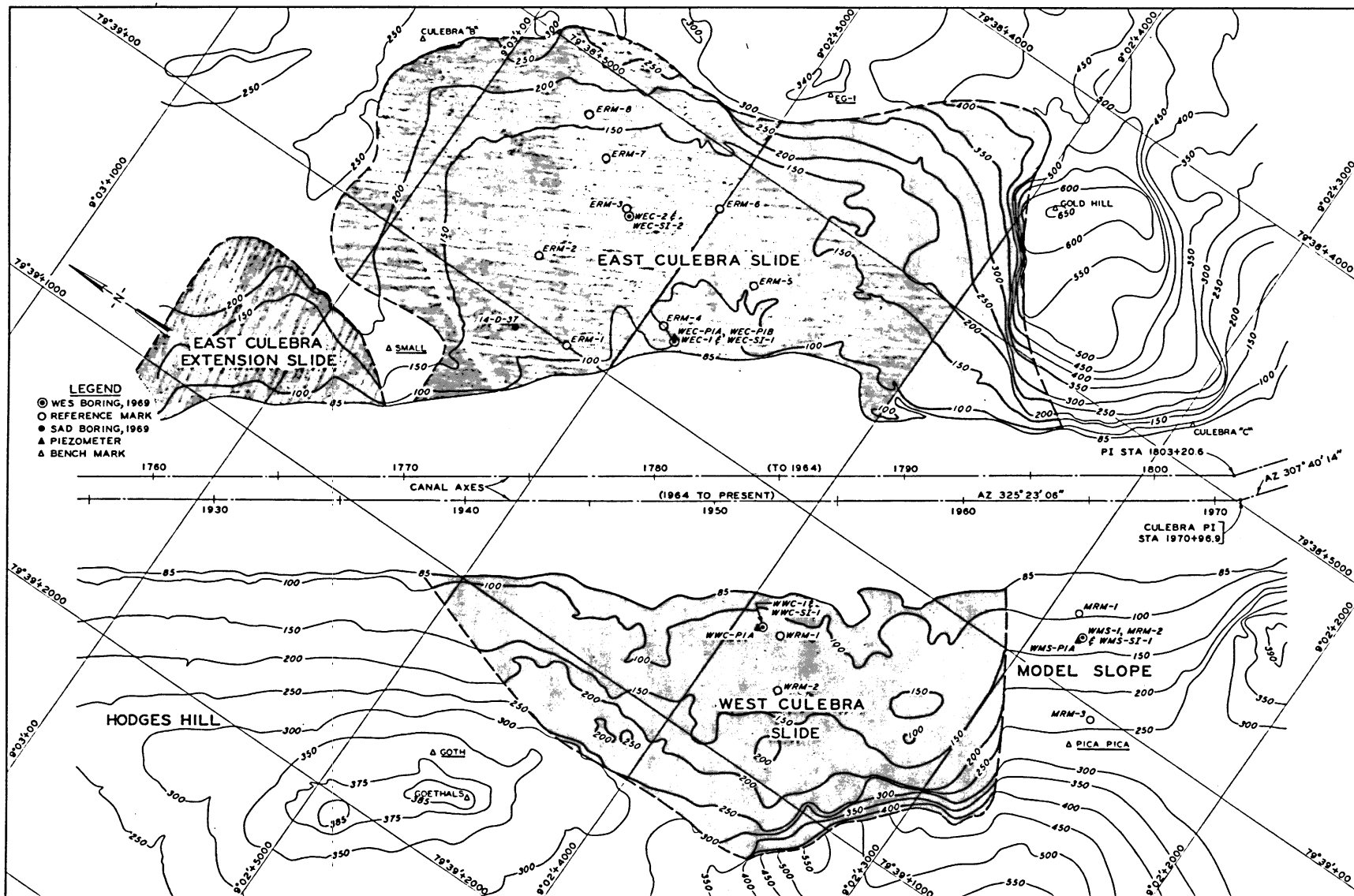


Fig. 10. General plan of study area.

composed basically of tuffaceous shale, and the underlying Culebra formation of tuffaceous shale, siltstone, and sandstone are involved in sliding. Characteristics of the Cucaracha formation that have made it particularly susceptible to sliding are: (1) an abundance of shale strata; (2) a high content of montmorillonite in the shale; and (3) well developed, small-scale fracturing in the form of slickensides in certain layers. The lower third of the Cucaracha formation seems to be a transition zone, identifiable on the basis of lithology and the presence of scattered limy concretions. The zone may be locally cemented and possibly has a slightly greater strength, but this is not corroborated by laboratory testing.

History of Sliding

When the United States assumed control in 1904, the excavation had been cut through the divide area to a depth of about 140 ft with the bottom at about el +175. Both banks of the excavation were largely overgrown with vegetation, and only vague evidence remained of surficial sliding previously experienced by the French. Subsequently, the first failures occurred in 1906-07 at sta 1777 on the west bank and sta 1789 on the east bank. Both sides were partly in residual weathered clay, but they soon evolved to deep slides.

The slides grew significantly from 1908 to 1914 by deepening in pace with excavation and progressively incorporating new blocks of ground farther back in the bank. By 1911 slides had occurred in both banks at intervals between sta 1760 and 1794. As the final excavation bottom at el +40 was reached in 1913, the East Culebra slide had attained a depth corresponding approximately with the top of the transition zone in the Cucaracha formation. Sliding was apparently taking place along one or a few favored stratigraphical horizons. Cracking of the bank usually preceded failure. Then after a few months the blocks separated by cracks gradually settled, sometimes also tilting toward

the excavation and was followed by massive failure of the front block and its sliding into the excavation in a few hours or days (Goethals, 1916).

Sliding activity accelerated on the east side to a major break in 1914, almost a year after the canal was flooded for dredging. By January 1915, the West Culebra slide was divided by long cracks along geological fractures into several potentially unstable blocks. The blocks evolved to graben and horst structures as the mass crept slowly canalward. In August and September 1915, both slides converged in massive failures accompanied by upheaval of the canal bottom and blockage of the canal. An example of the reconstructed history of sliding for one section in the East Culebra slide is shown in Fig. 11.

Slow movement with occasional increased activity has continued to the present. This stage has apparently reflected a drastic deterioration of material strength since the driving force due to large volume and height are not comparable to those operating at the prime of slide activity.

Upheaval of the bottom of the excavation has been characteristic of the sliding activity since 1907. The most impressive case occurred in the culminating movement of 1915. The upheaval mass appears to correspond to a passive wedge at the front of a basically translational slide. Deep sliding surfaces have been suspected from construction days at about el -50 to -100 ft, but the more conventional picture has the sliding surfaces at el +100 to +25.

Results from Field Instrumentation - 1969

Slope Indicators. Data collected from slope indicators installed in 1969 located the present depths of sliding in the East Culebra and West Culebra slides near the bottom of the canal. These data complement interpretation of borings and give good information as to the extent of the masses now involved in sliding. Data obtained at the Model Slope installation indicated small movements at several depths which are probably closely related to the geologic structure and surface expression of cracking in the area.

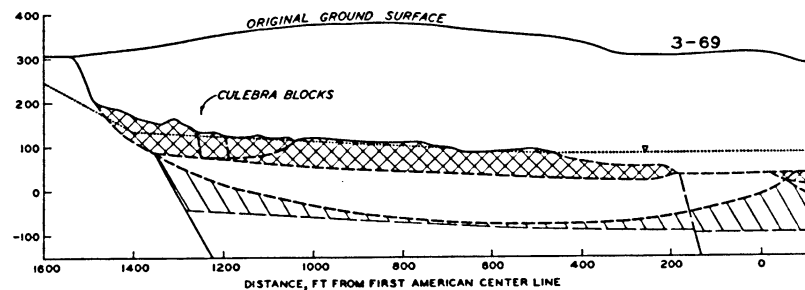
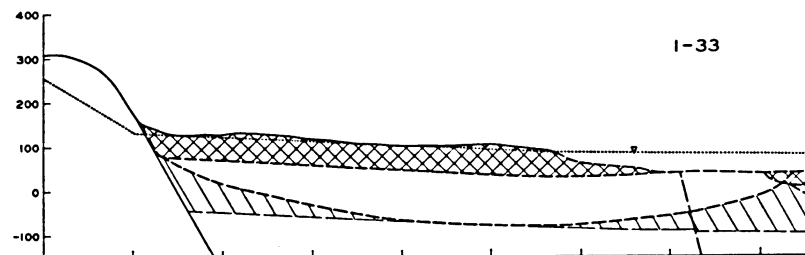
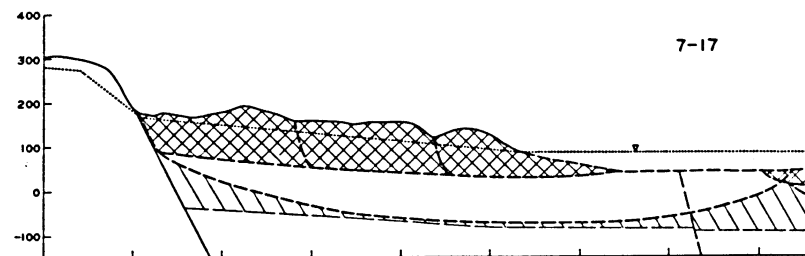
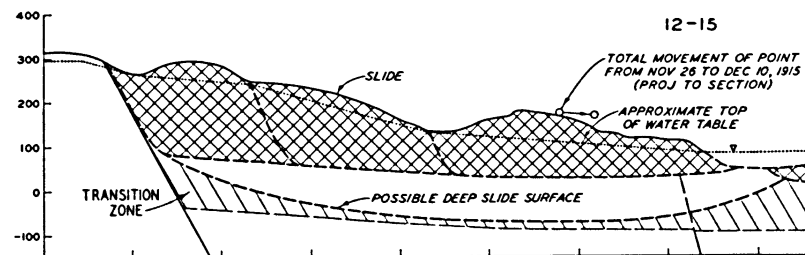
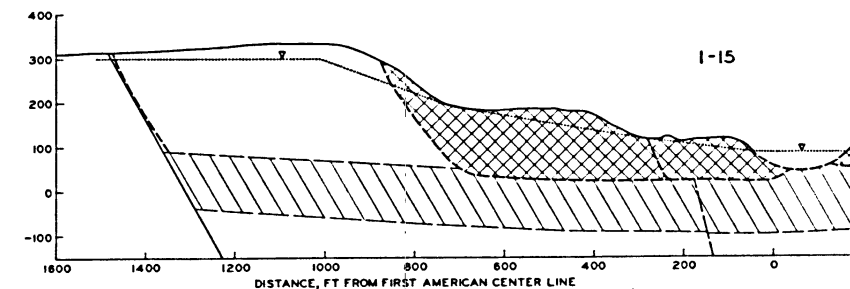
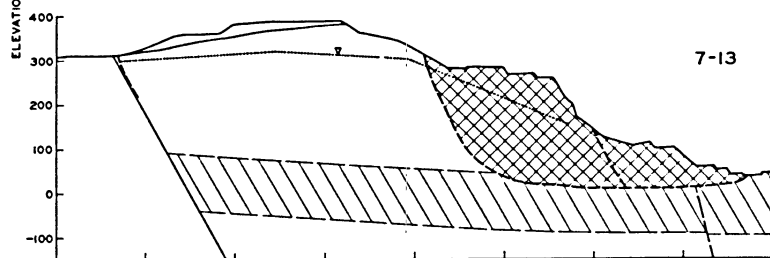
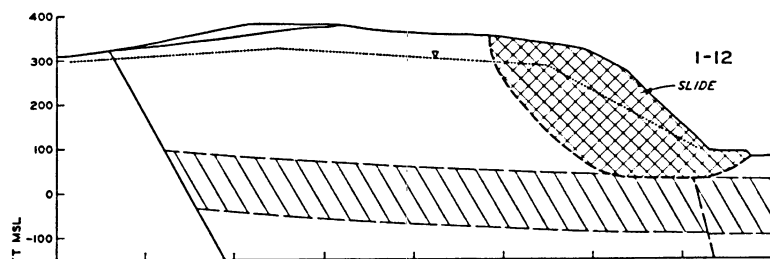
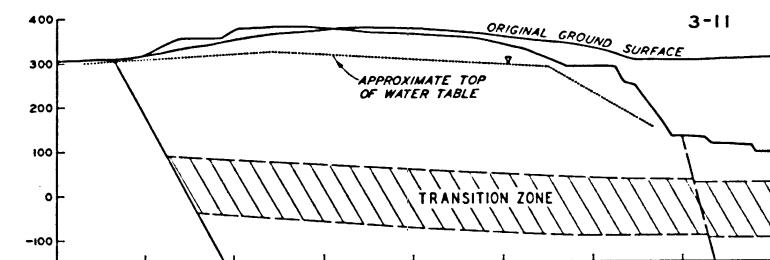


Fig. 11. Development of East Culebra slide at sta 1782+50, March 1911 to March 1969.

Surface Survey Monuments. Surface reference markers indicated continuing movements at all installations. Movements at the surface of the East Culebra and West Culebra slides were toward the canal, whereas those at the surface of the Model Slope were strongly influenced by the geologic structure as reflected by the surface crack pattern. Movement measurements are being continued to obtain a more accurate account of the rates and direction of movements. The data collected thus far serve to indicate the pronounced influence of geologic structure on slope behavior in the study area.

Piezometers. Piezometers located in the study area were placed in slickensided zones in clay shale materials. Deep piezometers in the East Culebra and West Culebra slides were located below the depth of active sliding and indicated a reduced pore water pressure (below canal water level) which probably reflects rebound or swelling occurring as a result of canal excavation. A low pore water pressure was similarly noted in the Model Slope installation. The shallow piezometer in the East Culebra slide indicated a piezometer level equal to that in the canal. Additional piezometers are being installed beneath the East Culebra slide and the Model Slope to give a more complete picture of the piezometric level with depth. The concept of pore water pressure reductions resulting from excavation may be an important factor in the apparent reduction of shear strength with time. Since shearing resistances are a function of effective stresses, a gradual increase of pore water pressure should be accompanied by a decrease in shear resistance that may be sufficient to cause failure some time after excavation. As a result of the occurrence of Cucaracha clay shales of extremely low permeability overlying Culebra sandstone, it is possible that high pore water pressures could exist in the sandstone regardless of pore water pressures in the overlying clay shale. Thus, the failures may have been influenced in part by low effective stresses and resulting low shear strengths at the clay shale-sandstone interface.

Results from Laboratory Tests

Classification Data. The laboratory classification of shale samples from the Cucaracha and Culebra formations indicated that the materials generally plotted close to the A line on the plasticity chart. The results indicated little differences in average values of the liquid and plastic limits but that exceptional values could occur at any depth. Slaking reactions were similar in the two formations with materials slaking very rapidly when air-dried and immersed in distilled water. The most important difference noted between shales in the two formations was the slight increase of materials lost in the insoluble residue test in the Culebra formation. The increase reflected the increase in marine fossils in the Culebra. A small increase was similarly noted for samples taken from the transition zone of the lower Cucaracha. From the laboratory results it was concluded that very little difference existed in the clay shale portions of the Cucaracha and Culebra formations.

Water Contents. Water contents generally ranged from 7 to 15 percent below the plastic limit; higher water contents were noted in samples taken from the slide debris. Significantly, water contents about double the average water content were determined for slickensided zones at the probable depth of sliding in the East Culebra and West Culebra slides.

Consolidation Characteristics. Preconsolidation stresses were determined to be as high as 200 tons/sq ft; preconsolidation loads from geologic evidence were estimated to be as high as 130 tons/sq ft. Swell pressures increased with depth and generally ranged from 0.8 to 1.3 times the computed effective overburden pressure assuming hydrostatic pore pressures. The expansion index averaged about 0.045, the coefficient of consolidation averaged about 0.35×10^{-4} cm²/sec, and the coefficient of permeability averaged about 1×10^{-10} cm/sec from the unloading portion of consolidation curves between loads existing before

excavation was commenced in the study area and those now existing. The extremely low value of the coefficient of permeability is corroborated by results computed from piezometer observations.

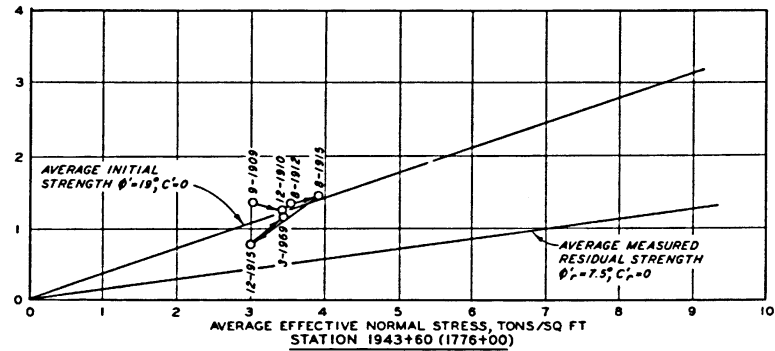
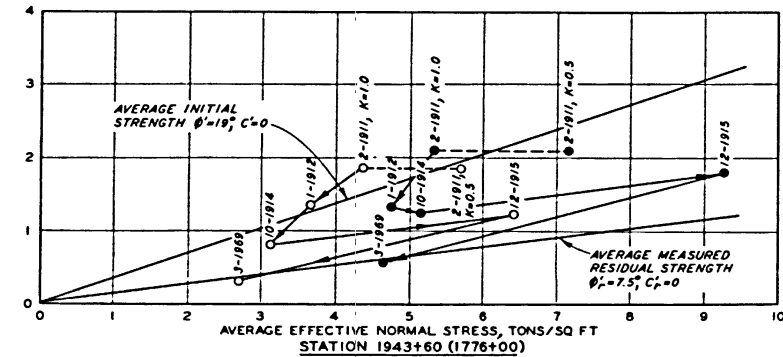
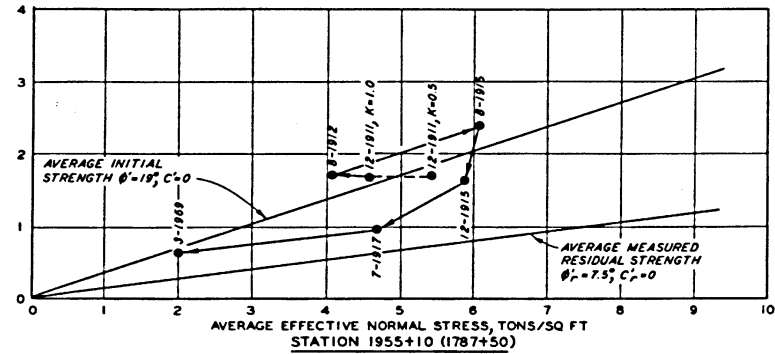
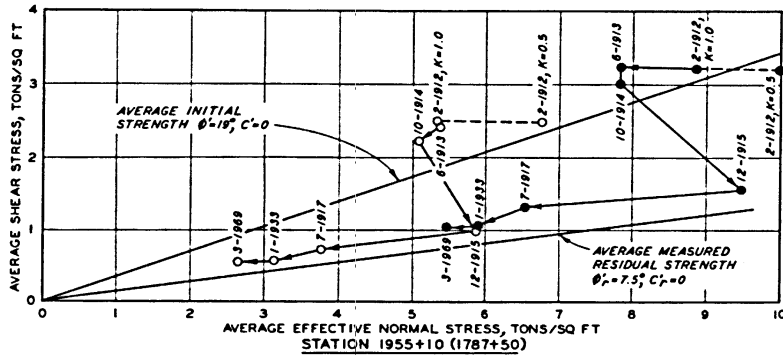
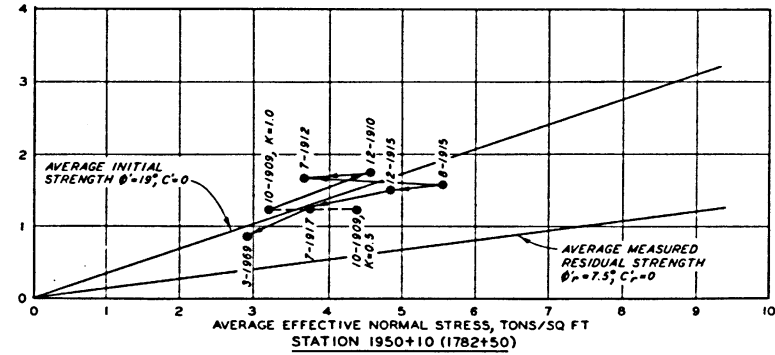
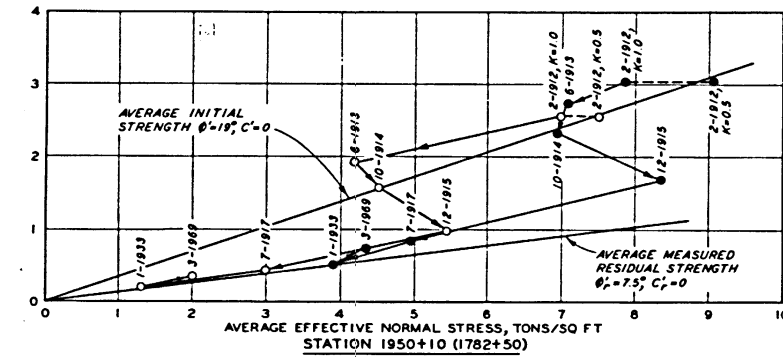
Strength Tests. The majority of previous tests conducted on Cucaracha shale samples indicated a peak drained friction angle of from 17 to 22 deg and averaging about 19 deg. The effective cohesion intercept was shown by several tests to be extremely variable, depending on the occurrence of slickensides, sandy seams, etc. Laboratory drained direct shear tests and slow triaxial tests indicated cohesion values of about 6 tons/sq ft; the single cohesion value from this study was 5 tons/sq ft. Residual friction angles determined in this study varied between 4.5 and 10.5 deg with an average value of approximately 7.5 deg.

Stability Analyses

East Culebra and West Culebra Slides. Cross sections of the slopes were reconstructed at selected times (see as an example Fig. 11) for stability analyses. The average strength at the time of first-time failure and the variation in average strength with time are of particular interest. Possible deep critical failure surfaces and failure surfaces corresponding to present slope indicator data were used to determine the peak effective strength, but the effect of assumed depth of sliding was small. The effect of reduced pore water pressure was also considered. Stability analyses of initial failure conditions at selected sections along the canal indicated an average effective strength envelope of $\phi' = 19$ deg and $c' \approx 0$ (Fig. 12). The friction angle thus determined compares closely with friction angles determined from test results. Analyses of failure conditions for various dates of sliding indicated that the average shear strength between the time of initial failure and the massive movements (Oct 1914 for the East Culebra slide and Aug 1915 for the West Culebra

EAST CULEBRA SLIDES

WEST CULEBRA SLIDES



LEGEND

- LOWER FAILURE SURFACE
- UPPER FAILURE SURFACE

NOTE: ALL POINTS PLOTTED FOR $K = 1.0$ EXCEPT POINTS FOR INITIAL FAILURE WHERE RANGE IN AVERAGE EFFECTIVE NORMAL STRESSES ARE INDICATED BETWEEN $K = 1.0$ AND $K = 0.5$. K IS A FACTOR USED WITH THE BISHOP PORE PRESSURE PARAMETER $r_u = Ku/\gamma h$.

Fig. 12. Average stresses from analyses of East and West Culebra slides.

slide) remained high and near the strength envelope determined for initial failure despite the large movements that occurred. Analyses of failure conditions subsequent to the time of massive movements to the present indicate a large reduction in required shear strength (Fig. 12). In most cases the required shear strength was slightly higher than the average residual strength from laboratory tests. This is consistent with continued slope movements occurring in the East Culebra and West Culebra slides.

Model Slope. Analyses of the Model Slope were made by computing safety factors using the peak effective strength parameters, i.e., $\phi' = 19$ deg and c' varying from 0 to 0.4 ton/sq ft. Sections were chosen perpendicular to the canal and perpendicular to the major crack presently seen on the surface of the Model Slope. Analyses using strength parameters of $\phi' = 19$ deg, $c' = 0.4$ ton/sq ft generally produced safety factors slightly greater than 1.0. The most critical stage in the existence of the Model Slope was represented by the analyses of conditions in July 1917. Conditions on this date represent the canal geometry when excavation had been completed in the canal and the Model Slope had not been reduced in inclination. Analyses for 1969 conditions indicated factors of safety greater than 1.0. However, the assumption of peak strength for design purposes may not be conservative in view of results obtained during the most critical period and because of subsurface movements now occurring. Even small movements during the critical period could have served to reduce the strength so as to lower the calculated factor of safety for today's configuration.

Application of Study Results

Initial Stability of Excavated Slopes. The results of stability analyses provide a means of extrapolating field shear strengths for design of slopes higher than those covered by available experience. However, extrapolation to higher slopes assumes that groundwater conditions, anisotropy, and geologic

defects and structure are similar to those of the East Culebra and West Culebra slides. However, when extrapolating to conditions where higher normal stresses will be acting, consideration should be given to possible reduction in friction angle with higher pressures as indicated by several studies and this aspect requires further investigation. While results to date must be corroborated by similar studies on other canal slope failures before they can be seriously considered for slope design, present indications are that field strengths for high (600 ft) slopes may be 25 percent higher than the previous strengths resulting from the previous studies (PCC, 1947). However, no material difference exists for slopes 300 to 400 ft high or lower. These conclusions are preliminary and will be studied further at these and other slides during subsequent phases of the study.

Decrease of Shear Strength with Time and Movement. The results obtained indicate a large decrease in shear resistance with time following initial sliding. However, the shear strength did not appear to decrease rapidly immediately following initial sliding, and the slides enlarged, with large movements, before a substantial decrease in shear strength developed. If this is corroborated by further study at the East Culebra and West Culebra and other slides, the feasibility of an observational approach to modify slope design during construction would be indicated. However, while present indications are encouraging in this respect, much additional study of the aspect is required to corroborate the findings and to establish their applicability.

SUMMARY

The reported studies have given information on the stability of clay shale slopes. The approach has been to study the geology and history of sliding, to determine the physical properties from laboratory tests, and to make stability

analyses of the slopes. Geology of slopes in five different formations along the upper Missouri River were studied. The most expedient manner to view the stability characteristics of these slopes was through empirical slope charts. Design experience in the study area had used similar approaches with success when local site geologic and hydrological conditions were considered. In the Panama study, detailed records were available to reconstruct the sliding history and allow limit type analyses to be performed. Both approaches can be utilized to determine design slopes by giving proper recognition to the limitations of each.

APPENDIX I.—REFERENCES

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13. ABSTRACT

The paper gives information on the stability of clay shale slopes from studies of the geology and history of sliding, determination of physical properties from laboratory tests, and stability analyses of the slopes. Slopes in five different formations along the upper Missouri River were studied. The most expedient manner to view the stability characteristics of these slopes was through empirical slope charts. Design experience in the study area had used similar approaches with success when local site geologic and hydrological conditions were considered. In the Panama study, detailed records were available to reconstruct the sliding history and allow limit type analyses to be performed. Both approaches can be utilized to determine design slopes by giving proper recognition to the limitations of each.

