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IGN AND CONSTRUCTION OF COMPACTED SHALE EMBANKMENTS

Vol. 2. Evaluation and Remedial Treatment of Shale Embankments

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16. Abstract

The purpose of the three-phase study is to develop design and construction methodologies that will enable shales which have caused settlements and slope failure in highway embankments to be identified and used successfully in future construction. The purpose of Phase II was to develop (a) techniques for evaluating the stability of existing embankments and (b) remedial treatments for distressed embankments. Information obtained from State and Federal agencies and the literature was reviewed. Types and probable causes of distress, evaluation techniques, and remedial treatment measures are discussed. Evaluation techniques recommended are: instrumentation with piezometers, inclinometers, and settlement markers; undisturbed sampling and laboratory testing; in situ testing with borehole devices (Menard pressuremeter and Iowa shear test device); and back analyses of failed slopes. The primary considerations in remedial treatment should be subsurface drainage (mainly horizontal and vertical drains) and surface drainage (mainly paved ditches). Slope flattening, berms, retaining walls, cement grouting, and/or complete reconstruction should be considered in addition to drainage measures when extensive movements and/or shale deterioration have caused large reductions in shear strength. This is the second of two volumes. Volume 1, covering Phase I, is published as FHWA-RD-75-61 subtitle: Survey of Problem Areas and Current Practices.

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PREFACE

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

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

Sections on reinforced earth and lime, cement, and chemical stabilization were prepared by Drs. M. M. Al-Hussaini and F. C. Townsend, respectively.

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

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

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

Units of measurement used in this report can be converted as follows:

Multiply	By	To Obtain
U. S. Customary to Metric (SI)		
inches	2.54	centimetres
feet	0.3048	metres
square feet	0.09290	square metres
gallons (U. S. liquid)	3.785412	litres
cubic feet	0.0283168	cubic centimetres
tons (2000 pounds, mass)	907.1847	kilograms
pounds (force) per square inch	6894.757	pascals
pounds (force) per square foot	47.88026	pascals
kips	4448.222	newtons
degrees (angle)	0.0174533	radians
Metric (SI) to U. S. Customary		
millimetres	0.0394	inches
centimetres	0.3937	inches
metres	3.2808	feet
cubic centimetres	0.06102	cubic inches
kilograms per square centimetre	14.223	pounds per square inch
centimetres per second	1.968	feet per second

I. INTRODUCTION

Background

Construction of the modern interstate highway system has required large, high embankments using economically available materials from adjacent cuts or nearby borrow sources. Because of the widespread distribution of shale,* it has been necessary to use this material for embankments in many locales. This practice has led to numerous problems caused by excessive settlements and slope failures of large embankments. While these problems have occurred in several states (Chapman and Wood, 1975), the more severe problems have occurred in the east central States where the climate is humid. Remedial treatment of shale embankments is often expensive. For example, repair of three embankment slope failures along Interstate Highway 74 (I-74) in southeastern Indiana cost $2 million (DiMillio and Haugen, 1974).

The problems are apparently caused by detrimental time dependant changes in the properties of certain shales after construction of the embankment. Some shales have the appearance and properties of sound rock upon excavation, but they may deteriorate after placement into soft clay or silt having a low shear strength. When shale is placed as rock fill (i.e., little or no compaction), its deterioration can cause large settlements, blockage of seepage, and eventual slope failure. Other shales, often interbedded with limestone, tend to degrade when excavated; but when attempts are made to compact the shales as soil fill, the larger more durable shale and slabby limestone pieces can prevent adequate compaction. The causes of the problems are complicated by variations in stratigraphic and lithologic characteristics of weak sedimentary rocks, climate and groundwater conditions, and construction procedures.

A number of States have delineated problem shale formations and

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* Unless otherwise noted, the term "shale" will refer to all weak sedimentary rocks such as claystones, siltstones, mudstones, etc.

have developed special design procedures and construction specifications applicable to compacted shale embankments. Several States have also used field test strips to aid in selecting compaction requirements. In some States all shales are required to be compacted as soil fill in the absence of proven criteria for classifying shale durability. A comprehensive study, recently conducted at Purdue University (Deo, 1972) for the Indiana State Highway Commission, has led to the development of preliminary classification criteria based on laboratory tests. These criteria are currently being used in Indiana.

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

Comprehensive guidance is required concerning field investigations, design, and construction of compacted shale embankments. The required information includes (a) determination of strength and compressibility properties for both end-of-construction and long-term conditions; (b) pretreatment requirements, compaction specifications, and compaction control techniques; and (c) design criteria and investigations.

Of more immediate concern is the need for suitable methods of evaluation and remedial treatment of embankments which are exhibiting signs of distress. Some embankments are settling excessively. In the past, this settling has led to slope failure in some embankments, while in others the settlement has ceased and no further distress has occurred. Determining the likelihood of failure and the appropriate types of remedial measures are paramount problems. The presence of large, hard, rock pieces hinders undisturbed sampling. Improved methods are needed to evaluate in situ densities, permeability, shear strengths, and compressibility. Guidance is needed on appropriate instrumentation for monitoring deformations, groundwater conditions, and pore water pressures.

The repair of embankment failures has usually involved the removal of all or a portion of the slide materials, installation of drainage measures, and reconstruction using flatter slopes and/or berms to ensure stability. Criteria and guidance on the applicability of these and other less expensive methods of stabilization are needed.

The needs outlined above led to the development of a planned 4-yr research study addressing compacted shale embankment problems. The study is sponsored by the Federal Highway Administration (FHWA).

Objectives and Scope of Study

The basic objectives of the research effort are:

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

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

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

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

The research study was divided into three phases. The scope of the study is summarized in Table 1 which contains a description of task assignments under each phase. Objectives a, b, and c were identified as critical needs by FHWA and were accomplished concurrently during the first years's effort; objective a was developed under Phase I and objectives b and c under Phase II. Results of Phase I are reported by Shamburger et al. (1975); results of Phase II are contained in this report. Phases I and II accomplishments provide the necessary foundation for development of objective d under Phase III of the study.

The study concerns the behavior of compacted shale embankments as controlled by properties of the constituent shales. The study excludes the following:

a. Settlement and stability problems originating in the embankment foundation.

b. Stability problems originating in cut slopes.

c. Deterioration of compacted shale arising from frost action.

Table 1. Scope of research study and schedule.

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

Task A

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

Task B

1. Identify the geologic, stratigraphic units and the specific geographic localities of shales that have caused problems in compacted embankments.
2. Accomplish preliminary identification and validation of the intrinsic and extrinsic factors and combinations of these factors causing the problems.

Task C

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

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

Task A

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

Task B

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

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

Task A

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

Task B

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

PHASE III (continued)

Task C

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

1. The grain-size distribution of the laboratory specimen approximates that of the compacted shale mixture in the embankment.
2. The particles in the laboratory specimen are substantially smaller than particles in the compacted shale mixture in the embankment.

Task D

1. Develop a methodology for evaluation of the end-of-construction shear strength and compressibility of the shale mixtures compacted in test strips.
2. Conduct field tests to evaluate methodology developed in D-1 above.

Task E

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

Task F

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

1. The grain-size distribution of the laboratory specimen approximates that of the compacted shale mixture in the embankment.
2. The particles in the laboratory specimen are substantially smaller than particles in the compacted shale mixture in the embankment.

Task G

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

Task H

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

1. Determining the design strength and compressibility parameters from test data.
2. Performing the stability analysis.
3. Selecting design features.

Approach to Phase II*

Federal and State highway offices and other organizations (listed in Appendix B) were contacted, and discussions were held with key personnel concerning types and causes of embankment problems, evaluation methods, and remedial treatment. Several sites were visited where embankments were either showing signs of distress or had done so in the past and had been repaired. Pertinent literature and project reports were retained or later forwarded from the State and Federal agencies; however, the majority of information concerning evaluation and remedial treatment of shale embankments was obtained from Indiana, Kentucky, and Tennessee. A general review was made of literature pertaining to evaluation and remedial treatment measures applicable to embankment problems. Results of investigations under Phase I were reviewed to aid in identification of the typical types and causes of embankment instability.

Scope of Report

This report contains results and conclusions of Phase II of the study concerning embankment evaluation techniques and remedial treatment. Types and probable causes of embankment problems are discussed with examples presented. Evaluation techniques reviewed include: (a) embankment instrumentation devices such as piezometers, settlement gages, and slope inclinometers; (b) field exploration and sampling methods which include geophysical methods, visual inspection (e.g., via borehole television or film cameras), disturbed sampling, and undisturbed sampling; (c) laboratory and in situ measurements emphasizing in situ testing; and (d) analysis and prediction of settlement and slope stability including determination of shear strength from back calculation (commonly using a slip circle or wedge stability analysis) and laboratory tests. Remedial measures investigated were: (a) pavement overlay; (b) drainage systems;

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* For convenient references, the approaches for Phases I and III are contained in Appendix A.

(c) slope flattening, berms, and buttresses; (d) retaining walls (including reinforced earth); (e) lime, cement, and chemical stabilization; and (f) reconstruction (removal and replacement). Experience with the various techniques and their advantages and disadvantages are discussed. Recommendations are given concerning evaluation and remedial methods.

II. EMBANKMENT PROBLEMS

Embankment problems can be separated into two types: settlement and slope failure. The cause of these problems is discussed below, followed by a review of settlement and slope failure conditions in which specific examples are cited.* Shale embankments can also exhibit surface heave caused by wetting of expansive clays. This problem is not discussed herein; however, it was considered in depth in a separate FHWA study conducted at the U. S. Army Engineer Waterways Experiment Station (WES) and reported by Snethen et al. (1975).

Cause of Problems

Excessive settlement and slope failure in shale embankments often result from a combination of factors; however, the susceptibility of shale materials to degradation after placement is of primary importance. The degradation process can involve swelling, slaking, and other forms of weathering of shale materials. Degradation within an embankment is dependent on properties of the borrow shales as well as the overall characteristics of the embankment and surrounding area (see Shamburger et al., 1975; Deo et al., 1973).

Shale embankments have commonly been constructed as "rock fills" with material containing large rock pieces placed dry in lifts of 3 to 4 ft with compaction by hauling and spreading equipment. This practice has resulted in large voids which are likely to be present within embankments even where soil compaction procedures are employed. Attempts to compact the shale fill in thin lifts are complicated by the natural variability in the types of borrow materials. A portion of the shales may breakdown during placement, but adequate compaction is prevented by the presence of larger, more durable shale, sandstone, or limestone pieces.

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* A general discussion of shale embankment behavior and the magnitude of the problem across the United States is given in Shamburger et al. (1975).

It is theorized that degradation is caused largely by (a) slaking of shale materials as a result of infiltration of water and subsequent seepage through the permeable embankments and/or (b) breakage of rock pieces at contacts under high compressive stress concentrations created by the weight of the overlying fill. Embankment shales can be exposed to moisture from direct rainfall, surface runoff from adjacent slopes or pavement, or subsurface seepage. Slaking of shale materials in combination with breakdown at contacts may cause large settlements possibly occurring as creep movements involving both vertical subsidence and lateral spreading. Over a period of time, the degradation process forms weak zones of soft clay or silt intermixed with shale and limestone pieces. The overall reduction in embankment shear strength increases the likelihood of failure. The majority of failures have involved sidehill fills (downhill slope), apparently because of their suspectibility to underseepage from the adjacent natural ground (see Figure 1). The low permeability of degraded materials tends to prevent free drainage of the fill. Failures normally occur during rainy periods when the embankment shearing resistance, already lowered through a period of shale degradation, is further decreased due to saturation and increased pore water pressures. Failures tend to initiate in the lower portion of the fills and move upslope, possibly because of the more frequent wetting and subsequent rapid degradation in the toe area.

Many sidehill fills are underlain by a layer of residual soils (commonly 5 to 20 ft thick) overlying the parent shale formations (see Figure 1). These soils are commonly involved in the sidehill fill failures, although the degradation of the embankment shale may be largely responsible for failure. For example, during rainy periods the residual soils (like the embankment) may be subjected to saturation and increased pore water pressures due to the constriction of drainage through the deteriorated embankment. The reduction in shear strength of the residual soils may be significant and lead to their involvement in failure of the sidehill fill. These failures are considered within the scope of this report since the embankment characteristics and behavior were likely major contributors to failure. The possible involvement of

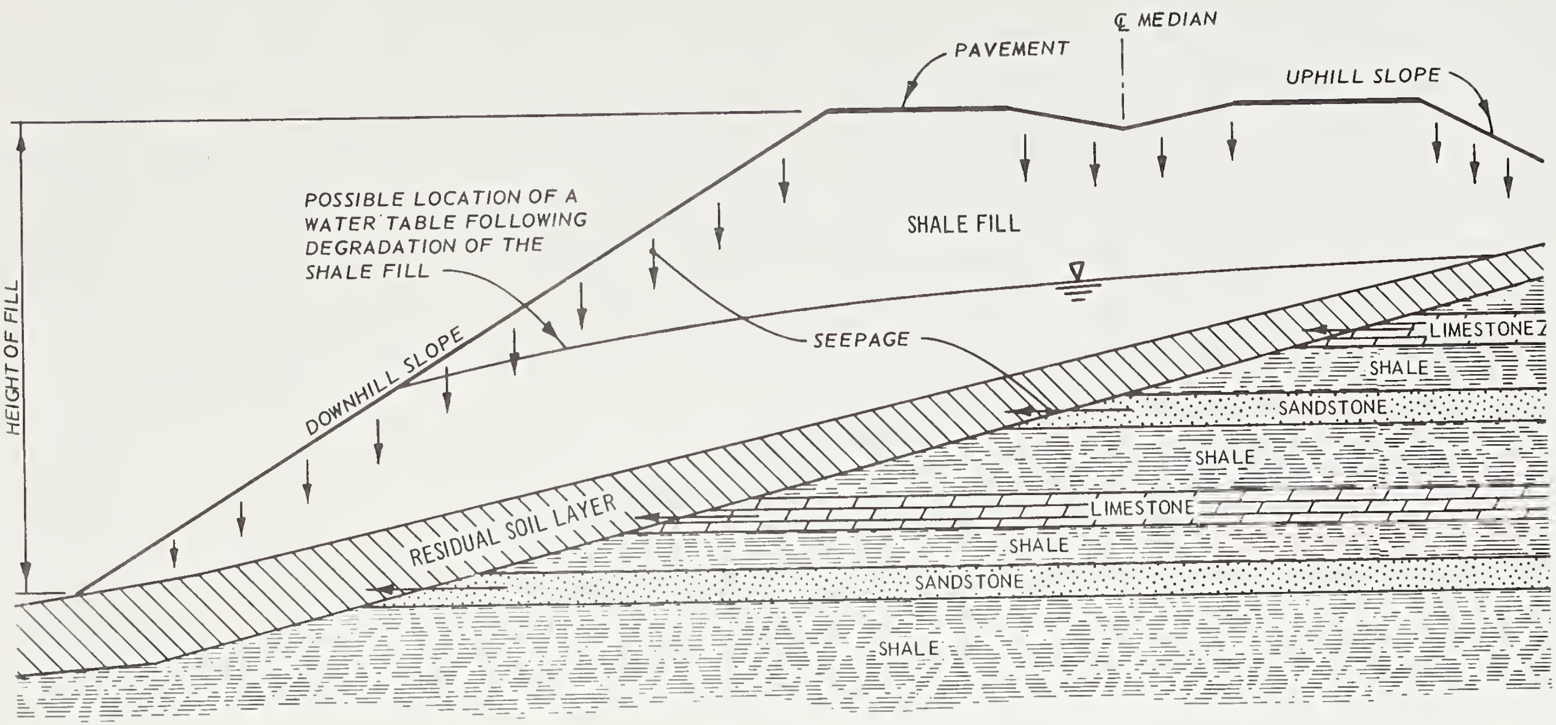

Figure 1. Movement of water through a sidehill embankment.

residual soil foundations in sidehill embankment failures should be considered in evaluation studies and in design of remedial measures.

Settlement

Excessive vertical settlement and lateral spreading of embankments can cause cracking and dips in the roadway sufficient to require costly repair. In Tennessee, several shale embankments along I-75 have undergone longitudinal cracking and nonuniform vertical settlement which have continued since initial paving of the embankments in the mid 1960's. For example, at sta 950+00, as much as 8 in. of pavement overlay has been required. Borings have shown the fill to be a heterogeneous mixture of "weathered shale and siltstone of all sizes and stages of decomposition" (Law Engineering Testing Company, 1972).* Large portions of the fill were shown to consist of shale and siltstone particles embedded in a silty clay matrix, while in other areas boulders and granular material were encountered with little soil matrix.

During construction of a shale fill for a ramp in connection with the four-level I-75 and I-275 interchange near Covington, Kentucky, a bridge column moved 11 in. from planned center as a result of vertical settlement and lateral movement of the embankment surrounding the column. The fills involved were placed in 1-ft lifts and compacted with a sheepsfoot roller; however, it is possible that proper compaction was hampered by large slabby limestone fragments contained in the fills.

In West Virginia, a series of shale fills along U. S. Highway 460 have experienced as much as 12 to 15 in. of vertical settlement requiring intermittent pavement overlays since roadway completion in 1971. The shales were placed as rock fill in 24-in. lifts and compacted by hauling and spreading equipment.

Embankment movements become most troublesome near bridge approaches

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* Law Engineering Testing Company (1972), "Report of Fill Investigations, Interstate Route I-75, Campbell County, Tennessee: State Agreement Number 0133, Priority 1: Fill Areas," Volumes 1 and 2, B-1440, conducted for Tennessee Highway Department.

and abutments. Vertical and lateral movements of shale embankments have caused tilting, translation, and cracking of abutment structures and associated cracking and distortion of approach pavements. In one case in Ohio, a 6-in. vertical settlement of an abutment occurred prior to placement of the deck concrete. This settlement required the addition of steel plates under the abutment bearings. During the following 3-yr period, an additional 12-in. settlement accompanied by lateral movement occurred which required raising the abutment and approaches. Eventually, the lateral deformation forced contact between the abutment and superstructure steel. In certain cases, the Ohio Department of Transportation has corrected abutment movements by injecting cement grout into the embankments. The volume of grout used in these operations suggests that the grout served to fill large voids within the shale fill. These types of problems led to the development in 1963 of embankment construction specifications requiring shale to be placed and compacted as soil fill. The Ohio Department of Transportation reported that shale embankments built under current specifications have performed satisfactorily.

Twin bridges in Clifton Forge, Virginia, have been experiencing abutment settlement problems since their completion in early 1971. The shale fill at the abutments was placed in about 2-ft lifts and compacted. By late 1972, vertical and lateral deformations were sufficient to require raising the abutments and horizontal trimming of the superstructure steel. Settlement of the abutments is continuing, and additional repair will likely be necessary.

Slope Failure

Settlement and associated dips and cracks in the pavement have often been the prelude to the more costly problem of slope failure. Several embankments along I-75 in Tennessee and Kentucky have failed following a 5- to 10-year period of pavement cracking and subsidence. A shale embankment slope failure affecting some 500 ft of roadway near sta 840+00 along I-75 in Tennessee is shown in Figure 2. During a 7-yr period prior to failure (1965-1972), pavement cracking and surface

Figure 2. Embankment slide at sta 840+00, I-75, Tennessee (courtesy of the Tennessee Department of Transportation).

subsidence required periodic pavement overlays. This failure and other embankment failures along I-75 were investigated and results reported by Law Engineering Testing Company (1972).* The slide and subsequent investigations at sta 840+00 are also discussed by Tice and Sams (1974). The embankment was a sidehill fill approximately 140 ft in height** and included zones of rapidly deteriorating siltstone and shale. Water entering the fill from surface runoff and subsurface seepage apparently caused deterioration of the shale. Slope inclinometer data and boring explorations indicated that the shear surface was contained largely within zones of shale-siltstone fragments surrounded by a silty clay matrix. Water level data indicated the existence of a water table high within the fill, and numerous areas of seepage were observed on the failed slope. According to Royster (1973), the failure was initiated in the toe area and progressed upslope. The original embankment, failure stages, and corrected slope are depicted in Figures 3-6.

Three embankment failures in southeastern Indiana (see location maps, Figures 7 and 8) were similar to the slide at sta 840+00 (I-75), Tennessee. The failures involved sidehill fills and occurred approximately 10 yr after construction. Borrow materials for the fills consisted of a mixture of limestone, shale, and clay weathered primarily from the shale. The fill was generally constructed as rock fill with borrow material placed in lifts up to 4 to 5 ft. Large voids within the embankments permitted surface and subsurface water to permeate through the fill causing degradation of the shale and saturation of the fills. Settlement problems in the form of bumps and sags in the pavement became apparent approximately 4 to 5 yr prior to slope failure. Figures 9-12 show the failed slope and the 10- to 15-ft-high failure scarp directly beneath the roadway at the location of SLIDE-1 (Figure 8). Limestone and shale pieces and soil mixtures were observed along the failure scarp, and the water table was noted to be high within the fill (Wood et al.,

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* See footnote on page 10.

** Heights given for sidehill fills refer to the height of the downhill slope as shown in Figure 1.

1973). Each of the embankments were underlain by a 10- to 15-ft-thick layer of residual soil overlying shale. The residual soil zone was apparently involved in SLIDE-3 as evidenced by the uplift of natural ground to a distance of 25 to 30 ft beyond the toe of the 15-ft-high fill.

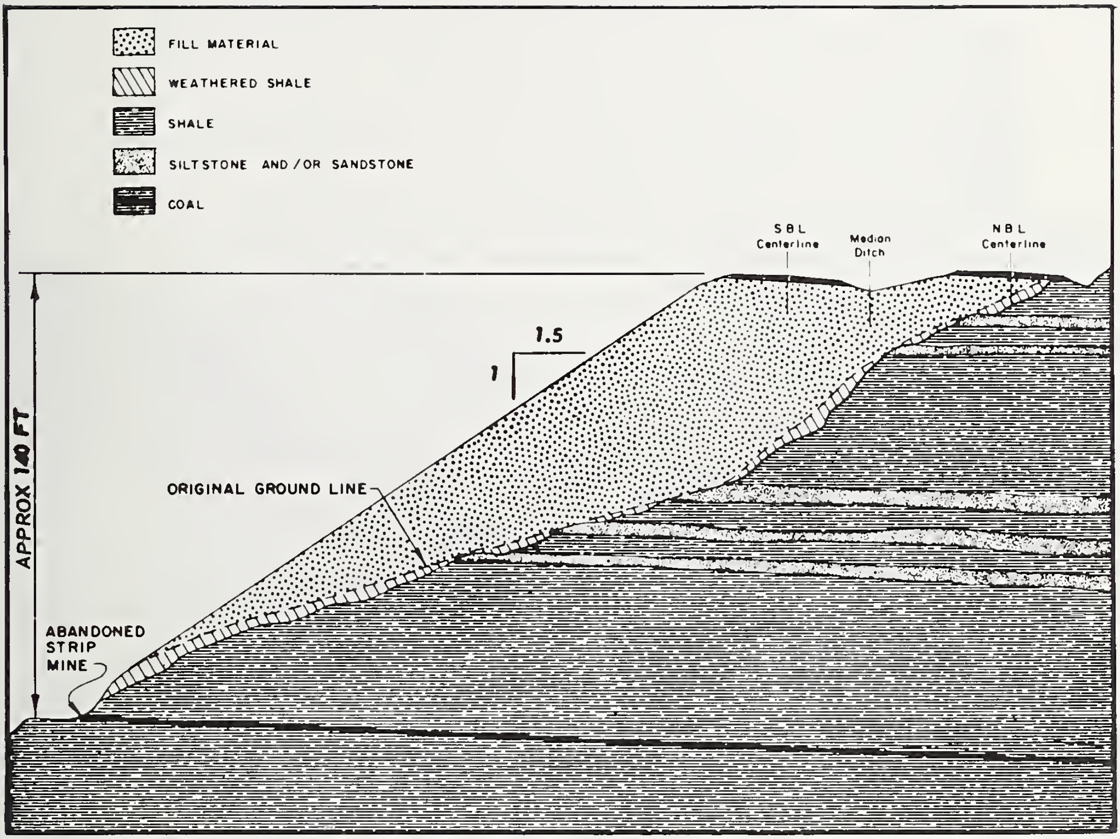

Figure 3. Sta 840+00, I-75, embankment as constructed (Royster, 1973).

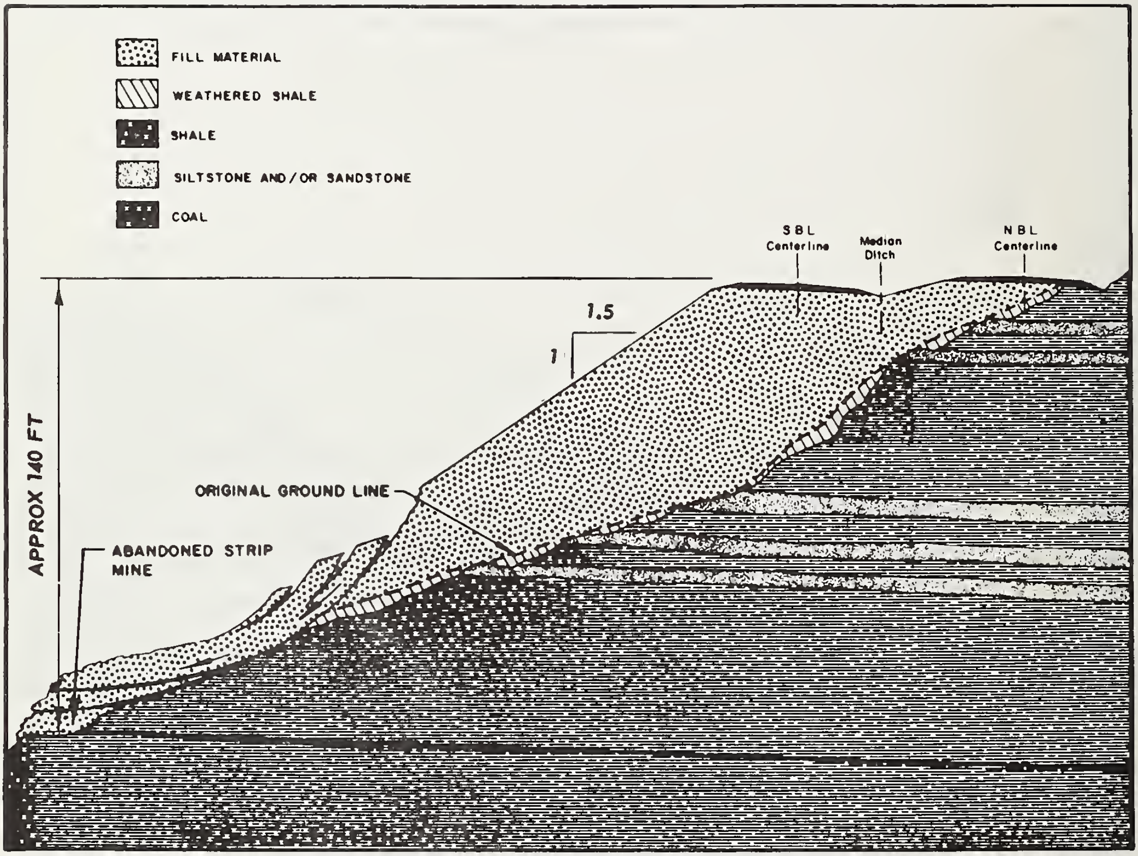

Figure 4. Sta 840+00, I-75, embankment in first stage of failure (Royster, 1973).

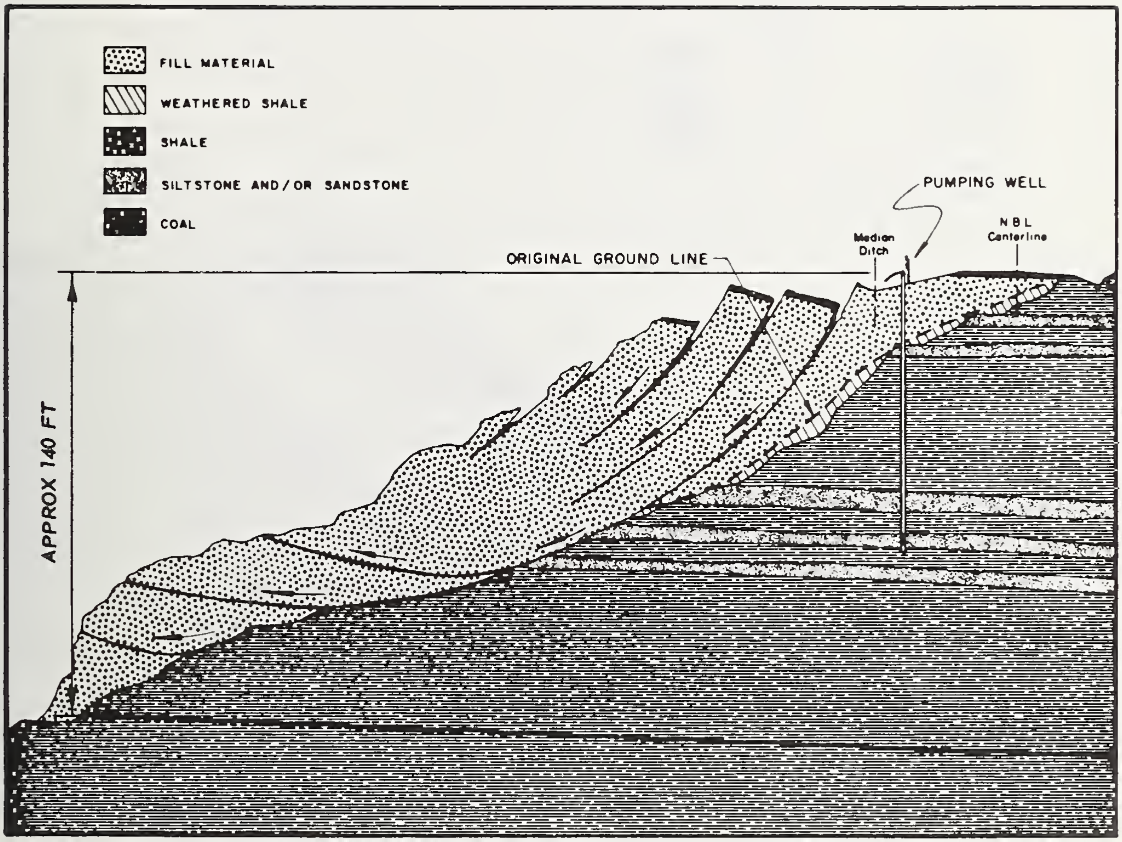

Figure 5. Sta 840+00, I-75, embankment in last stage of failure (Royster, 1973).

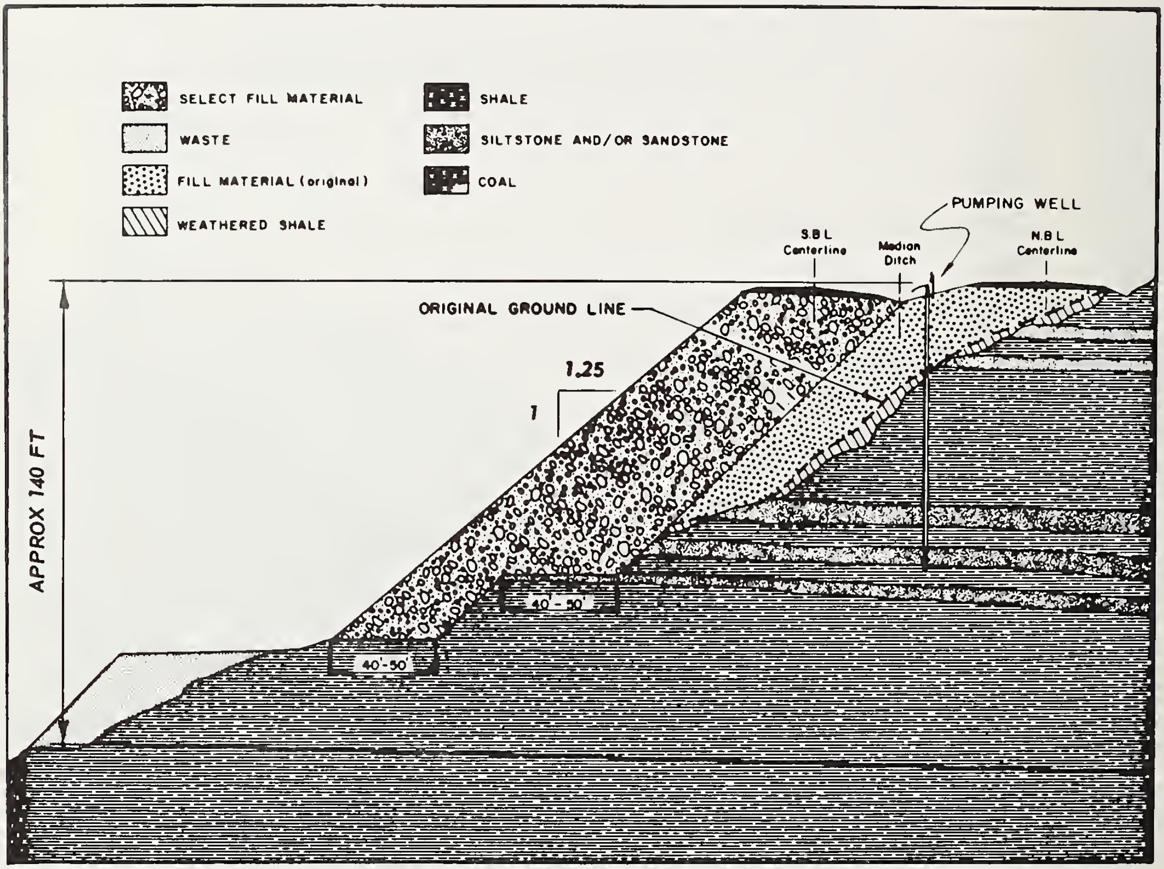

Figure 6. Sta 840+00, I-75, embankment after repair (Royster, 1973).

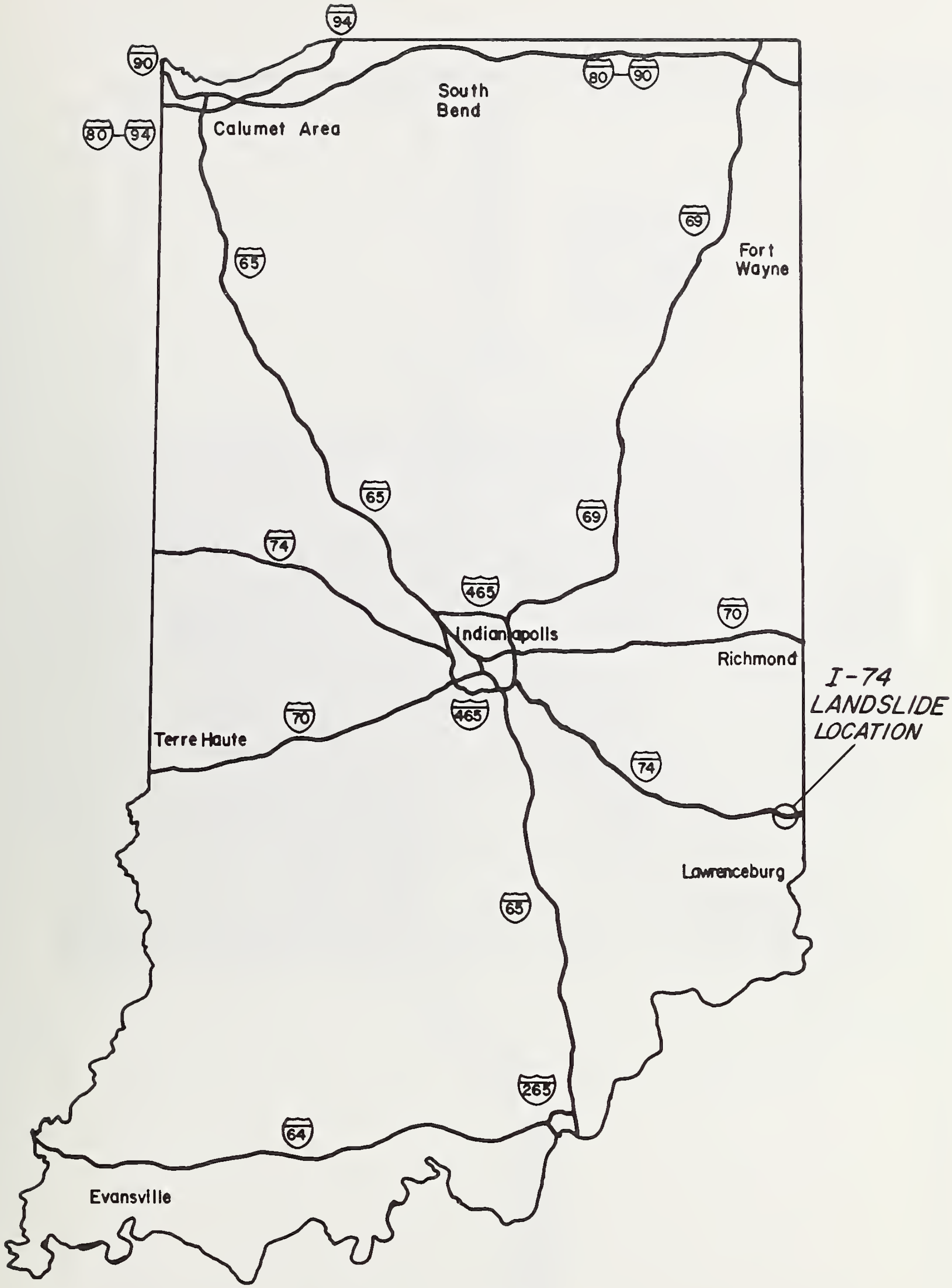

Figure 7. General location of embankment problem area, I-74, Indiana (courtesy of the Indiana State Highway Commission).

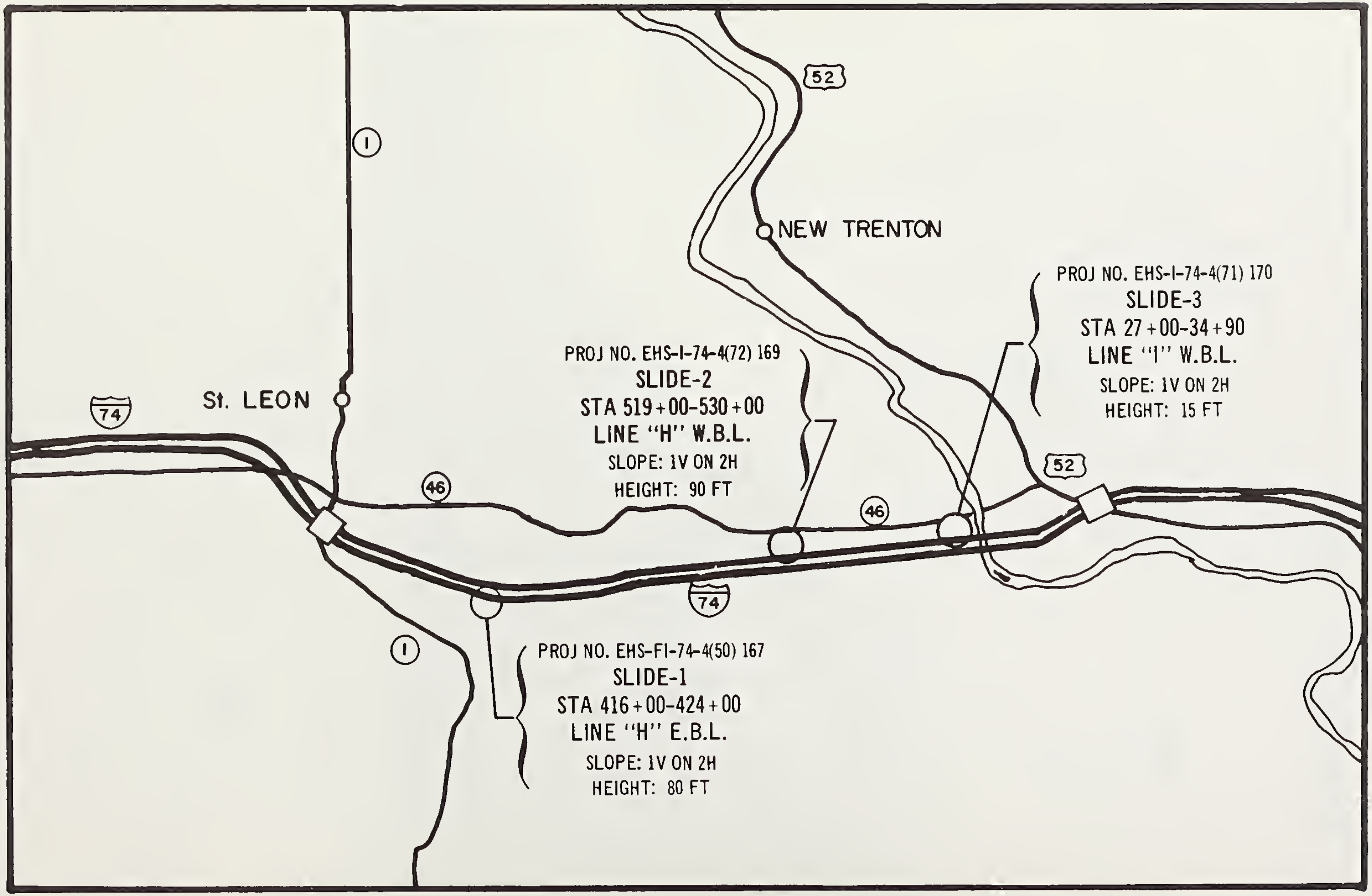

Figure 8. Location of embankment failures, I-74, Indiana (courtesy of the Indiana State Highway Commission).

Figure 9. Western portion of failure scarp, SLIDE-1 on I-74, Indiana (courtesy of the Indiana State Highway Commission).

Figure 10. Central portion of failure scarp, SLIDE-1 on I-74, Indiana (courtesy of the Indiana State Highway Commission).

Figure 11. Eastern portion of failure scarp, SLIDE-1 on I-74, Indiana (courtesy of the Indiana State Highway Commission).

Figure 12. Failed slope, SLIDE-1 on I-74, Indiana (courtesy of the Indiana State Highway Commission).

III. EVALUATION TECHNIQUES

When settlements of the roadway embankment become excessive or the number or rate of recurrence of surface slides on the embankment slopes becomes too large, attempts to evaluate the embankment should be undertaken. The purpose of such an evaluation is to determine whether the settlements or surface slides are indications of impending failures. Continuing settlements could be indications of a breakdown of the fill material which might eventually lead to a decrease in permeability of the embankment, a reduction in shear strength, and a shear failure. Surface slides could also be indicative of a breakdown of the material near the surface of the slope due to inadequate drainage provisions. This surface breakdown can lead to a progressive deterioration deeper into the fill and a shear failure of the embankment that would involve all or part of the roadway.

The first step in an evaluation should be a review of the history of design and construction of the distressed embankment section including the geology of the area, the foundation conditions, and the materials used in the embankment and their method of placement and compaction. The next step should be a field reconnaissance of the site. Field examination would emphasize an inspection of the site to locate causes of the problem such as seepage exiting from the slopes, clogged drainage systems, or movement as indicated by leaning trees or fence posts. Should the review of the site disclose reasons for the continuing problems, corrective actions can be initiated; however, if the review does not disclose such reasons, additional methods of investigation can be used.

Instrumentation

Several common and inexpensive methods of instrumenting a fill are available to the engineer as an aid in evaluating an embankment's present and future behavior. Instrumentation is usually used in conjunction with other methods, discussed in subsequent paragraphs, to define parameters for analyses. In the following paragraphs examples will be given showing the installation of certain instrumentation. For cases in which the instrumentation is believed applicable to compacted shale embankments, examples from other types of embankments will be used to illustrate methods of placement or location.

Piezometers. A piezometer is used to determine the groundwater level or pore water pressure within a medium. In certain cases, the field permeability may be determined from piezometer records. Further interpretation of piezometer data can indicate the source of the water and the efficiency of any drainage system beneath the fill. Information on different types of piezometers and guidance on selection, installation, and use in measuring groundwater levels and pore water pressures are given in EM 1110-2-1908 (USAE, OCE, 1971a). The three basic types of piezometers are open-system piezometers, hydraulic or closed-system piezometers, and diaphragm piezometers. Table 2 compares the advantages and disadvantages of these three types. A list of piezometers commonly used on Corps of Engineers (CE) projects is shown in Table 3.

A major consideration in selecting the proper type of piezometer for measuring pore water pressures is the permeability of the medium in which the pore water pressure is to be measured. Selection of a piezometer should be based upon the cost, durability, reliability, ease of installation, site conditions, expected fluctuation of the water table at the site, frequency of observations, and the response time of the piezometer (Terzaghi and Peck, 1967). Response time is perhaps the most important of these considerations from the standpoint of the data collection. If the piezometer installed does not respond quickly enough to changes in groundwater conditions, the data collected will probably not reflect field conditions at the time of the observation. For this reason, care should be exercised in selecting a piezometer, and tests for determination of response time should be conducted after installation.

Procedures have been outlined for determining the response time (Hvorslev, 1951; Terzaghi and Peck, 1967); the procedure basically

Table 2. Comparison of piezometer types as given in EM 1110-2-1908 (USAE, OCE, 1971a).

Basic Type	Advantages	Disadvantages
Open-system	Simple; comparatively inexpensive; generally not subject to freezing; relatively long life; fairly easy to install; long history of effective operation.	Long time lag in impervious soils; cannot measure negative pore pressure; cannot be used in areas subject to inundation unless offset standpipe is used; must be guarded during construction; no central observation station is possible; requires sounding probe.
Closed-system	Small time lag in any soil; can measure negative pore pressures; can be used in areas subject to inundation; comparatively little interference with construction; can be read at central observation stations.	Observation station must be protected against freezing; fairly difficult to install; fairly expensive compared to open systems; sometimes difficult to maintain an air-free system; most types are fragile; some types have limited service behavior records.
Diaphragm	Simple to operate; elevation of observation station is independent of elevation of piezometer tip; no protection against freezing required; no de-airing required; very small time lag.	Limited performance data, some unsatisfactory experience; some makes are expensive and require expensive readout devices; fragile and requires careful handling during installation.
	Pneumatic. Electrical source not required; tip and readout devices are less expensive than for electrical diaphragm types.	Often difficult to detect when escape of gas starts; negative pressures cannot be measured; condensation of moisture occurs in cell unless dry gas is used; requires careful application of gas pressure during observation to avoid damage to cell.
	Electrical. Negative pressures can be measured.	Devices subject to full and partial short-circuits and repairs to conductors introduce errors; some makes require temperature compensation and have problems with zero drift of strain gages; resistance and stray currents in long conductors are a problem in some makes.

Table 3. Commonly used piezometers as given in EM 1110-2-1908 (USAE, OCE, 1971a).

Type	Name	Manufacturer or U. S. Supplier
Open-system	Casagrande	Locally fabricated or several suppliers
	Geonor	Soil and Rock Instrumentation, Inc. 377 Elliot St. Newton Upper Falls, Mass. 02164
	Wellpoint	Local suppliers
	Portland District	Locally fabricated
Closed-system	USBR	Plasticrafts, Inc. 2800 North Speer Blvd. Denver, Colo. 80211
	Bishop	Soil Instruments, Ltd. Townsend Lane London NW9, England
Diaphragm		
Pneumatic	Warlam	A. A. Warlam Box 122 Saddle River, N. J. 07458
	Hall	Geo-Testing, Inc. P. O. Box 959 San Rafael, Calif. 94902
	Dames & Moore	Dames & Moore 2333 West 3rd St. Los Angeles, Calif. 90057
	Terra Tec Thorpezio	Terra Tec, Inc. 250 N. E. 49th St. Seattle, Washington 98105
Hydraulic	Terrametrics Hydrostatic Pore Pressure Cell	Terrametrics 16027 West 5th Ave. Golden, Colo. 80401
	Gloetzl	Terrametrics
Electric strain gage	Carlson	Terrametrics
	Wes Transducer	U. S. Army Engineer Waterways Experiment Station P. O. Box 631 Vicksburg, Miss. 39180
	University of Alberta, GSC	Locally fabricated
Electropneumatic	Pore Pressure Transducer	Slope Indicator 3668 Albion Place North Seattle, Wash. 98103
Electrical acoustical	Maihak	Soil and Rock Instrumentation, Inc.
	Telemac	Soil and Rock Instrumentation, Inc.
	Geonor	Soil and Rock Instrumentation, Inc.

consists of either filling with water or emptying the piezometer and measuring the time it takes to approach its initial equilibrium condition. This is done by measuring the change in piezometric level at different time intervals. From these data, the basic time lag can be determined. The exact value of the time lag is not as important in the evaluation of the fill as the knowledge of the drainage characteristics of the fill obtained from such a test. Even though equations have been presented for determining permeability from piezometer readings (Hvorslev, 1951), the knowledge that the fill is "free draining" or "relatively impermeable" is important in an analysis.

Figure 13 shows a plot of response time versus permeability for various types of piezometers. From permeability tests on compacted (AASHTO T 99) samples of crushed shale and shale-rock mixtures for CE projects (Table 14, Shamburger et al., 1975), the permeability of shale embankments is estimated to be on the order of 10^{-1} to 10^{-3} cm/sec. Based on this permeability estimate and because the open-system piezometer is the simplest, least expensive, and easiest to install, it is considered to be most applicable for use in compacted shale embankments. The Casagrande open-standpipe porous-tube piezometer and the well point piezometer are most frequently used (Figure 14). Also, a simple slotted or drilled pipe is sometimes used. Several different types of piezometers were used by the States in an attempt to evaluate problem fills. Tennessee has used slotted inclinometer casings as piezometers in evaluating several I-75 embankments (Law Engineering Testing Company, 1972).* Kentucky has also used perforated pipes in bridge abutments on the Bluegrass Parkway bridges over the Chaplin River; however, little useful data were obtained (Hopkins, 1973). Casagrande-type piezometers have also been used by Kentucky (Hopkins et al., 1973; Hopkins, 1969). Nebraska has used the single-tube Casagrande-type piezometer in evaluating a clay fill (Nebraska Department of Roads, 1968).

A typical embankment instrumented with piezometers is shown in Figure 15. Plotting of piezometer elevations versus time along with

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* See footnote on page 10.

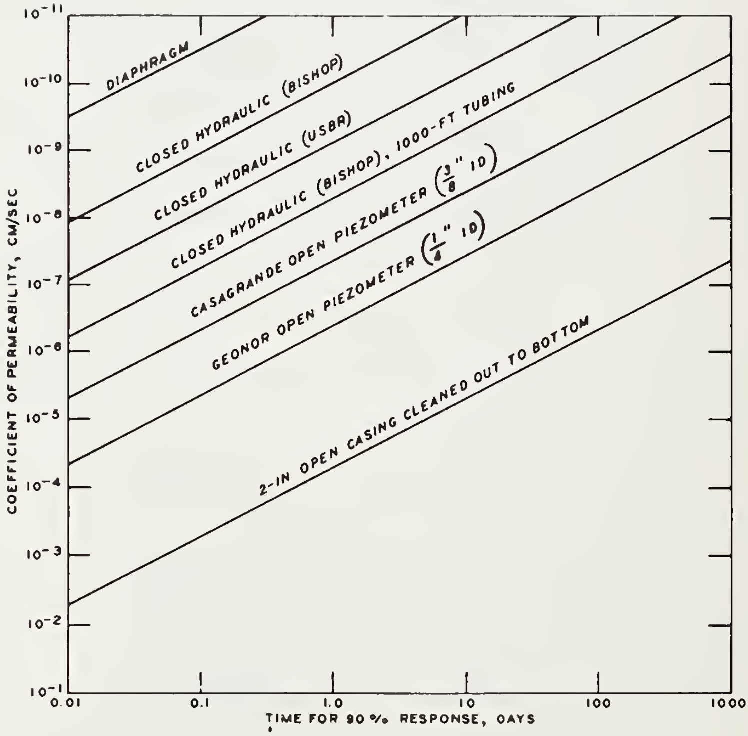
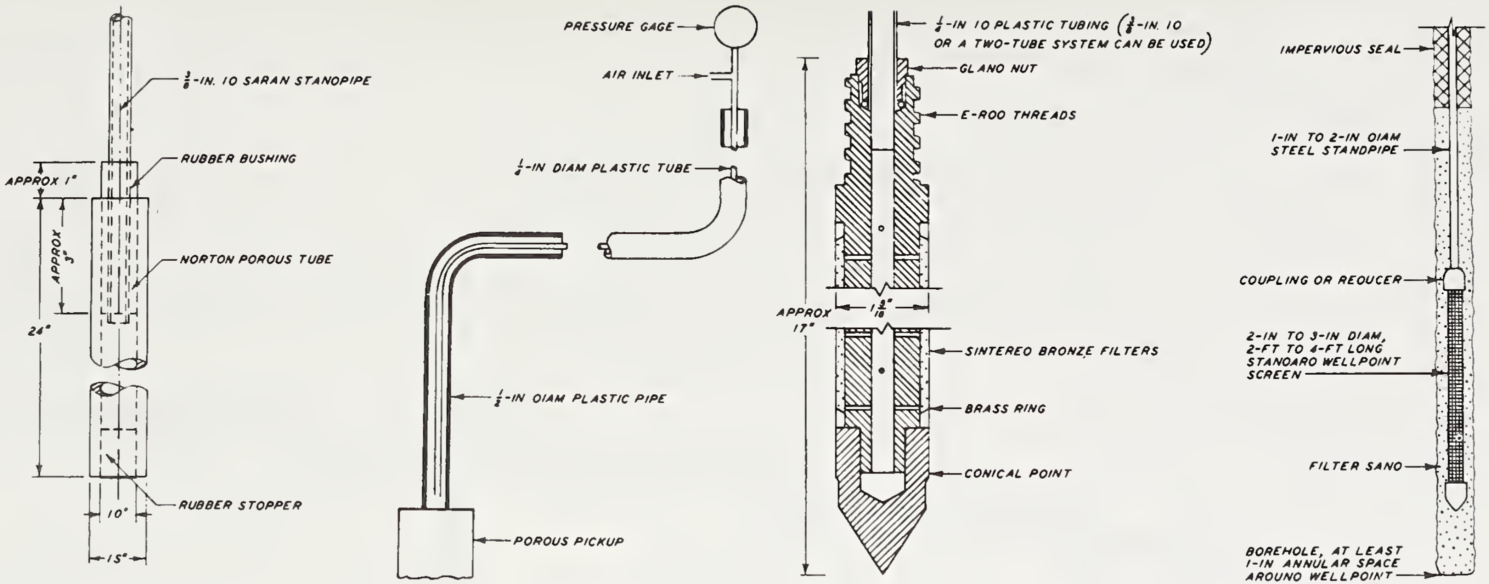


Figure 13. Approximate response time for various types of piezometers (Terzaghi and Peck, 1967).



a. CASAGRANDE POROUS TUBE

b. TWIN-TUBE CASAGRANDE (AFTER DE LUCCIA, 1958)

c. GEONOR PIEZOMETER

d. WELLPOINT

Figure 14. Open-system piezometers as given in EM 1110-2-1908 (USAE, OCE, 1971a).

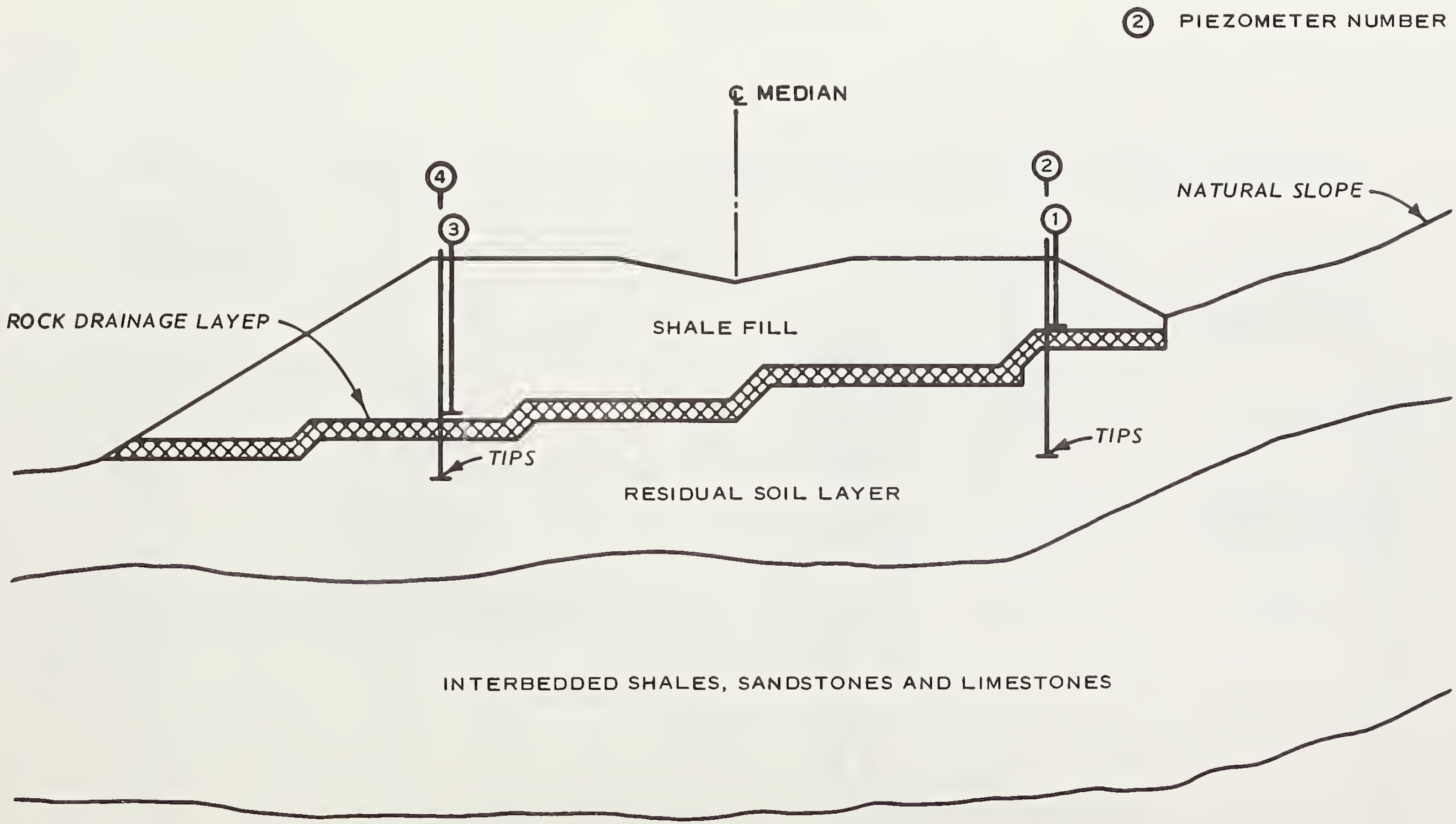


Figure 15. Typical piezometer locations.

rainfall data or local stream flow can pinpoint the source of water within a fill. Examination of the readings will indicate a well drained fill if readings for embankment piezometers 1 and 3 are always near the elevation of the top of the drainage blanket or if they are always dry. Piezometers 2 and 4 will reflect groundwater elevations within the foundation. A fill with poor drainage will conversely be indicated if piezometers 1 and 3 indicated piezometric elevations within the embankment; and foundation piezometers 2 and 4 also indicated elevations within the embankment. In such a case, the embankment will have actually blocked a natural drainage pattern, a condition long recognized as potentially dangerous (Highway Research Board, 1958). Further indications of poor drainage can be seen if the water surface elevations in piezometers 1 and 3 increased within the embankment following periods of heavy rainfall and took an extended time to regain their normal levels after the rainfall had ceased.

The frequency of readings obtained from a system of piezometers will depend upon a number of factors and will most likely be altered many times before a suitable interval is obtained. Some of the factors affecting the frequency of reading are:

- a. Rainfall, stream fluctuation, or groundwater changes.
- b. Magnitude of problems under study.
- c. Initial evaluation of water surface at the time of installation.
- d. Character of fill material.
- e. Water level fluctuations within the fill.
- f. Schedule of reading of other instrumentation.
- g. Proximity of personnel for making readings.

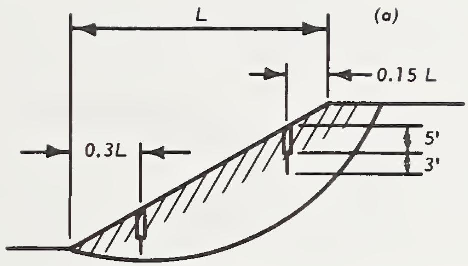
The evaluator must consider all the factors involved and obtain an optimum interval for reading the piezometers.

Surface deflection measurements. Distress may first appear as horizontal and lateral movement of the fill. When this occurs, an indication of the seriousness of the problem and the magnitude of future movements may be obtained by observing the movement. Conventional surveying methods can be employed for monitoring movements on markers, pins, or stakes placed on the embankment surface.

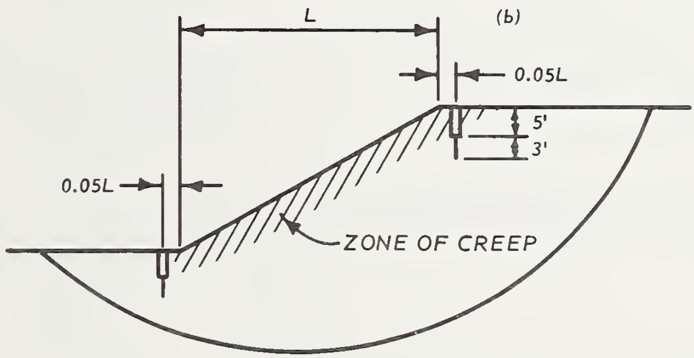
Establishing lines of markers for measuring settlement and horizontal movement can be helpful in locating potential failure surfaces. These lines can be established on the roadway, on the embankment slope, and on natural ground at the toe of the slope. Figure 16 gives a recommended position of markers for different types of expected movement. Care must be taken in establishing lines for monitoring settlements if horizontal movement is also to be measured. The reference point used in establishing the lines must be outside the zone of expected movement. Because of creep of the slope's outer portion (Figure 16), markers should be installed such that the upper portion of the marker is not in contact with the fill material. This may be accomplished by drilling a 4- to 6-in. hole to a depth of about 5 ft, casing it, and then driving a reference rod approximately 2 ft below the bottom of the casing (Terzaghi and Peck, 1967). Horizontal and vertical displacements can then be measured using the top of the rod. Figure 17a shows typical lines (A and B) established on a California fill for monitoring settlement and horizontal movement. Figures 17b and 17c show the results of observations.

Simple profiling is believed to be the most useful method of obtaining deflection data on roadway embankments. Profiles are usually made parallel or perpendicular to the roadway center line. Perpendicular profiles will be very useful in distinguishing between settlement and bulging of a fill. The interval between profiles depends upon the particular section under study.

Another type of surface measurement is made after the development of cracks and involves monitoring of changes in the crack openings. The measurements are generally made using referenced markers located on both sides of the crack simply by taping the distance between markers or by using more complicated devices such as vernier target gages (Wilson, 1967). Surface extensometers are also available for monitoring changes in crack openings and other surface movements. Table 4 lists several available methods of measuring surface movements. No reported uses of surface extensometers in evaluating highway embankments were found in this study.



a. POSSIBLE SLIDE ALONG TOE CIRCLE



b. POSSIBLE BASE FAILURE

Figure 16. Position of reference points to detect movement of slope (Terzaghi and Peck, 1967).

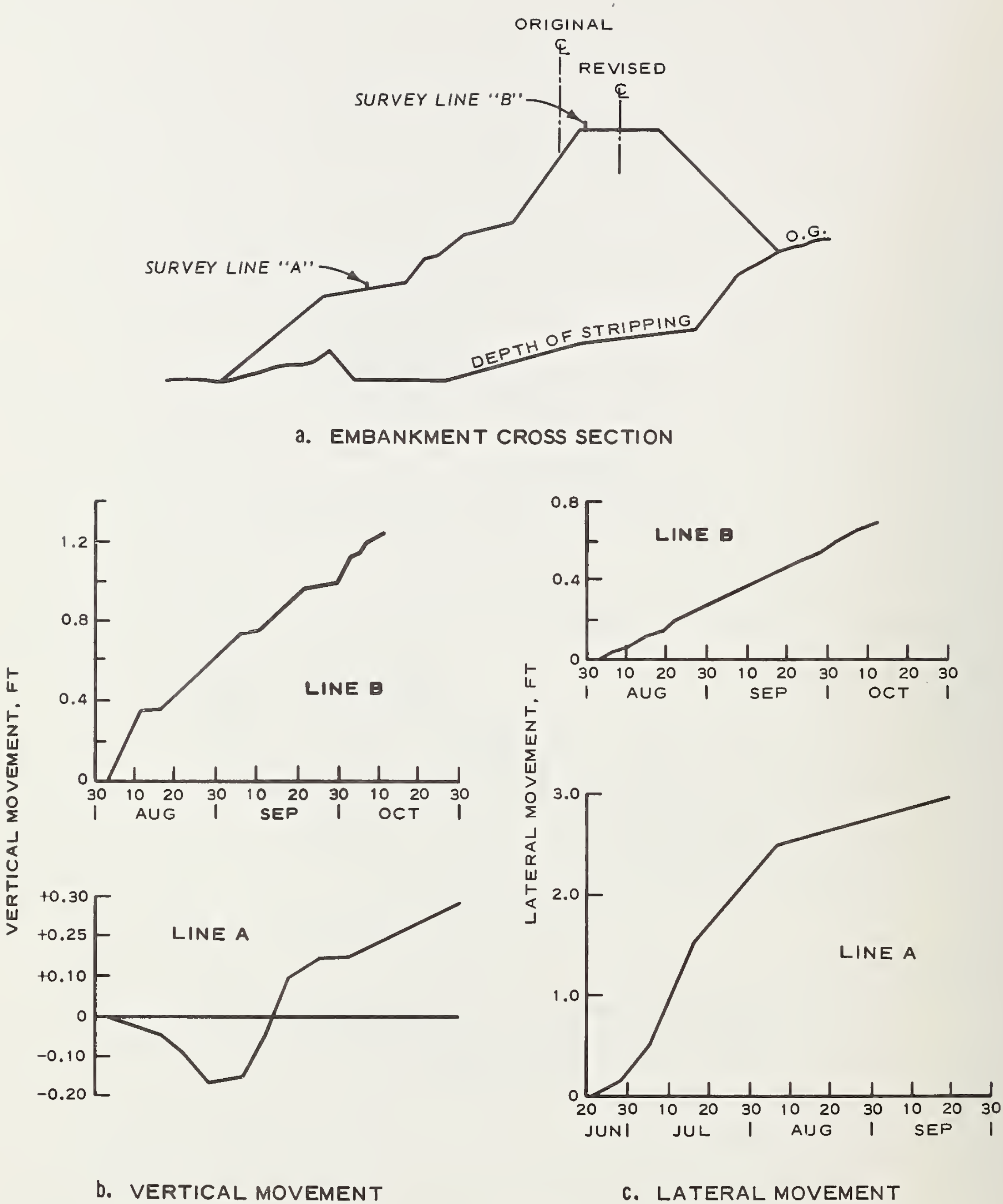


Figure 17. Vertical and lateral movement of highway embankment on U. S. 101 in California (after Durr, 1974).

Table 4. Surface movement measurement devices (Franklin and Denton, 1973).

Type	Operating principle	Gauge length (mm)	Reading range (mm)	Sensitivity (mm)	Manufacturer
Dial gauge Extensometer	Dial gauge micrometer incorporated in bar and lever system usually portable and locating in targets cemented to rock	To suit	—	0·02	Soil Instruments
		250 (50–2 000 available)	5	0·0002	W. H. Mayes
Vibrating wire extensometer	Similar to dial extensometer but employing vibrating wire measuring device in place of micrometer	114–250	0·1–100	0·00002	Maihak
		92	0·08	0·00005	Geonor
		100	0·30	0·00001	Slope Indicator
Tensioned tape or wire	Wire or tape extended between anchor points and incorporating transducer measurements	Up to 20 metres	50 + tape	0·1	Terrametrics Interfels
Disc and stylus	Stylus inscribes movement on acrylic disc	Up to 75	5	0·05	Building Research station (not marketed)
Photoelastic disc	Disc cemented into portable instrument: records displacement as photo-elastic pattern	250	5	0·0002	Stress Engineering

Movement within embankments. The monitoring of movement at depths within an embankment is also important in the detection and possibly the prevention of failures. When an embankment is unstable, movement often begins long before failure. Numerous examples can be found in the literature in which the horizontal displacement of a slope and settlement progressed until a failure occurred or remedial measures were taken (Henderson and Matich, 1962; Wilson, 1962; Wilson and Hancock, 1965).

Wilson (1970) reports movements of a natural slope (Portuguese Bend, California) that were initiated by placement of slide debris adjacent to an existing roadway in the spring and summer of 1956. Cracking of a concrete outlet structure beneath the roadway was found which required frequent repair. Inclinometers were installed and movement was still being observed in 1970. In another case illustrated in Figure 17, early detection of horizontal movement and settlement led to the initiation of remedial measures that corrected the problem and made reconstruction unnecessary (Durr, 1974).

Elaborate methods have been devised to study movements during and after construction in dams and roadway embankments (Smith and Weber, 1969; USAE, OCE, 1971b; Hopkins, 1969; Hopkins and Deen, 1972; Wilson, 1967; and others). Table 5 lists various types of instrumentation used for the measurement of movements within an embankment. Some of these devices are not appropriate for evaluating existing embankments since they must be installed during construction; however, they are included to show the variety of devices used when evaluation efforts are carefully planned from the design stage.

Most of the settlement devices installed during construction facilitate the measurement of the changes in elevation of a plate or some other object placed within the fill. Access to the plate is attained through a vertical riser. Measurements are made using a probe or by leveling to a vertical rod attached to the plate. Liquid-filled settlement gages which involve the measurement of a head difference between an instrument house and a cell or tube within the embankment have also been used (Figure 18).

As with settlement measurements, horizontal displacements within

Table 5. Selected movement devices as given in EM 1110-2-1908 (USAE, OCE, 1971b) and Franklin and Denton, 1973.

Type	Operating Principle	Reading Range ft	Sensitivity ft	Manufacturer
Hydraulic BRS Settlement device	Simple U-tubes with water. One-half set in tbe ground and balanced with standpipe in gauge house	±1.64	±0.003	ELE Soil Instruments
Mercury/pneumatic RRL Settlement device	Mercury U-tube used in conjunction with gas back pressure system. Electric contacts in ground used to obtain balance	6.56	±0.006	ELE Soil Instruments
Pneumatic/mercury Settlement device	Pneumatic pressure is used to balance a static mercury column in third tube system and measured to calculate the settlement	6.56 With pressure gauge 65.61 With transducer	0.003-0.030	Soil Instruments
Pneumatic/mercury Settlement device	Pneumatic pressure used to balance mercury in third tube system. Static or portable operating at any angle and along pipe systems	±9.84	0.015	Terratec
Geonor Settlement probe	Levels run to top of rod anchored at elevation below which settlement is desired	No limit	Depends upon accuracy of level circuits	Geomeasurements, Inc.
USBR internal vertical movement device	Probe lowered into vertical pipe senses location of cross arms installed at various intervals within fill (multi-point)	No limit	0.01	Local
Idel horizontal and vertical movement device	Magnetic field broken by plates embedded in fill when detector lowered through tubing. Probe transmits signal to receiver. Depth when signal is heard is measured by length of scaled cable in tubing	No limit	0.003	Terrametrics
Settlement probe	Probe used to detect lower edge of telescoping casing at joints	No limit	Standard surveyor's tape	Slope Indicator Co.
USBR horizontal movement device	Horizontal movements sensed by two vertical plates and transformed into vertical movements of two pipe counter weights	No limit	0.01	Local
Department of Water Resources horizontal movement device	Change from initial position of a mark on a tensioned cable within a horizontal casing in a fill is measured	No limit		Local

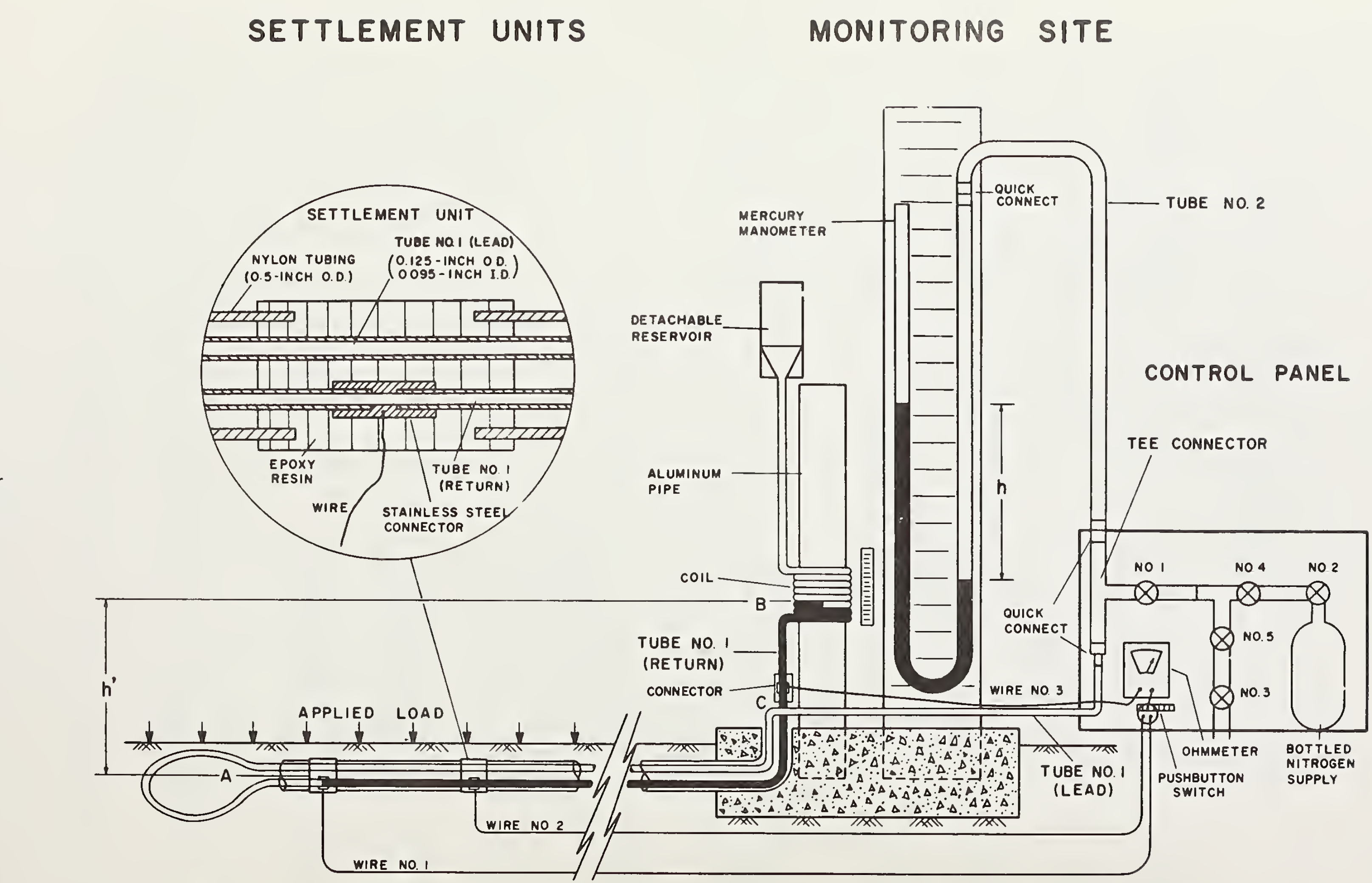


Figure 18. Components of multiple-point, mercury-filled settlement gage (Hopkins and Deen, 1972).

the embankment can easily be measured if the device is installed during construction. Most of these systems entail the installation of a network of vertical anchor plates and horizontal wires running through tubing to an instrument house located at the same elevation on the fill slope. The wires are tensioned, and the horizontal movement is computed by measuring the distance between the initial position of a mark on the wire and its present position (Figure 19).

Establishing a system of settlement and horizontal displacement measurement devices during construction in all highway sections in which stability problems are likely would be an impossible task. Therefore, in evaluating an existing embankment, either surface observations must suffice or another means of determining internal deformations must be used.

Both settlement and horizontal displacements at depths can be measured by the installation of various types of extensometers in boreholes (Table 6). The installation of these devices involves the anchoring of a wire, rod, or electrical measuring device at one or more depths within the hole. The magnitude of movement is measured by computing the change from the initial position of a mark on the rod or wire to its position at the time of the observation. Electronic means can be used for determining the movement in some installations. Settlement observations require a vertical hole and can provide settlement data at multiple depths. Horizontal displacement observations require horizontal holes and can provide movement data at different intervals along a horizontal line through the embankment. To measure horizontal displacements at different elevations, a separate installation is required for each elevation. Examples of the use of extensometers in a highway embankment have not been found in this study; however, they are workable means of obtaining movement data within a fill and should be considered in planning evaluation methods.

Another method of obtaining horizontal displacements is by the use of inclinometers which are devices for measuring the lateral deflection or tilt of boreholes. From a series of inclinometer observations, the depth, magnitude, and rate of lateral movement can be determined.

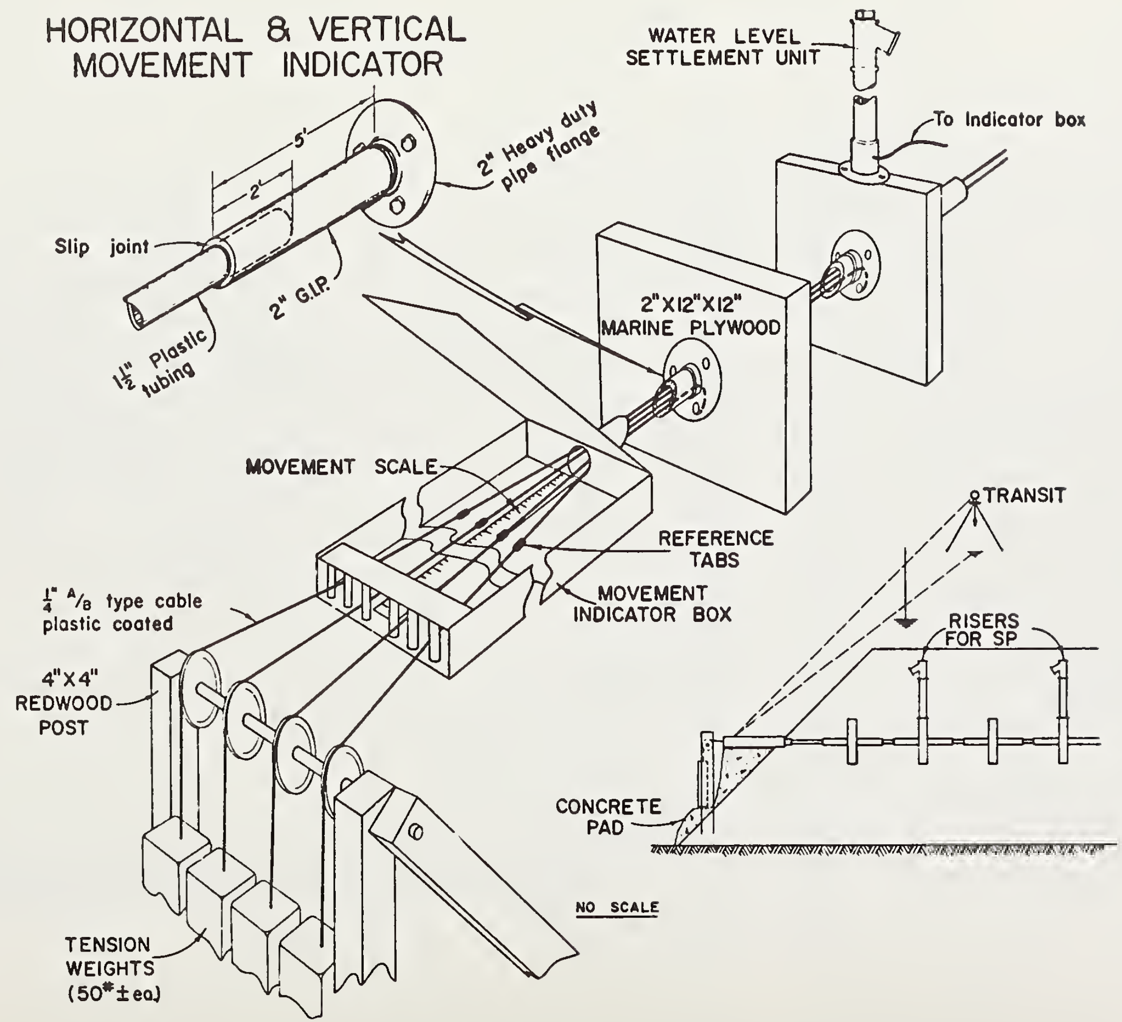


Figure 19. Horizontal and vertical movement indicator (Howe and Leech, 1966).

Table 6. Borehole extensometers (Franklin and Denton, 1973).

Type	Maximum No. anchors	Type of anchor	Drillhole diameter mm	Range without reset mm	Sensitivity mm	Manufacturer
Multi-wire vernier MK1 MK2	Not stated	2 Hydraulic 1 Mechanical	48–139 40–60	25 150	0·025 0·025	Peter Smith Instrumentation
Multi-wire	8	Mechanical	56	50 25	0·03 0·08	Terrametrics
Multi-wire screw micrometer	10	Mechanical	54	75	0·01	ELE (U. of Nottingham)
Multi-wire pointer and scale	10	Mechanical	54	100	0·5	ELE (U. of Nottingham)
Multi-wire electric transducer	8	Mechanical	56	15	0·025	Terrametrics
	6	Grouted	50–100	50	0·05	Slope Indicator
Single rod (fixed), dial micrometer	1	Mechanical Grouted or hydraulic	42 25	50 20	0·025 0·1/0·1	Slope Indicator Interfels
Single rod linear potentiometer	1	Mechanical	50	25/75/150	0·025/0·075/0·15	Slope Indicator
Single rod (probe), multiple magnetic anchor	Indefinite	Mechanical Magnetic	100	Infinite	1 (scale readout) 0·02 (micrometer)	ELE Soil Instruments
Induction probe	Indefinite	Hydraulic piates	400	Infinite	?1 mm	Soil Instruments
Idel sonde radio frequency transmitter probe	Indefinite	Grouted steel rings or plates	110	Infinite	1	Maihak
Multi rod dial micrometer	8	Grouted	60	20	0·1/0·01	Interfels
Single or multi rod vibrating wire	6	Mechanical or grouted	50–100	100	0·003	Maihak

Subsurface movements have been grouped into six different types (Wilson, 1962):

a. Elastic deformation.

b. Creep.

c. Large-scale earth deformation.

d. Abrupt shear.

e. Uniform shear.

f. Yielding of structures (not applicable to highway embankments except at bridge structures).

Many types of inclinometers are commercially available (Tables 7a and 7b); however, the most commonly used inclinometers are the probe type. This type consists of a control box and a probe which is lowered into the casing on a cable. In some probes a cantilevered pendulum with resistance strain gages, vibrating wire, or inductive transducers is used to measure cantilever deflection (Franklin and Denton, 1973). Other probes use the Wheatstone bridge principle (Slope Indicator Model 200B), the servo accelerometer principle (Slope Indicator Digitilt), or a differential transformer (Dames and Moore EDR). The probe generally requires a special casing as shown in Table 7b. The electrical output from the probe is measured at the control box and converted to visual display, punched tape, or graphic form.

Inclinometer casing should be installed in a near-vertical hole that intersects the failure zone.* The hole should extend beyond the zone of expected movement and at least 15 ft into soil or rock in which no movement is anticipated. Allowance should be made for loss of the bottom 5 ft of the hole where sediment accumulation occurs. Casings over 50 ft deep should be checked for twist using commercially available equipment since some of the casings may be received from the manufacturer with a built-in twist which would cause considerable errors in observations (USAE, OCE, 1971b).

Normally when inclinometer casings are placed in embankments

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* Measurements in nonvertical holes can be made with some inclinometers; however, before planning such holes the engineer should be aware of the limitations of the particular inclinometer being used (Table 7b).

Table 7a. Fixed-position inclinometers (Franklin and Denton, 1973).

Type	Trade name	Drillhole diameter (mm)	Maximum No. and type of anchors	Range		Sensitivity		Manufacturer
				mm/m	mins	mm/m	secs	
Anchored chain of rods with transducers at pivots	Lateral deformation indicator/chain deflectometer	116–146 (cased)	Not determined	±10	35′	0·1–0·01	20–2	Eastman Interfels
Pivoted rod and proximity transducer	Multiple position deflectometer	75–100	Not determined	±12	40′	0·03	6	Terrametrics
Flexible steel strip with strain gauges in parallel to monitor	Strip gauge	75 or larger as required	Continuous	60-mm radius subject to metal thickness	5400	6·0	1200	Savage (not yet marketed)
Tiltmeter incorporating pendulum and vibrating wire measurement for mounting on retaining walls etc. or rods in drill hole	MDS 81 MDS 81B MDS 82B		As required	±3 ±6 ±12	10′ 20′ 40′	0·01–0·002 0·025–0·004 0·050–0·007	2–0·3 5–0·7 10–1·4	Maihak
Flexible breakable strip with resistors in series to detect depth of movement horizon	Shear strips	76 or larger	60-m strip lengths in series	Shear detection movement only		2–50 mm		Terrametrics

Table 7b. Probe inclinometers as given in EM 1110-2-1908 (USAE, OCE, 1971b) and Franklin and Denton, 1973.

Type	Trade Name	Approximate Casing Size mm	Casing Type	Range mm/m	Range deg	Sensitivity mm/m	Sensitivity sec	Manufacturer
Strain-gaged pendulum	CRL Inclinometer	45 x 45	Square aluminum duct	±88 from vertical	±5	0.075	15	Cementation Research
	Inclinometer	50	Aluminium tubing with keyways	360 from vertical	±20	0.2	36	Soil Instruments
	Borehole clinometer	76 x 76	Square steel tube	±175 from vertical	±10	0.1	20	Structural Behavior Eng. Lab.
	C-350 slope meter	45 x 45	Square steel tube	±577 from vertical	±30	0.075	15	Soiltest
Pendulum with rheostat	Series 200-B slope indicator	81	Aluminum tubing	±467 ±87 from vertical	±25 ±5	1.0	180	Slope Indicator
2 electrolevels at 90 deg, servomotor and compass	Slope reader	51	Plastic	±175 from vertical	±10	0.1	20	Eastman
Servo accelerometers	Digitilt	30/70/81	Aluminium/ plastic tube	±577 infinite	±30 ±90	0.1	18	Slope Indicator
Pendulum with vibrating wire, 2 direction, compass or keyway	MDS 83	50 or larger	Aluminium or plastic, keyways optional	±290	±15	0.05	10	Maihak
Pendulum with vibrating wire	68-062 inclinometer	50	Aluminium alloy	±792	±45	0.15	30	ELE/Geonor
Pendulum with differential transformer, automatic recorder	Earth deformation recorder (EDR)	89	Plastic with grooves			0.3% for angles up to 4 deg; 0.15% for angles up to 8 deg		Dames & Moore
Pendulum with vibrating wire	MPF clinometer				15	1		Telemac

having large boulders or rock fragments, it is recommended that the casings be grouted in or carefully backfilled with various combinations of gravel and sand. However, Law Engineering Testing Company evaluated several fills in Tennessee and believes that grout tends to restrict the movements to a small section and thus may show a thin shear plane when actual deformations are occurring over a much larger depth interval. Their experience showed that when grout was used the hole was often blocked earlier (by displacement along a small depth interval) than if sand backfill were used. Inclinometer casings backfilled with sand or gravel were believed to aid in providing data that reflected the conditions more accurately and the casings stayed open longer. If grout is to be used as a backfill for inclinometer casings, weak grout mixtures are recommended. Strong grouts are not recommended as they are likely to produce premature blockage of the casing. The disadvantage of sand or gravel backfill is that several weeks may be required for the backfill to stabilize. During such time, readings may be misleading.

In cases when the embankment distress has not proceeded to a point at which the lateral extent of the failure can be determined or when no indications other than settlement are apparent, inclinometer casings should be located at intervals along the suspect embankment. In one instance, eight different inclinometer wells were installed to investigate a problem embankment on U. S. Highway 119 in Kentucky (Hopkins, 1972). Installation of inclinometer casings on the slope is desirable but not always practical. A more complete description of the failure surface is obtained with each inclinometer casing that penetrates the surface. Figure 20 illustrates the use of inclinometer installations along a slope to delineate the failure plane for various types of failure.

The frequency of observations depends on several factors, the most important of which is the rate of movement. It is necessary to read inclinometers frequently just after installation and, based on these observations, to adjust the interval of observations. Observations of piezometers, settlement devices, and other instrumentation should be coordinated closely with inclinometer observations.

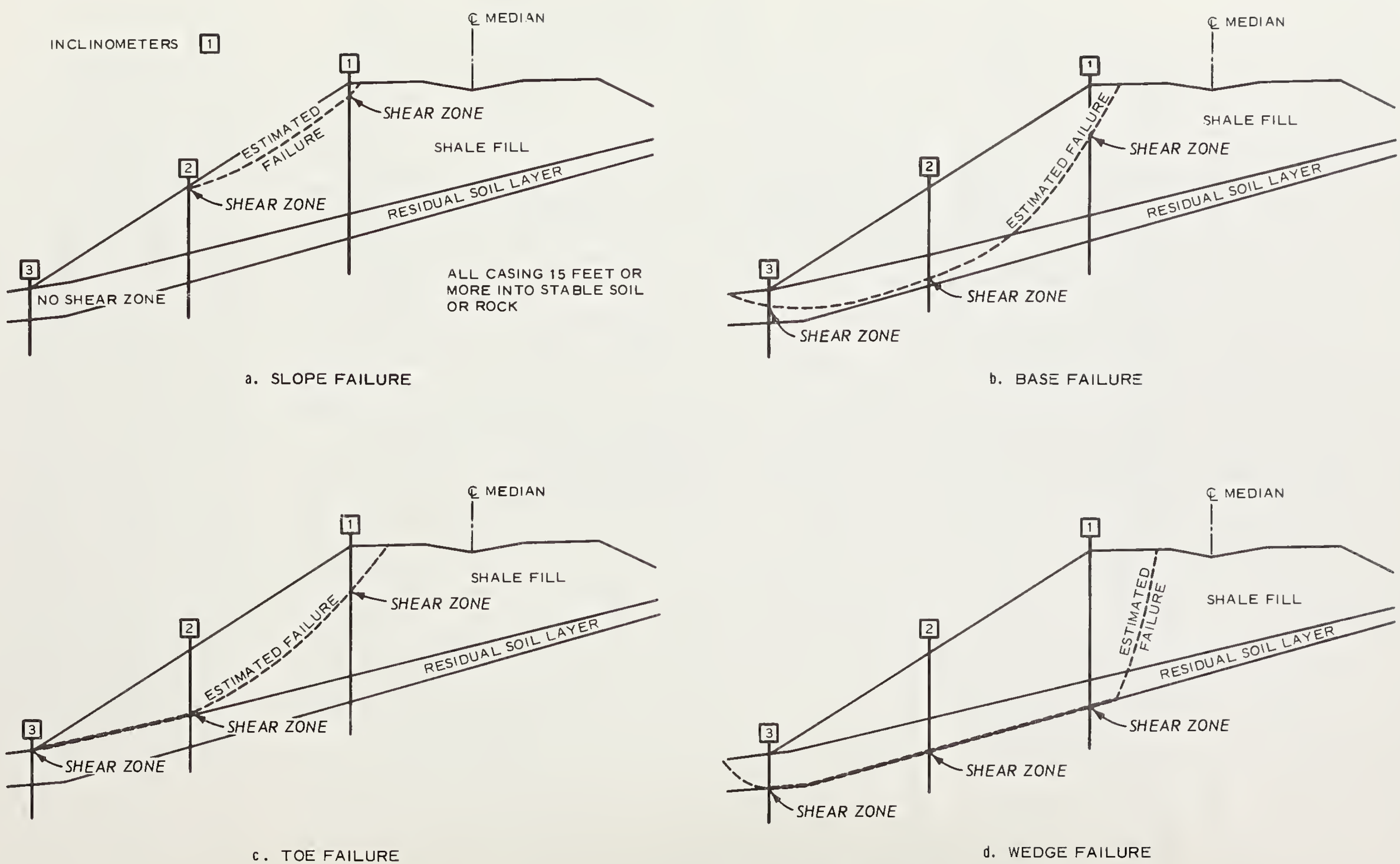

Figure 20. Estimation of failure surface from slope inclinometer data.

The primary purpose of gathering inclinometer data is to delineate the zone of movement. This zone of movement can then be used in analysis of sliding stability of the embankment with failure surfaces estimated as in Figure 20. Movement may also indicate consolidation of the foundation (Figure 21). Movement from consolidation would necessitate further analysis of the compressibility of the foundation to determine the magnitude of the stresses induced in the embankment due to consolidation. Stresses could simply cause displacements or they could be great enough to exceed the strength of the fill in shear.

The plotting of movement data in a manner consistent with piezometer, rainfall, and stream flow data is again recommended. Such a plot might indicate a trend with movements occurring at a greater rate following periods of heavy rainfall or at periods of high stream flows. The importance of such a plot is illustrated by Figure 22, which compares rainfall to the incidence of landslides in the Orinda formation during 1962 (Duncan, 1971). It is clear from the figure that in this area the occurrence of landslides is greater after periods of heavy rainfall. Duncan recognizes that these failures may be the result of a weakening of the soil over a period of time and that the rainfall may be the "straw that broke the camel's back." The same concept applies to the deterioration of compacted shale embankments.

Several types of plots can be made from inclinometer data. Movement versus time plots are used to show a rate of movement (Figure 23). It should be noted in this figure that the movement is also accelerated by rainfall. This relationship again indicates the importance of the comparison of inclinometer data with other available information such as rainfall records. A plot of movement versus depth enables the determination of the zone in which movement is occurring (Figure 24). A method of comparing shear strain observed in the failure zone with that observed in laboratory triaxial tests has been proposed (Department of the Navy, 1962). An extension of this idea that allows a factor of safety to be estimated by comparison of shear strains from inclinometer data with strains from laboratory shear tests is presented in Appendix C; however, this method gives a local safety factor and should be used with

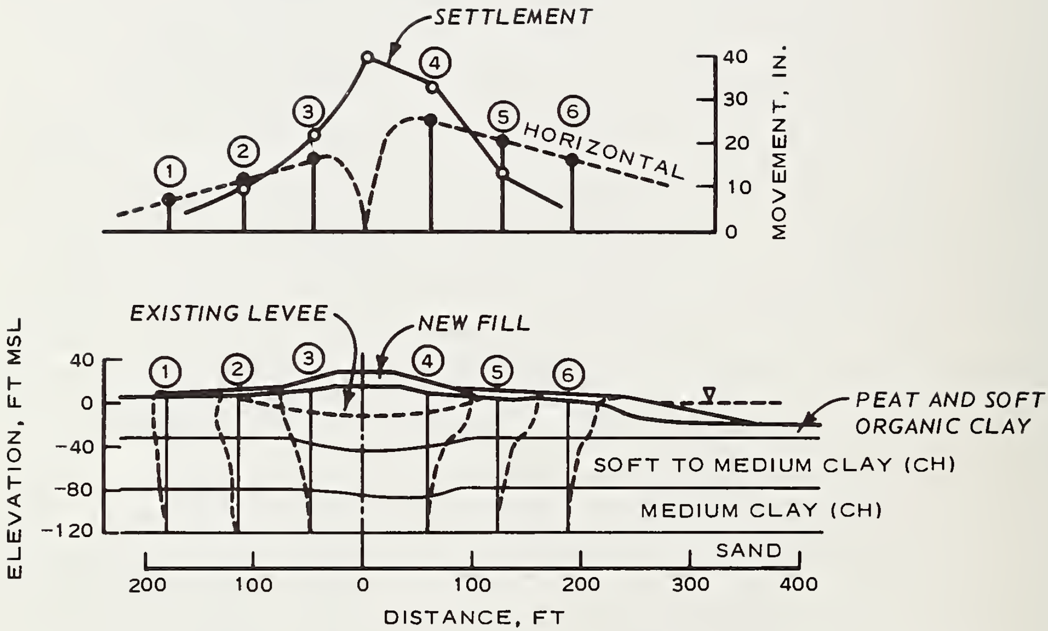

Figure 21. Foundation movement indicated by slope inclinometer data, Atchafalaya Levee, Louisiana (Wilson, 1970).

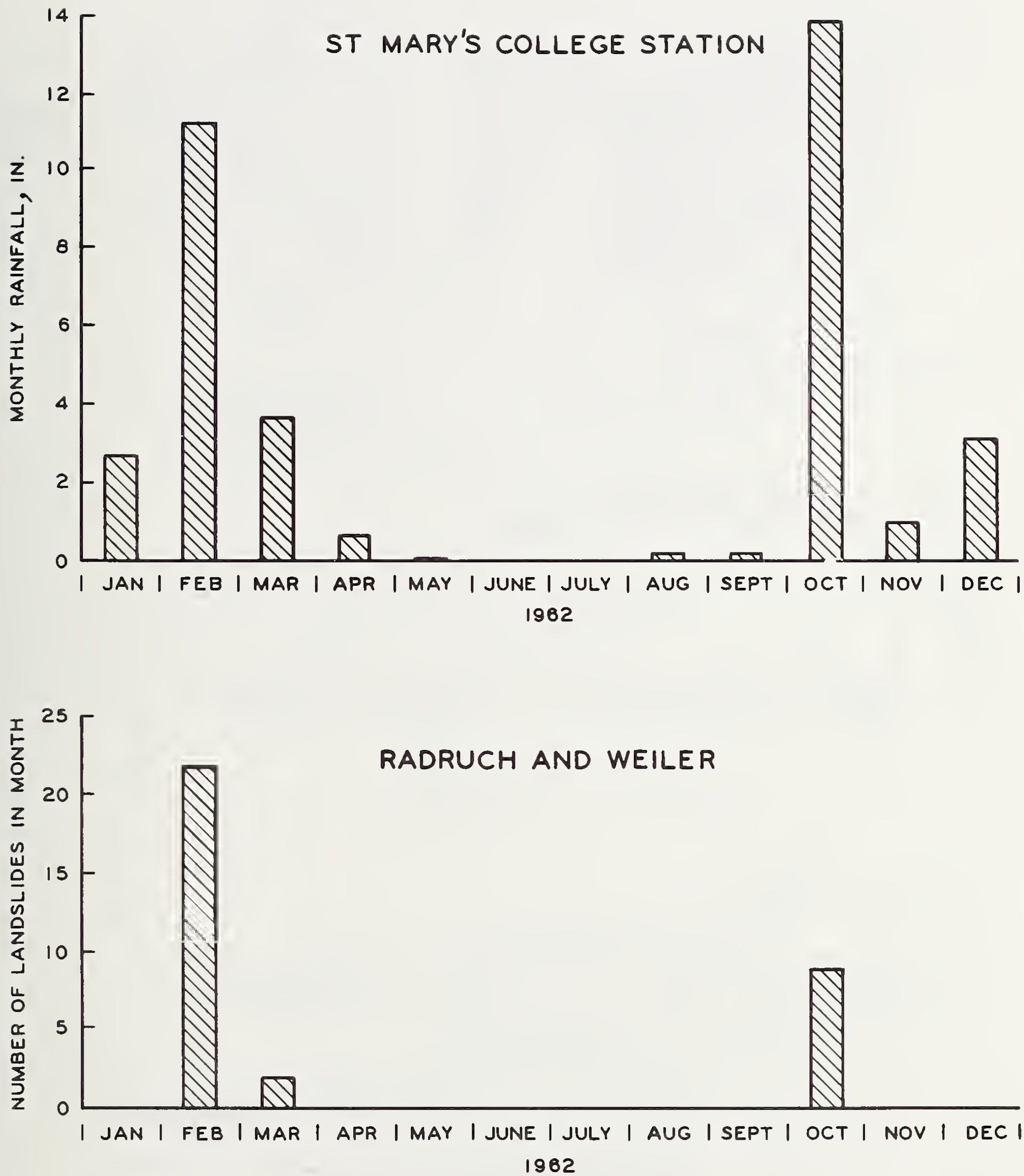

Figure 22. Rainfall and landslides in a portion of the Orinda formation (Duncan, 1971).

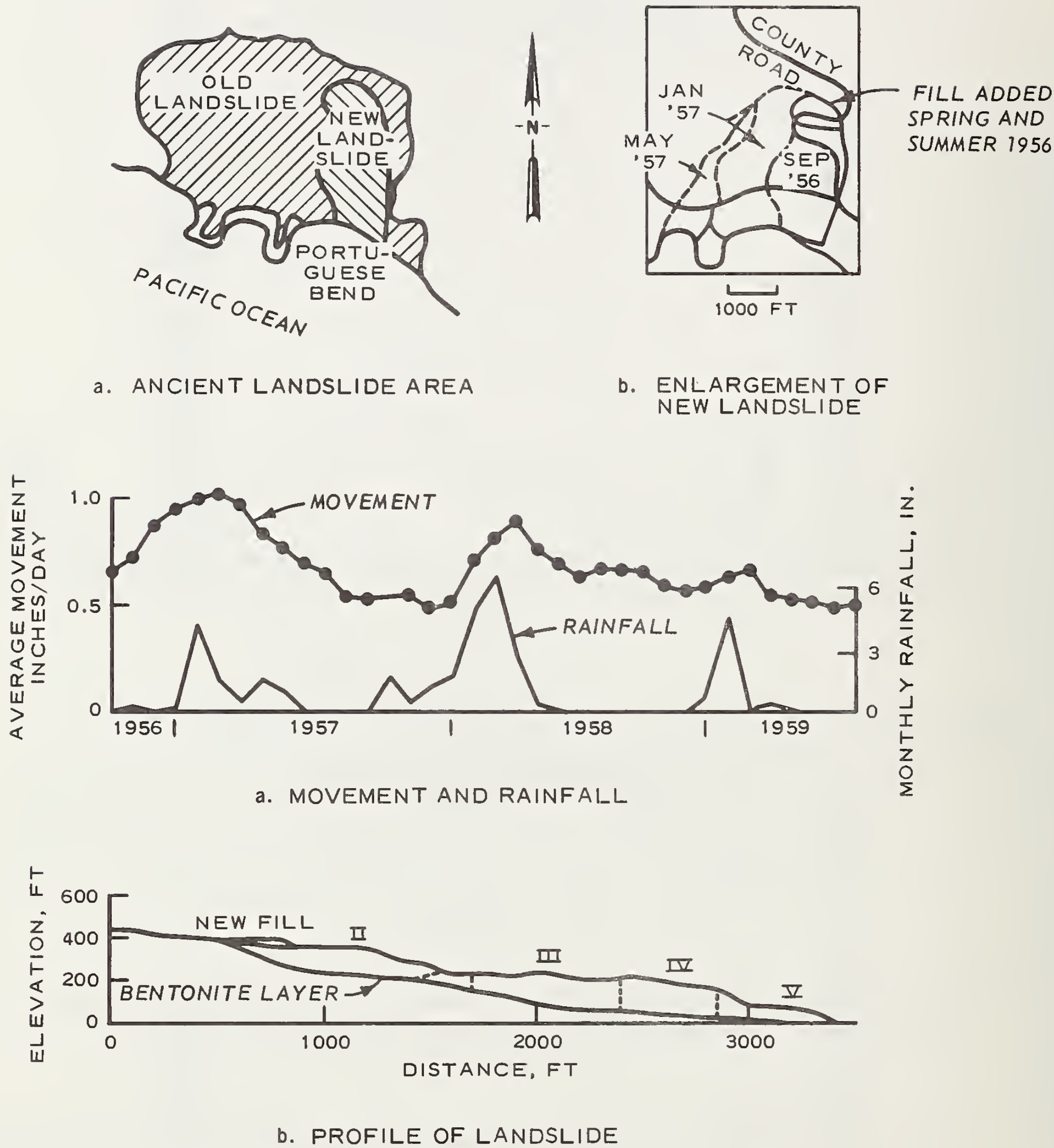

Figure 23. Movements at Portuguese Bend Landslide, California (Wilson, 1970).

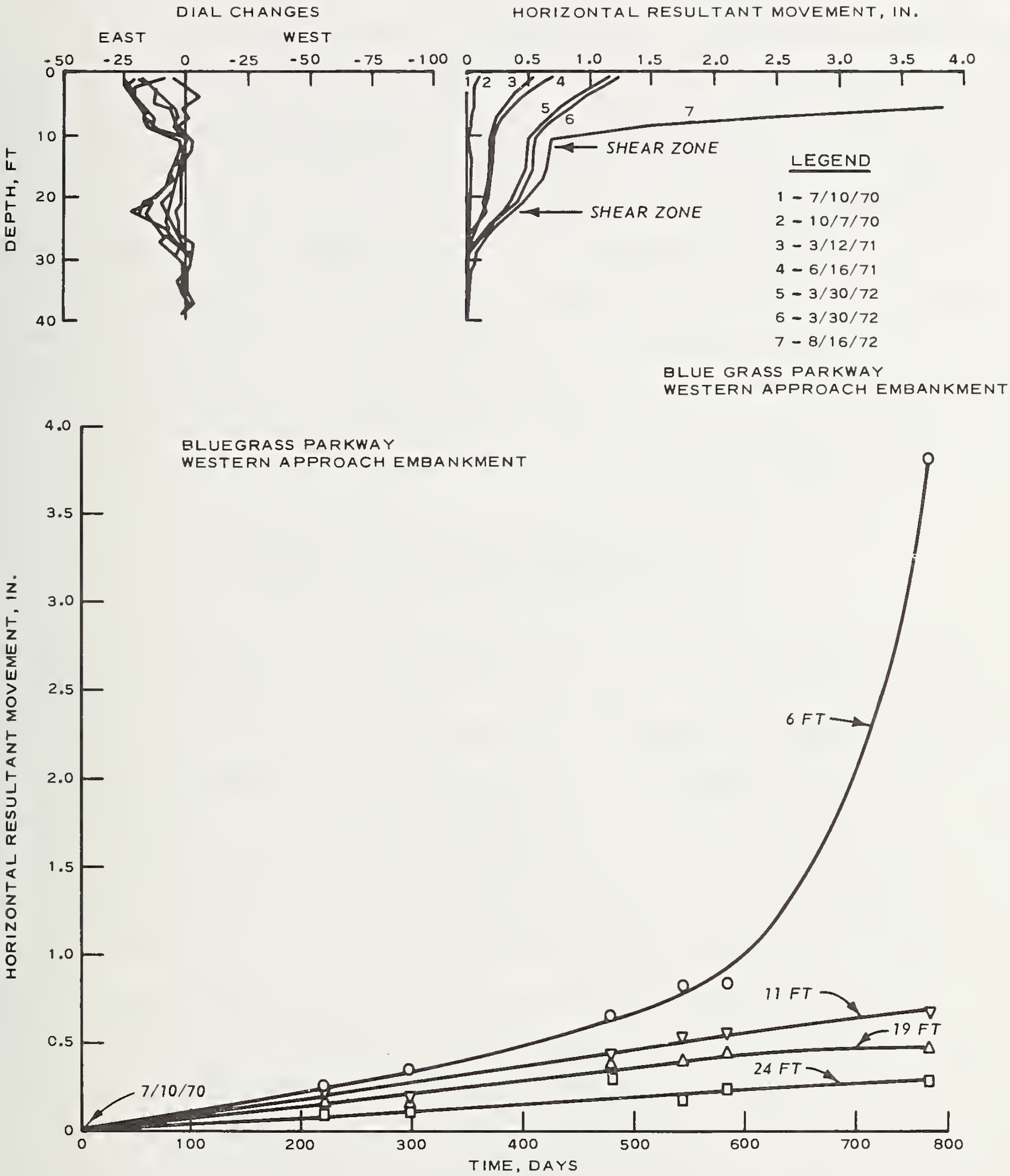

Figure 24. Slope inclinometer results, western approach embankment, Chaplin River Bridges, Bluegrass Parkway, Kentucky (Hopkins, 1973).

caution. The preferred evaluation should include a complete analysis.

A simple and inexpensive means of determining an approximate radius of curvature of a deflected casing and the depth at which the curvature occurs is the poor boy. A short length of pipe (2 to 3 ft) with an attached wire is lowered to the bottom of the casing and remains there until an observation is needed. At that time, the pipe is raised until the curvature of the casing prevents further upward movement. The depth is then measured and recorded as the lower limit of the movement zone. A longer pipe, normally about 10 ft, is then lowered down the hole until movement is prohibited by the bend. Shorter lengths of pipe are then used to determine the longest pipe which just passes through the bend; this length is taken as the chord of the bend. The poor boy method is recommended for use in inclinometer casings between regular readings (USAE, OCE, 1971b).

Sampling and Laboratory Testing

The fill characteristics will largely determine the extent to which sampling and testing will be successful. Investigations have shown that placement of shale in embankments may vary from thick layers of dumped rock pieces to thin layers of well compacted soil (Shamburger et al., 1975). When large particles of hard shale, sandstone, or limestone are present in a fill, conventional undisturbed tube sampling methods are rendered ineffective or entirely useless. Good core samples using double-tube core barrel samplers are also difficult to obtain (Golder, 1971). However, fills placed as soil in thin well compacted layers can be sampled successfully with these methods. Methods of sampling have been presented by Mohr (1936), Hvorslev (1948), Goode (1950), Cambefort (1955), Lowe (1960), and USAE, OCE (1972), and, consequently, will not be discussed here. The following discussion will be limited to the types of samplers and testing methods available. Examples are given from experience among the States surveyed in the study.

Undisturbed sampling. When an embankment exhibits signs of distress, borings are made and undisturbed samples are often taken to determine the strength and compressibility of the fill material and to evaluate the existing stability. Although no sample is truly undisturbed (Hvorslev, 1948), proper sampling techniques will enable samples to be recovered with a minimum of disturbance. Undisturbed samples are a primary means for direct determination of material properties.

Most compacted shale embankments contain at least some rock pieces. In spite of this, both Tennessee and Kentucky have been successful in obtaining undisturbed samples using thin-walled sampling tubes (ASTM D 1587-67 and AASHTO T 207-74). In evaluating a fill on I-75 at sta 840+00 (see Figure 2), Law Engineering Testing Company (1972)* obtained samples using this method and conducted unconsolidated-undrained triaxial compression tests in the laboratory. These tests indicated a wide range of shear strength apparently due to the presence of rock pieces. Kentucky used thin-walled tube sampling methods on numerous problem fills and was successful in obtaining undisturbed samples on many fills investigated. In several instances, however, the rocky nature of the fill either prevented undisturbed samples from being obtained or severely limited the number which could be taken (Hopkins, 1972; Hopkins, 1973; and Hopkins et al., 1973). Consolidated-undrained tests with pore pressure measurements were normally conducted on undisturbed samples obtained by investigators in Kentucky. Indiana also used thin-tube sampling to investigate SLIDE-2 on I-75. Unconsolidated-undrained triaxial and unconfined compression tests were conducted. However, tests on embankment materials were not considered reliable due to "the variable composition and condition of the material" (Indiana State Highway Commission, 1972).** Shear strengths determined from laboratory tests were not used in the analysis for corrective measures.

When compacted shale embankments consist of soillike materials with few rock pieces, good undisturbed samples may be obtained using 3-in.-diam thin-walled, fixed-piston samplers pushed hydraulically with

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* See footnote on page 10.

** Indiana State Highway Commission (1972), "Report of Slide Investigation: Sideslopes Failure of Westbound Lane on I-74 between Stations 519 and 530, Dearborne County, Indiana," EHS-Project 74-4 (72) 169 P. E., Indianapolis, Jul.

one continuous drive for the length of the sample to be obtained (usually 30 in.). Dynamic driving of samplers disturbs the sample and should not be used (USAE, OCE, 1972). A list of various types of samplers and the materials in which they can be used is shown in Table 8. The type of sampler chosen for a soillike embankment will depend on the stiffness of the material; however, the fixed-piston sampler (Figure 25), compared with the other thin-tube samplers listed, has been used with success for a wider range of materials.

Embankments containing large limestone or shale pieces require the use of double-tube core barrel samplers to obtain good quality undisturbed samples. Double-tube core barrels have internal or bottom discharge bits set with tungsten carbide teeth at 20- or 30-deg angles with respect to the radius. This set causes a slicing action forcing the cuttings and drilling fluid away from the core. Core barrels cut a kerf around a core by a rotary drilling process. The core barrel consists of a head assembly, inner and outer tubes, outer tube extension, and a bottom assembly that consists of inner tube extension, core lifter, and bit. Standard sizes* for double-tube core barrels are 2-3/4 by 3-7/8 in., 4 by 5-1/2 in., and 6 by 7-3/4 in. (USAE, OCE, 1972). Common double-tube core barrel samplers are also listed in Table 8. Figure 26 shows a WES sampler.

In addition to the selection of the proper type sampler, other considerations are necessary to obtain good results from undisturbed sampling programs. A prime factor is the experience of the driller. An experienced driller can be the difference between obtaining good quality undisturbed samples and poor samples with a low recovery rate. Procedures for undisturbed sampling must also be considered. The diameter of the borehole, the method of advancing the borehole between samples, the manner in which the borehole is stabilized, and the procedure used to clean the hole are factors which must be decided prior to sampling since they affect the quality of samples obtained. After sampling is accomplished, such factors as handling, extraction,

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* Inside and outside diameters, respectively.

Table 8. Selected types of undisturbed samplers.

Sampler	Nominal Tube Size	Used in Sampling	Remarks
		Thin-Wall Tube Samplers	
Fixed piston	2-, 3-, and 5-in. diameter, 3- and 4-1/2-ft lengths	Very soft to stiff clays and sands	Above or below water table
Hydraulic piston	2-, 3-, and 5-in. diameter, 3- and 4-1/2-ft lengths	Soft to medium clays and silts	Generally requires stiff, large diameter drill rods for deep holes
Free piston or semifixed piston	2-, 3-, and 5-in. diameter, 3- and 4-1/2-ft lengths	Stiff clays or partially dry silts and clays	
Swedish Foil	2-5/8-in. diameter, 8.2-ft-long barrels may be coupled to any length	Soft cohesive soils	Long continuous samples possible; rate of penetration relatively unimportant
Open tube	2-, 3-, and 5-in. diameter, 3- and 4-1/2-ft lengths	Friable partially dry silts and clays	Not recommended for use in boreholes
		Double-Tube Core Barrel Samplers	
Denison	4- and 5-in. diameter, 2-ft lengths	Coarse sands, gravelly soils and formations too hard to sample with push tube type samplers	Sample core is caught and stored in light metal liner
Pitcher	3-, 4-, and 6-in. diameter, 3- and 5-ft lengths	Same as Denison	Variation of Denison which has a spring-loaded inner sampling tube that telescopes into the outer cutter barrel as the formation hardness increases
WES	5-in. diameter, 2.5-ft lengths	Same as Denison	Modified Denison; not commercially available

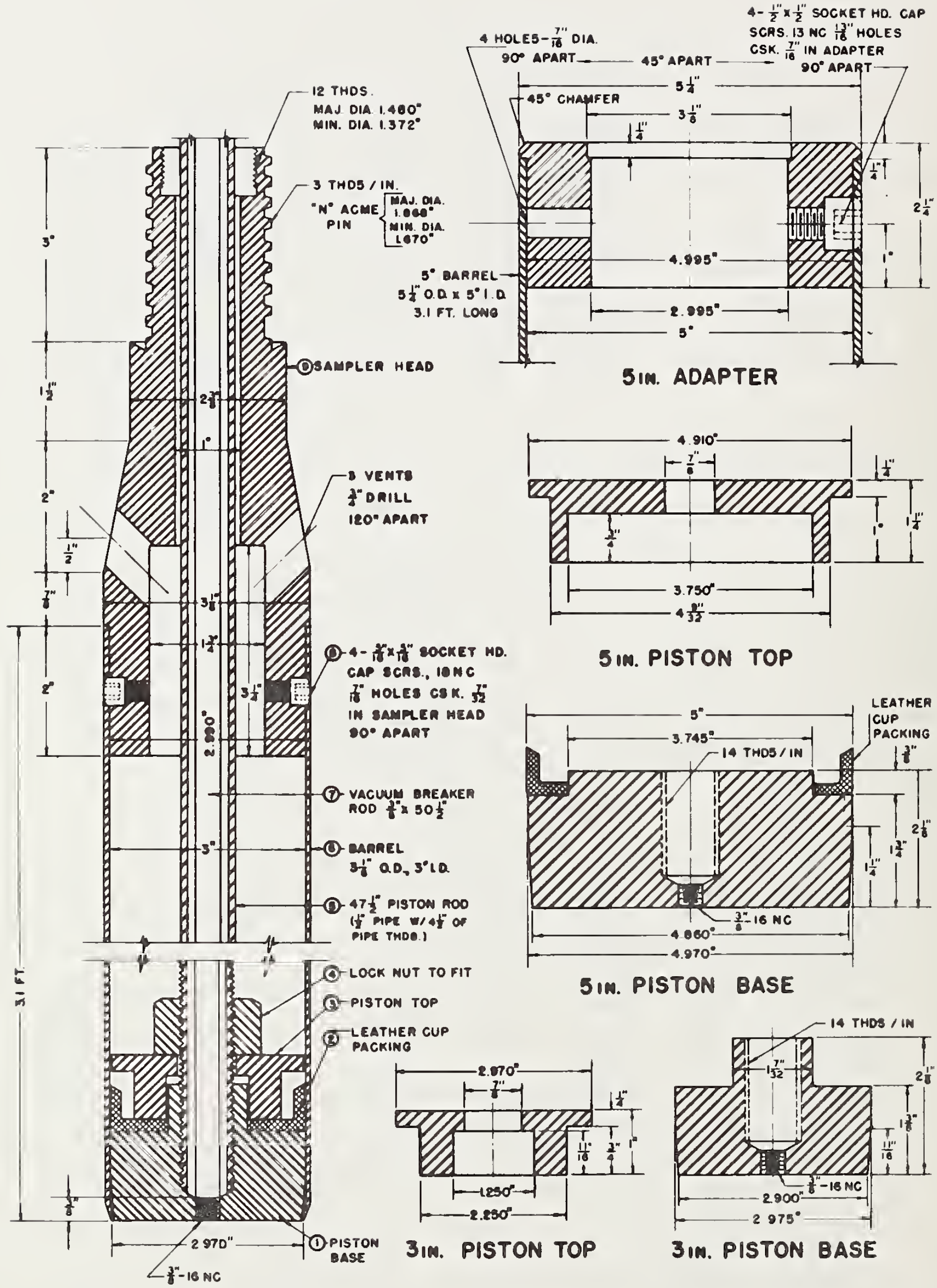


Figure 25. Hvorslev type 3-in.-diam fixed-piston sampler with 5-in. adaptor (USAE, OCE, 1972).

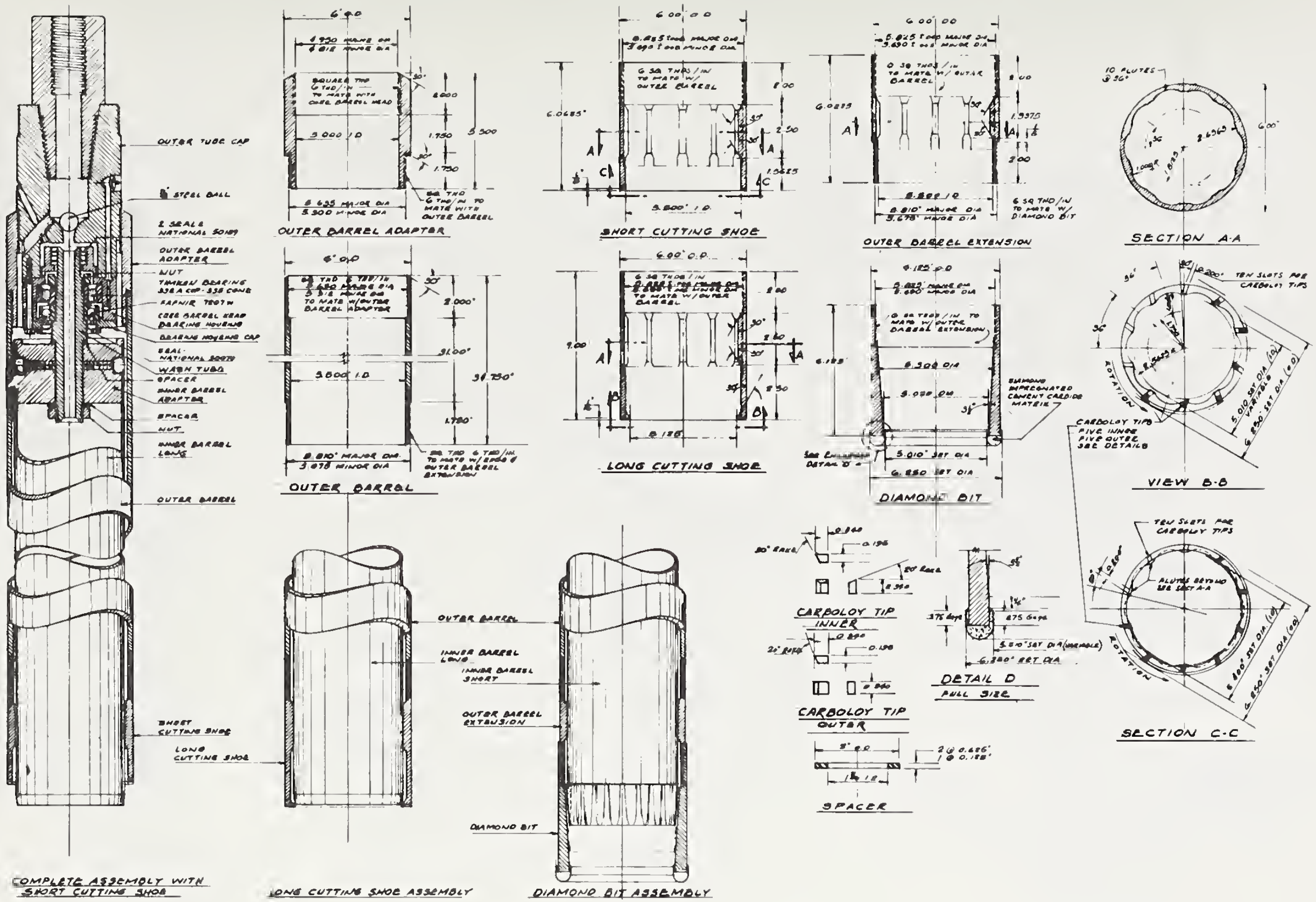


Figure 26. DCDMA standard 4- by 5-1/2-in. core barrel with WES conversion to 5- by 6-1/4-in. soil and rock sampler (USAE, OCE, 1972).

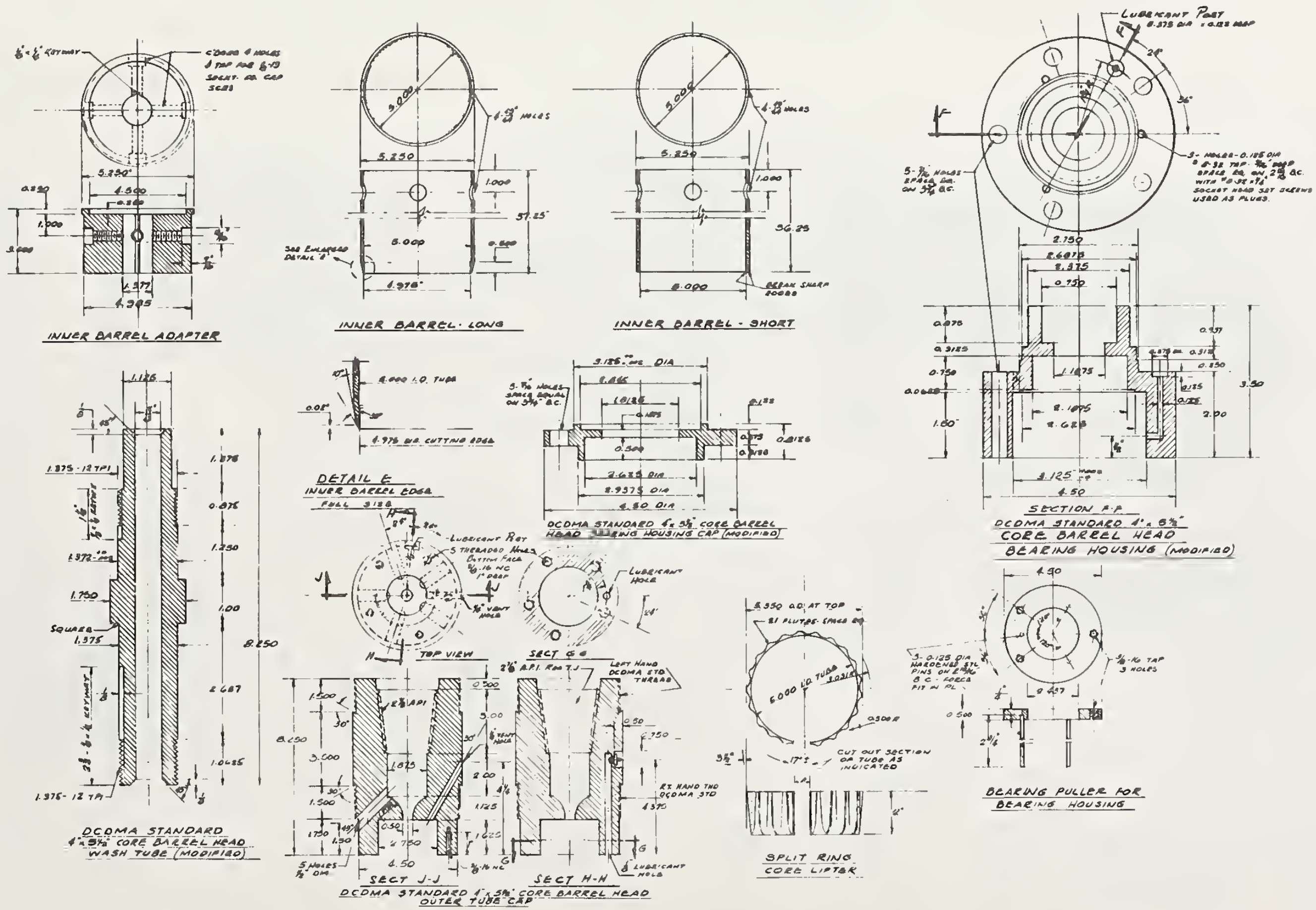


Figure 26. DCDMA standard 4- by 5-1/2-in. core barrel with WES conversion to 5- by 6-1/4-n. soil and rock sampler (USAE, OCE, 1972). (Concluded.)

preservation, and shipment also affect the quality of the sample. For example, a sample having low cohesion may be difficult or impossible to extract without disturbance. Experience has shown that freezing is often necessary for both extraction and storage of a sample with low cohesion (Strohm et al., 1964; and USAE, OCE, 1972). Therefore, when undisturbed sampling is conducted, the engineer must assure himself that the methods, procedures, and equipment used in sampling have resulted in samples which are sufficiently undisturbed and representative.

Disturbed sampling. Disturbed samples can be obtained from boreholes, test pits, or from the surface. These samples contain all the constituents of a particular stratum, but the structure of the stratum has been altered. Various types of augers and drive samplers are suitable for obtaining disturbed samples. Types of augers include hand, barrel, helical, continuous-flight, and bucket. Types of drive samplers include open-drive, displacement, cable-tool, split-spoon, and free-piston.

When samples are obtained using a split-spoon sampler, data are available for relating standard penetration resistances to the strength properties of the materials (Table 9). The correlations shown in Table 9 can be used as a rough guide only for soils that do not contain gravel- or cobble-size material which would cause erroneously high blow counts. Other correlations with strength have been made with Atterberg limits; however, the applicability of correlations of limits to actual strength properties of a compacted shale embankment must be considered as doubtful. Disturbed sampling usually provides the engineer with little more than an impression of the range of properties that an embankment may have.

Laboratory testing. Laboratory testing in regard to design studies is related to the type conditions expected for the embankment to be designed. Tests conducted on undisturbed samples can include unit weight determination, permeability, consolidation, triaxial compression, unconfined compression, and direct shear. Disturbed samples can be used to determine water contents, Atterberg limits, specific gravity, grain size, and compaction characteristics. In addition, samples can be

Table 9. Density, standard penetration, and unconfined compressive strength as given in EM 1110-2-1907 (USAE, OCE, 1972).

Basic Soil Type	Density or Consistency	Range of Standard Penetration Resistance(1)	Range of Unconfined Compressive Strength (q_u)(2)
Cohesionless	Very loose	Less than 4 per foot	Not applicable
	Loose	4 to 10	Not applicable
	Medium dense	10 to 30	Not applicable
	Dense	30 to 50	Not applicable
	Very dense	Greater than 50	Not applicable
Cohesive	Very soft	Less than 2 per foot	Less than 0.25 ton/sq ft
	Soft	2 to 4	0.25 to 0.5
	Medium stiff	4 to 8	0.5 to 1.0
	Stiff	8 to 15	1.0 to 2.0
	Very stiff	15 to 30	2.0 to 4.0
	Hard	Greater than 30	Greater than 4.0

(1) Number of blows from 140-lb weight falling 30 in. to drive 2-in. OD, 1-3/8-in. ID, sampler.

(2) q_u may also be approximated using a pocket penetrometer or Torvane shear apparatus.

remolded to obtain permeability and strengths at particular remolded densities using grain-size distributions found in the fill. Test procedures are contained in EM 1110-2-1906 (USAE, OCE, 1970a). Use of laboratory data in conjunction with analysis and prediction of the embankment stability will be discussed in a later section of this report.

In Situ Investigations

In an evaluation of a problem embankment, an assessment of the strength and compressibility must be made before the embankment stability can be determined. Strength properties can be determined from laboratory tests on undisturbed samples, as previously discussed, or through in situ investigations. In situ investigations can either provide strength properties directly through field testing or merely permit a detailed examination of the fill which aids in the selection of strength properties. Selected methods of investigating fills, their applicability to compacted shale embankments, and their use by the States as determined during this study will be discussed in the following paragraphs.

Menard pressuremeter. In situ properties of a material can be computed from deformations accompanying the expansion of a cylindrical probe in a borehole. A device developed by L. F. Menard is currently being used for testing a variety of materials from cohesive and cohesionless soils to rocks. Figure 27 is a schematic of a pressuremeter probe and a pressure-volumeter (located at the ground surface). Basically the method of collecting data consists of lowering the probe to the desired depth in the borehole, applying pressure to the measuring cell in increments, and recording the volume change in the pressure-volumeter. Volume changes are measured 15, 30, and 60 sec after the application of an incremental pressure. Pressure versus volume at the 60-sec measurement is plotted as the P-V curve. The volume change between the 30- and 60-sec reading plotted for each pressure is the creep curve. The inertia curve is obtained by measuring volume changes under incremental

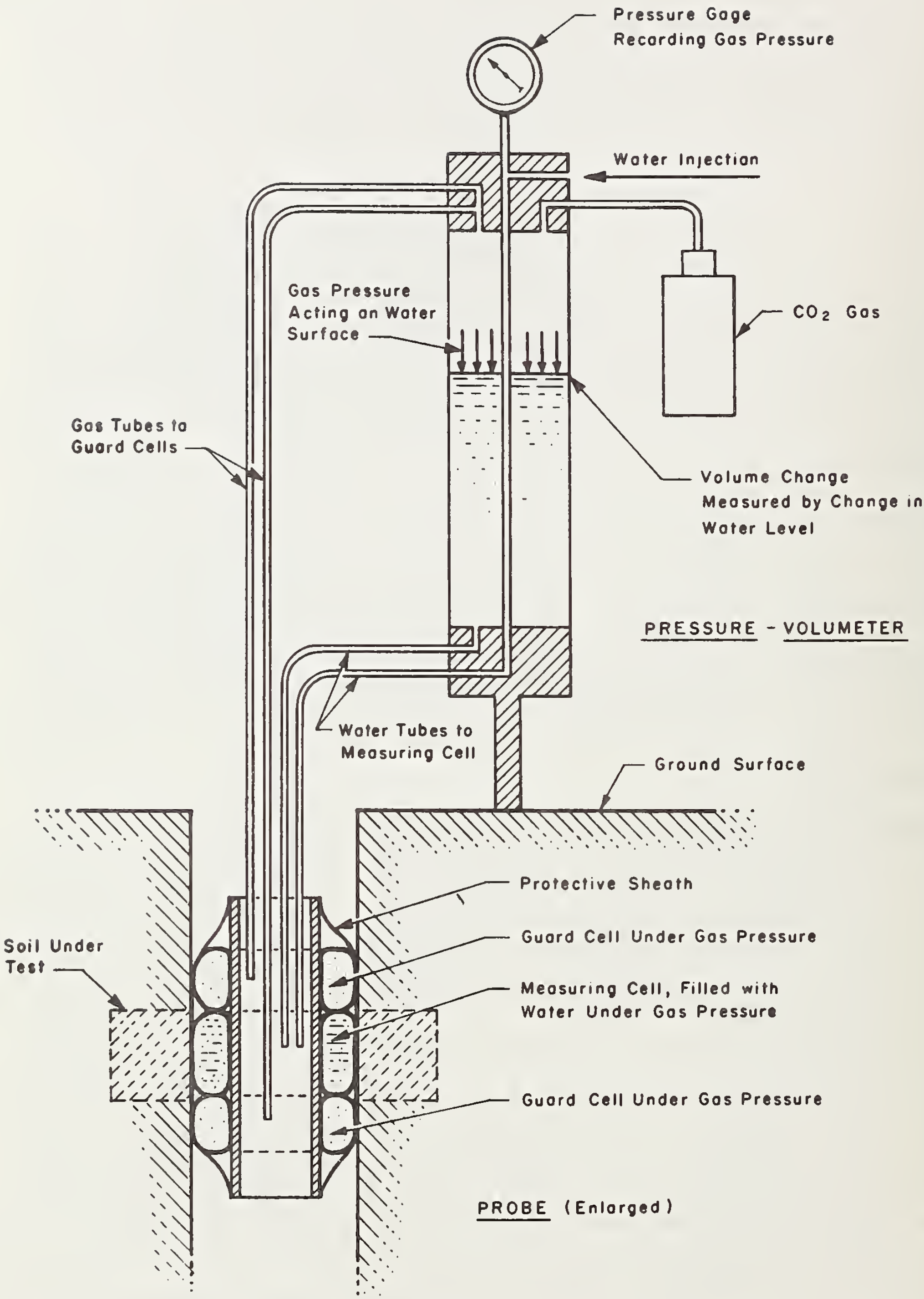


Figure 27. Menard pressuremeter (Campbell and Hudson, 1969).

pressure increases with the probe out of the borehole and is used to correct borehole pressure-volume curves for borehole conditions (Figure 28). From the pseudoelastic portion of the P-V curve an approximation of Young's modulus can be obtained. The angle of internal friction, ϕ , and cohesion, c , can be approximated by failing the soil at different depths (Smith and Smith, 1968; and Trofimenkov, 1973). The limit pressure, P_L , gives a direct indication of the bearing capacity of the material. The methods for calculating these values have been given by Menard and summarized by Hall and Hoskins (1972), Smith and Smith (1968), Campbell and Hudson (1969), Higgins (1968), Dixon (1970), Hendron et al. (1970), and others.

Two of the States contacted in the study had used the Menard pressuremeter for investigations of embankments. Law Engineering Testing Company applied the Menard pressuremeter to in situ testing of three compacted shale embankments on I-75 in Tennessee. Figure 29 shows the plan and section of one of these fills, and Figure 30 shows a comparison of laboratory data, standard penetration resistances, and pressuremeter data. Pressuremeter data were used in combination with laboratory strength data in the evaluation of two of the I-75 fills. The presence of gravels in the third fill precluded extensive use of the pressuremeter and the fill was analyzed using assumed strength values based on experience obtained in testing the other fills.

Under California's "Movement Within Large Fills" project, Smith and Smith (1968) evaluated the Menard pressuremeter in tests conducted on three fills: (a) Liebre Gulch, a 210-ft-high shale fill on I-5 north of Los Angeles; (b) Squaw Creek, a 383-ft-high shale fill on U. S. Highway 101 north of Cummings; (c) Chadd Creek, a 100-ft-high clayey shale embankment south of Eureka. The purposes of the testing varied for each location. At Liebre Gulch, the strength characteristics were desired relative to the performance of the soils around distressed arch culverts. Stress-strain relationships were needed along with a verification of in-place strength of carefully compacted select material at Squaw Creek. At Chadd Creek, a stress-strain modulus was desired for use in analysis of soil stress and its effect on culverts. The

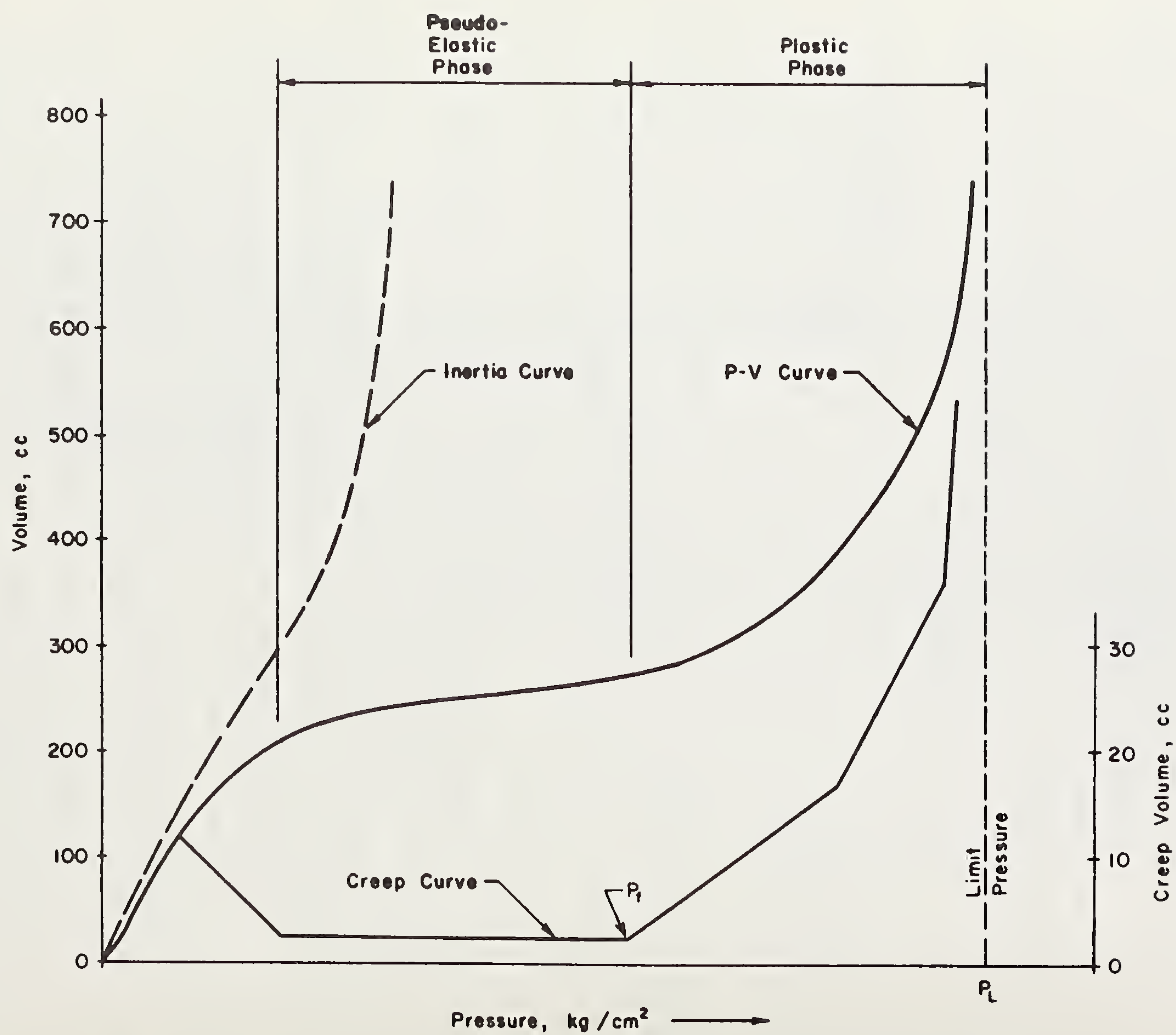


Figure 28. Curves derived from Menard pressuremeter data (Campbell and Hudson, 1969).

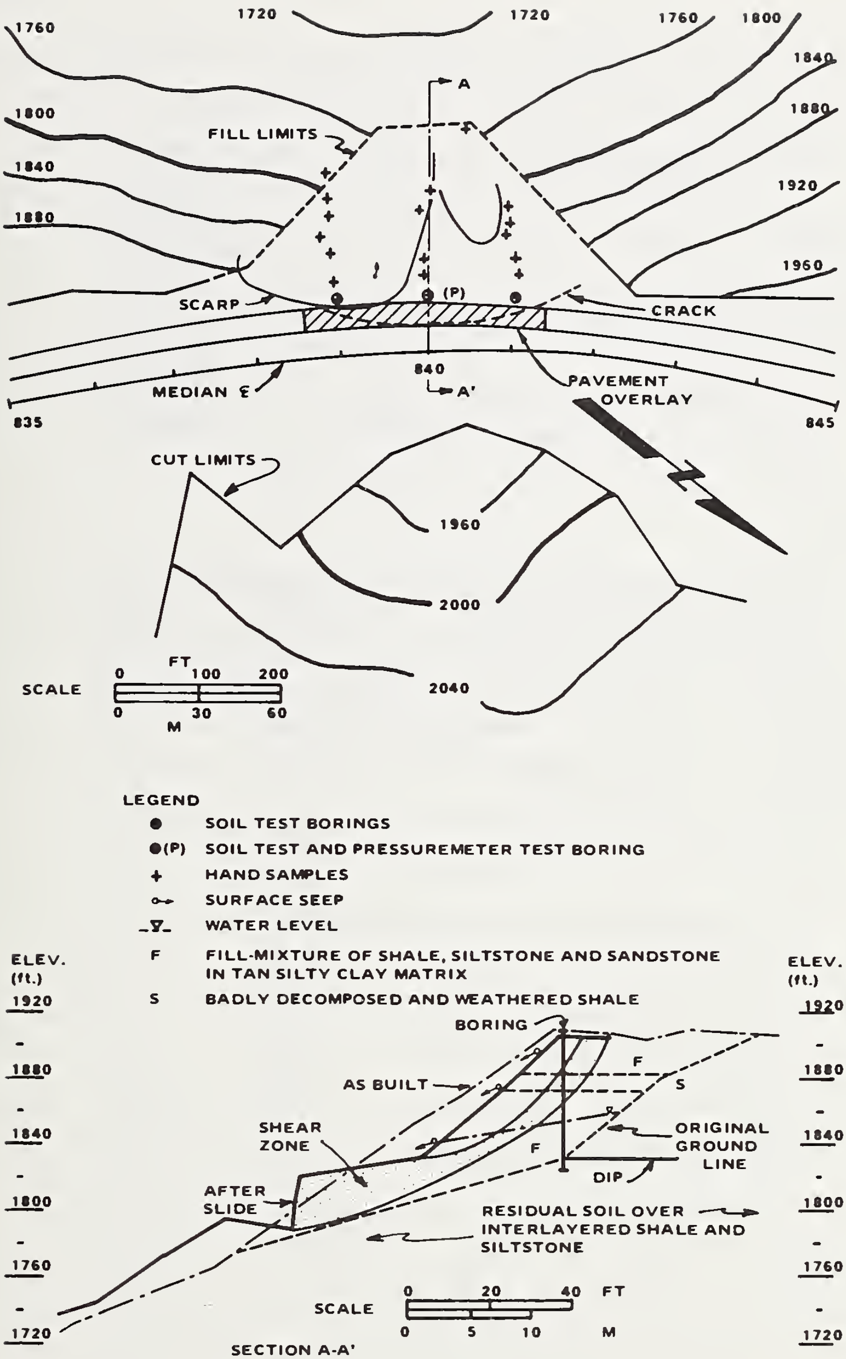


Figure 29. Plan and section of sidehill fill failure at sta 840+00, I-75, Campbell County, Tennessee (Tice and Sams, 1974).

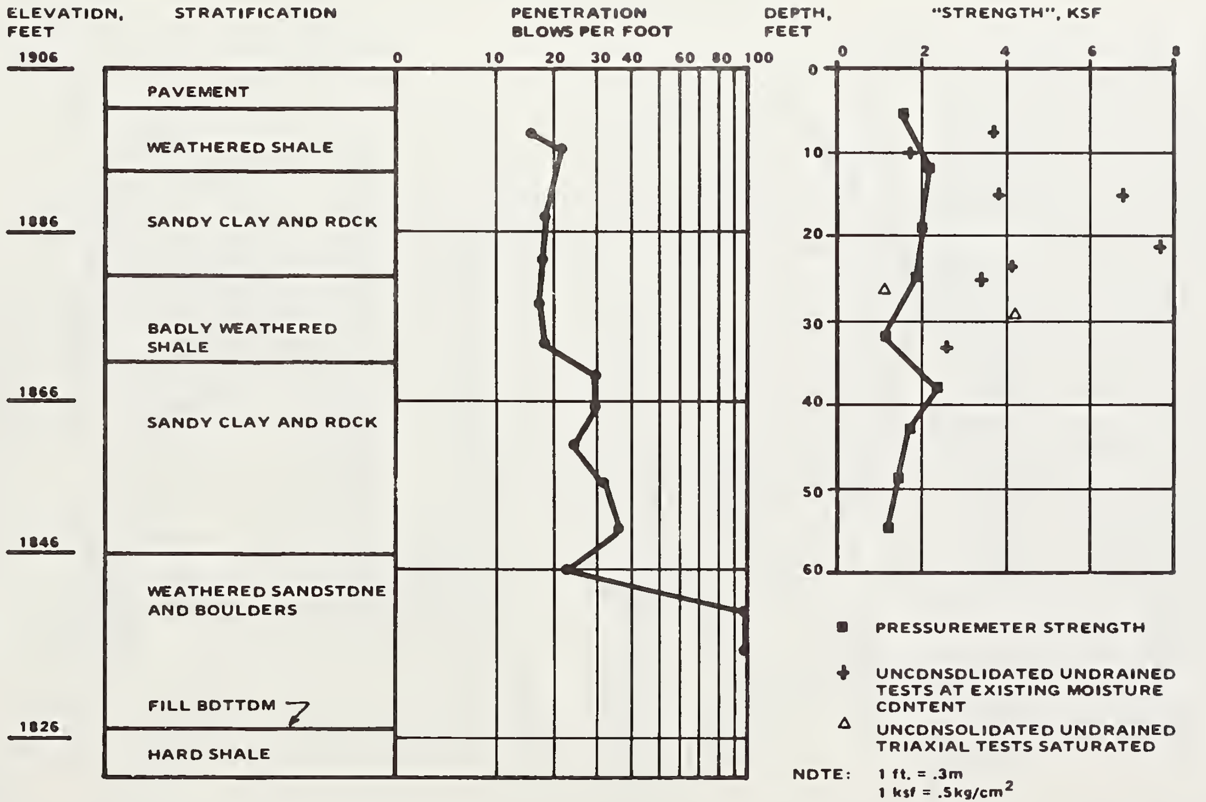


Figure 30. Comparison of laboratory test data with pressuremeter and standard penetration resistances (Tice and Sams, 1974).

pressuremeter accomplished the purposes of the investigations, and further use over an extended period on a variety of materials was recommended.

Problems encountered with the Menard pressuremeter have been caused by a rough or ragged borehole wall puncturing the urethane sheath (for rock) or the rubber sheath (for soil). The pressuremeter is sensitive to the borehole size. If the hole is too large, there may be insufficient water in the pressure-volumeter to determine the limit pressure. If the hole is too small, the soil may be disturbed by the pressuremeter being forced down the borehole. Law Engineering Testing Company drilled a hole 1/8 in. smaller than the pressuremeter probe using a flight auger. The pressuremeter probe with a protective tip was then pushed down the hole and tests were made. Only one sheath was ruptured in the 24 tests made. In the shale fills in Tennessee and California, sharp rocks ruptured the sheath, delaying testing and necessitating replacement of the sheath. In testing in Louisiana, Higgins (1968) determined that in soft soils the inertia curve for the probe approached the P-V curve obtained for the soil, thereby necessitating the use of the probe without the sheath and the use of a softer rubber membrane as the measuring cell. Trouble was also experienced with puncturing the sheath. Testing in sand was also found difficult due to caving of the hole. Pressuremeter tests were conducted in California in the surge chamber of the Castaic Power Plant (Dixon, 1970). Sixty-three tests were conducted in interbedded sandstone and shale at depths ranging from 75 to 400 ft; no problems were reported. Dixon reported additional testing along the route for Stokes Canyon Dam inlet-outlet tunnel located west of Los Angeles. Pressuremeter tests were conducted at 10 individual depths within a 166-ft hole. Marine sediments consisting of interbedded sandstones and clayey siltstones were tested without difficulty. In situ testing of dark gray and black shale and gray shale at a bridge site on I-280 crossing the Mississippi River near Rock Island, Illinois, was conducted at depths of up to 56 ft with no problems (Hendron et al., 1970). Several papers concerning the use of the pressuremeter are contained in Volume I of the preprint of <u>Proceedings of</u>

the American Society of Civil Engineers Specialty Conference on "In Situ Measurement of Soil Properties," 1975. These papers include pressuremeter test data and evaluation in clays, sands, and gravels; a finite element study of the elastic phase of pressuremeter tests; a method of determining volume changes during a pressuremeter test; and papers on evaluation of test results.

One advantage of using the pressuremeter over conventional undisturbed sampling and laboratory testing is its lower cost. In addition, the pressuremeter is portable and easy to operate. Since the test is conducted in situ, theoretically there should be less chance for disturbance of the soil and strengths are available in the field soon after testing.

Pressuremeter testing does not provide an immediate determination of strength properties. Correlations of pressuremeter data with strengths obtained from undisturbed sampling and testing need to be developed for use in compacted shale embankments. Field studies are needed to determine modifications necessary for pressuremeter testing in compacted shale embankments. The need for building a confidence in pressuremeter data from test results obtained in compacted shale embankments will require a combination of undisturbed sampling and laboratory tests (when possible) and pressuremeter testing. However, with time and experience, undisturbed sampling and laboratory testing could be reduced.

Borehole shear test devices. The Iowa shear test device and the University of Texas in situ device are two similar methods which may be useful for obtaining shear strength data in some compacted shale embankments (Fox, 1966; Handy and Fox, 1967; and Campbell and Hudson, 1969). None of the States contacted in the study had attempted testing with either device; however, these devices and the method they employ may be applicable to compacted shale embankments.

The principle of operation of both the Iowa and the Texas device is the same. A normal force is applied hydraulically to two curved metal surfaces bearing on opposite walls of a borehole. A tangential force is then applied and increased until a shear failure occurs in the soil close to the plate. The normal and tangential forces at failure are each divided by the area of the plates in contact with the soil to give the normal stress, σ, and the shear stress, τ. To determine shear strength, these stresses are incorporated into Coulomb's equation given below:

$$\tau = c + \sigma \tan \phi \tag{1}$$

The tests are similar to the laboratory direct shear test with three or more tests being preferred for a determination of c and ϕ (Figure 31).

Figure 32 is a schematic of the operation of the Iowa shear test device; the Texas device is illustrated in Figure 33. The Iowa device was originally designed for use in a 4-in. hole; however, the Kansas Highway Commission has used a 3-in. device of the same general design (Winter and Rodriguez, 1975). The Texas device was designed for use in holes 20 to 30 in. in diameter. Both devices were tested only to limited depths in design studies with the Iowa device being tested to depths of less than 15 ft and the Texas device to a depth of 30 ft. Winter and Rodriguez do not report on depths tested with the 3-in. device used in Kansas.

Limited tests were conducted during development of the Iowa device in glacial till, sand, Wabash loam, and Webster silty clay. A comparison of field values for the sand, loam, and silty clay with laboratory direct shear test results showed good correlation; the mean field values for ϕ and c were slightly higher than the laboratory values. Published experience in Kansas (Winter and Rodriguez, 1975) included tests in clays, silty clays, and one shale fill. Comparison of 19 field tests with consolidated-undrained triaxial test results and consolidated-drained triaxial test results indicates that the angle of shearing resistance is generally close to that obtained from consolidated-drained tests, but considerable scatter in the data was noted. Cohesion was commonly lower in borehole tests than in laboratory

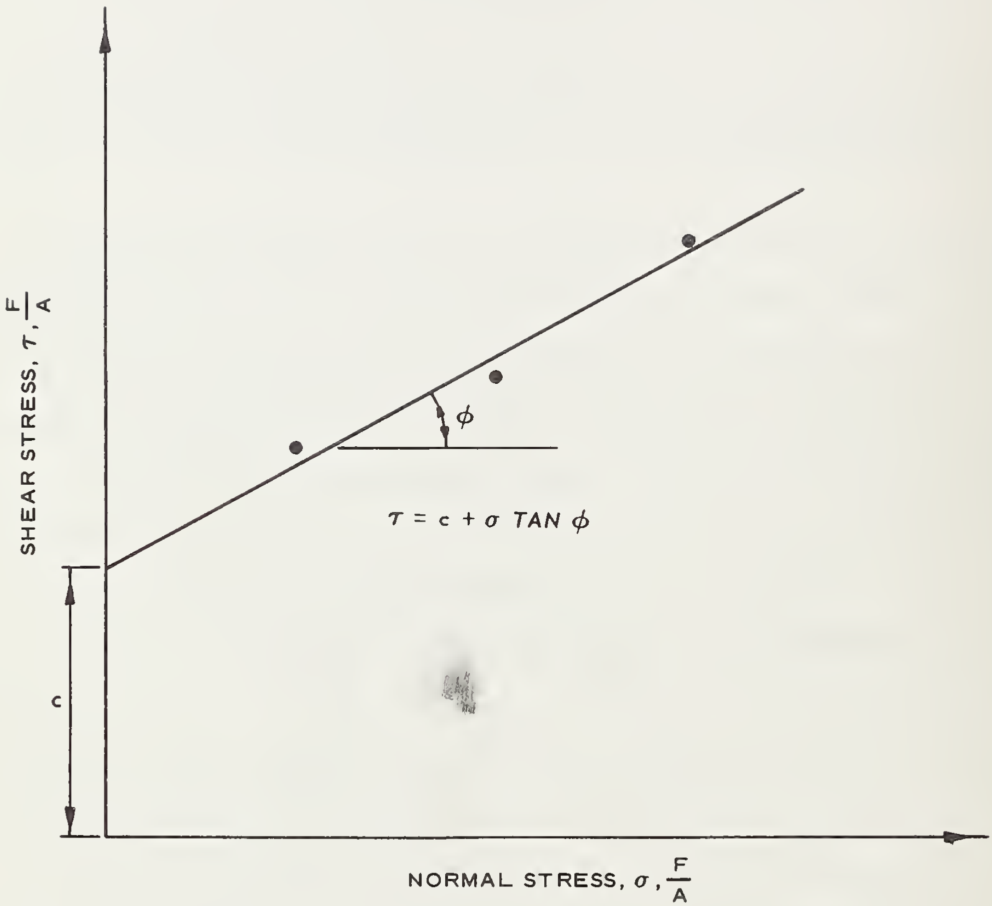


Figure 31. Shear stress versus normal stress from borehole shear device.

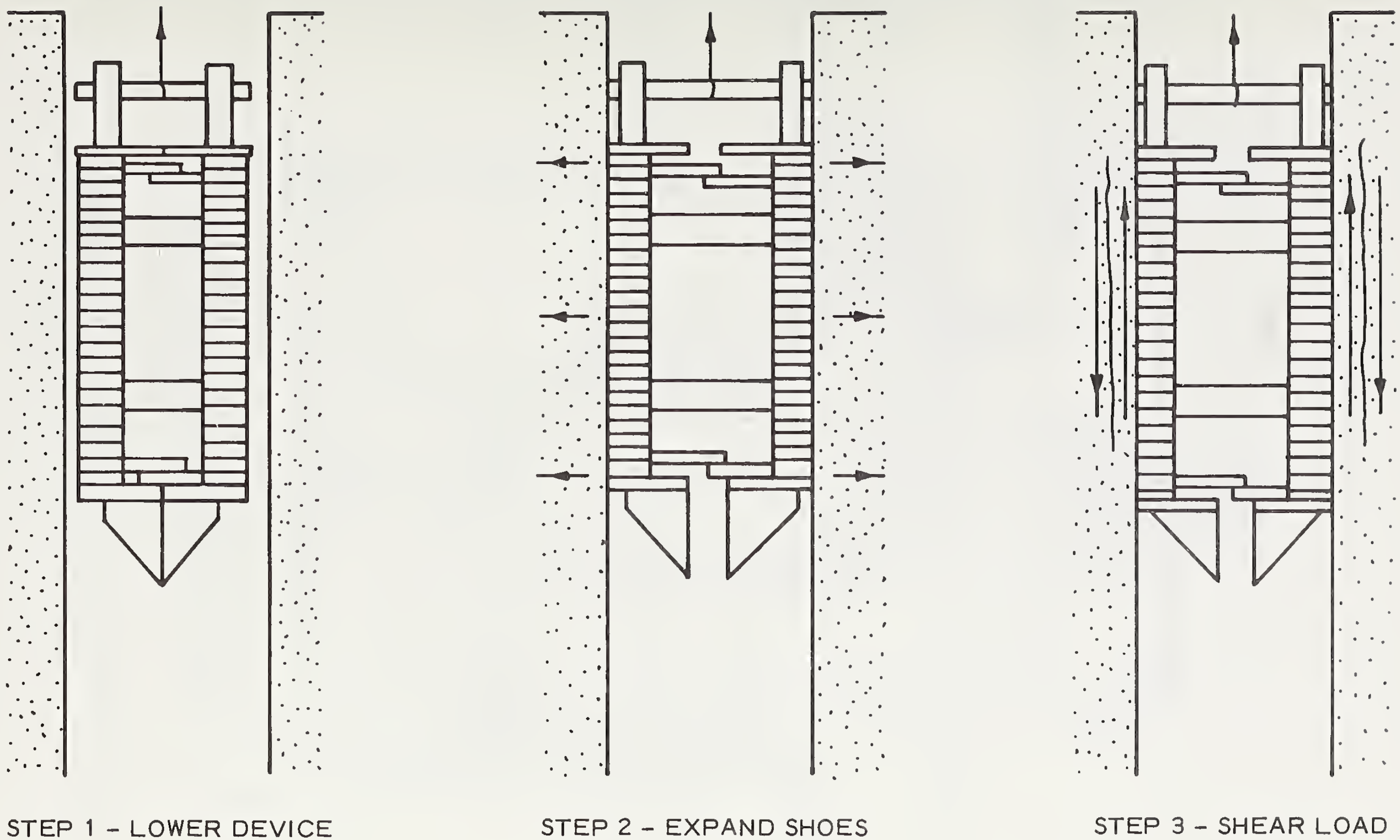


Figure 32. Operation of Iowa borehole shear device (Fox, 1966).

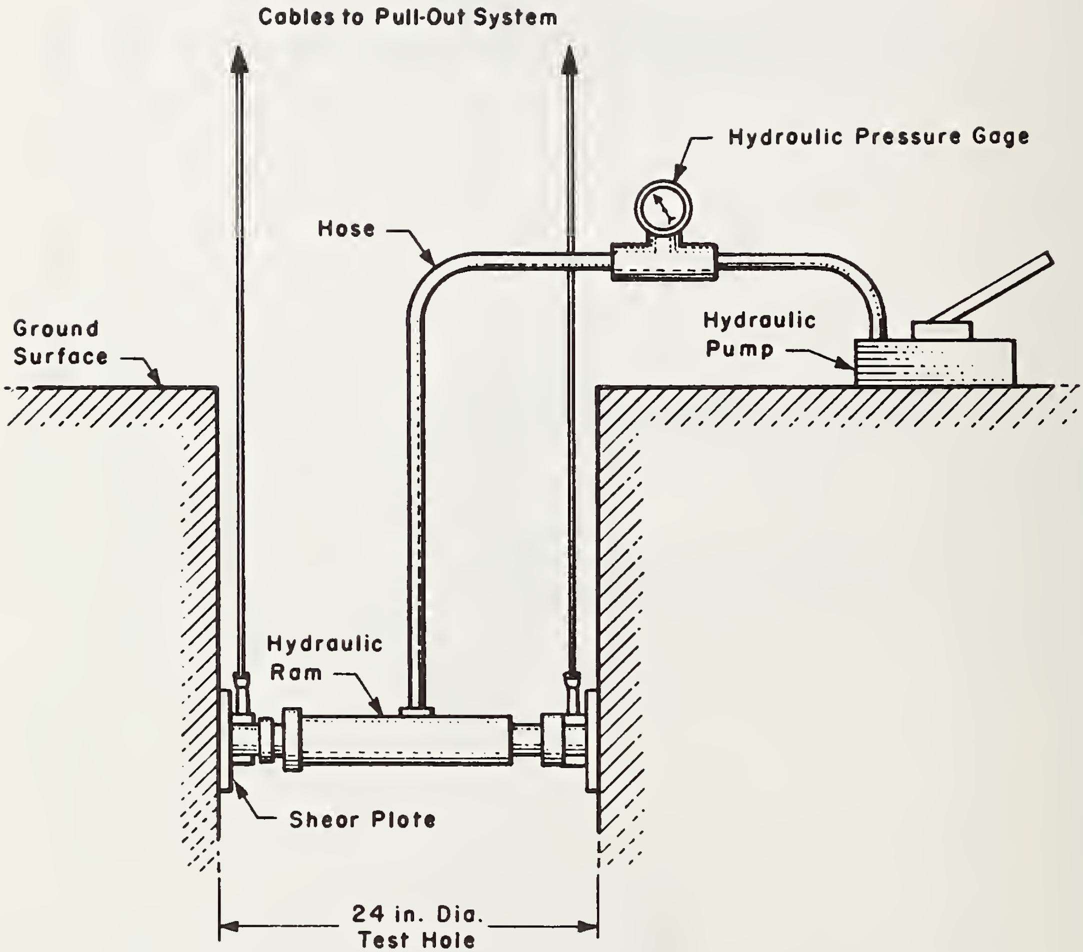


Figure 33a. Normal force system for University of Texas in situ device (Campbell and Hudson, 1969).

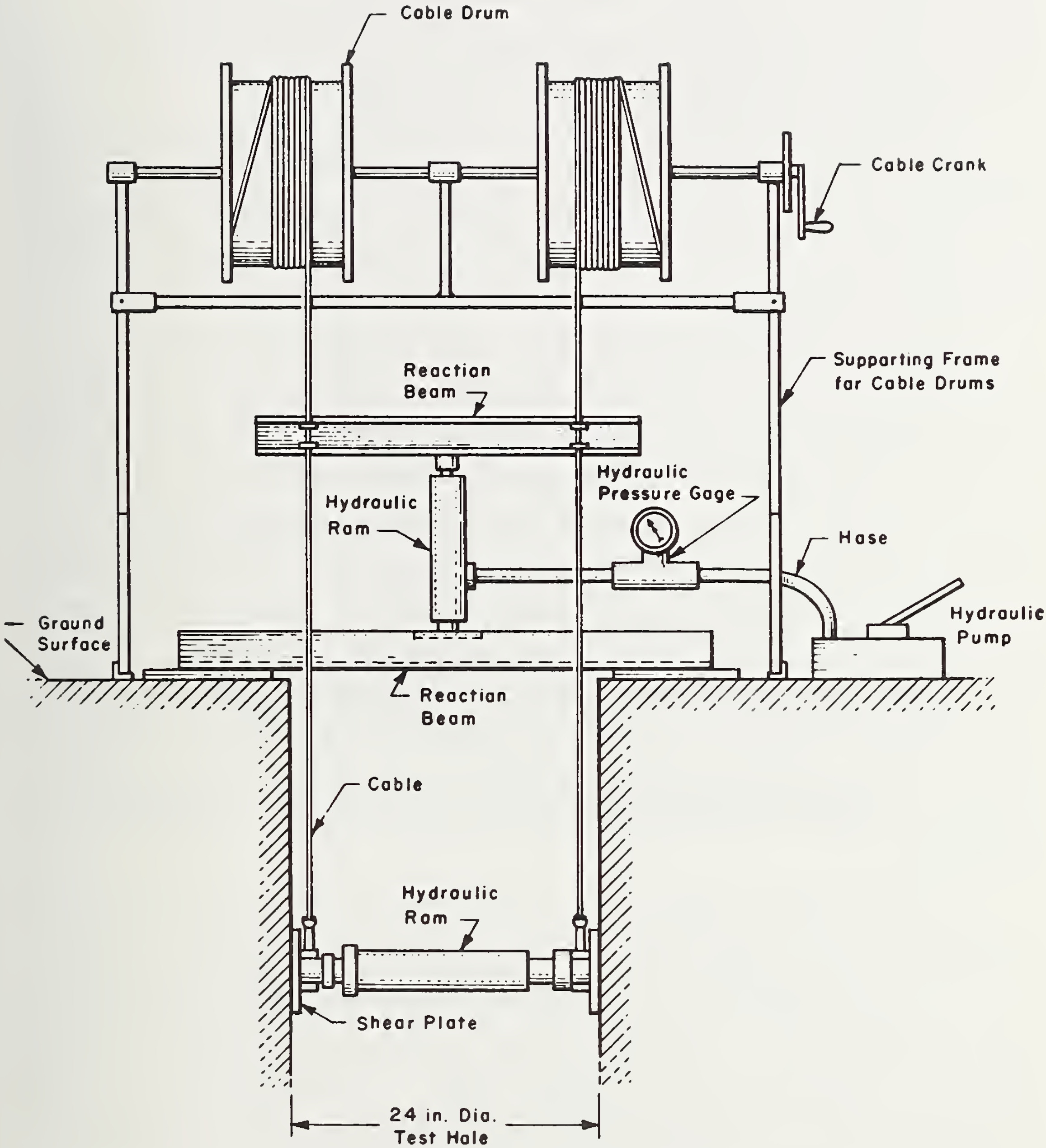


Figure 33b. Pull-out system for University of Texas in situ device (Campbell and Hudson, 1969).

tests. The Texas device was tested to a limited extent in clay. Correlations were attempted with direct shear tests and Texas Highway Department penetrometer results, but the Texas device did not correlate well with either of these. The poor correlation was attributed to operational difficulties. Discontinuing the use of the device was recommended until design improvement could be made.

Because of its limited success in soils, the Iowa device should be considered as a possible tool for evaluating soillike embankments. Use of the device should be coupled with undisturbed sampling and testing when possible. The Texas device should not be considered as a workable method of determining in situ strength properties.

Large-scale field shear testing. The possibility of obtaining strength properties of a fill material by conducting large-scale field shear testing should also be considered. However, these tests should be performed only where undisturbed samples cannot be obtained, where efforts to obtain strength parameters with borehole devices have failed, or where the presence of large particles renders the strengths obtained from tests of undisturbed samples or from borehole devices unrepresentative. Large-scale field shear tests include the direct shear test, the triaxial or multiaxial test, and the torsional shear test. A discussion of these tests relative to in situ testing of rock has been presented by Zeigler (1972). The following paragraphs discuss the testing methods, advantages, and disadvantages relative to the testing of compacted shale embankments.

The direct shear test is the simplest and most widely used of the large-scale field shear tests. It is conducted by applying a normal force to the specimen to be sheared and a shearing force perpendicular to the normal force. The normal force is held constant while the shear force is increased until the specimen fails. The failure is confined to a predetermined zone limited by the positioning of the test apparatus (Figure 34). Consolidation time under the normal force, rate of application of the shearing force, and moisture conditions of the specimen are all controlable parameters in the testing. However, the drainage conditons during the test may not be known since the test is usually

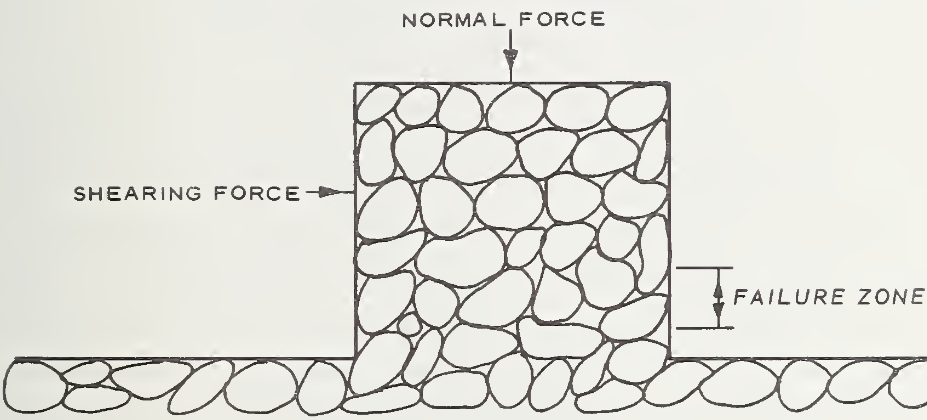


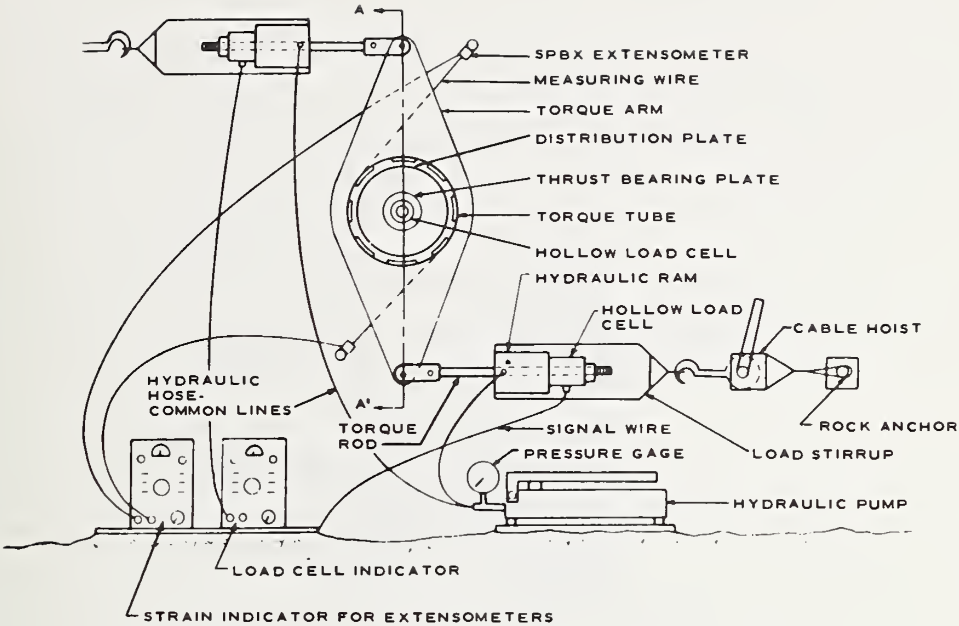
Figure 34. Principle of the in situ direct shear test.

completed in a relatively short time (a few hours). At least three tests under different normal loads should be conducted to determine the strength parameters ϕ and c .

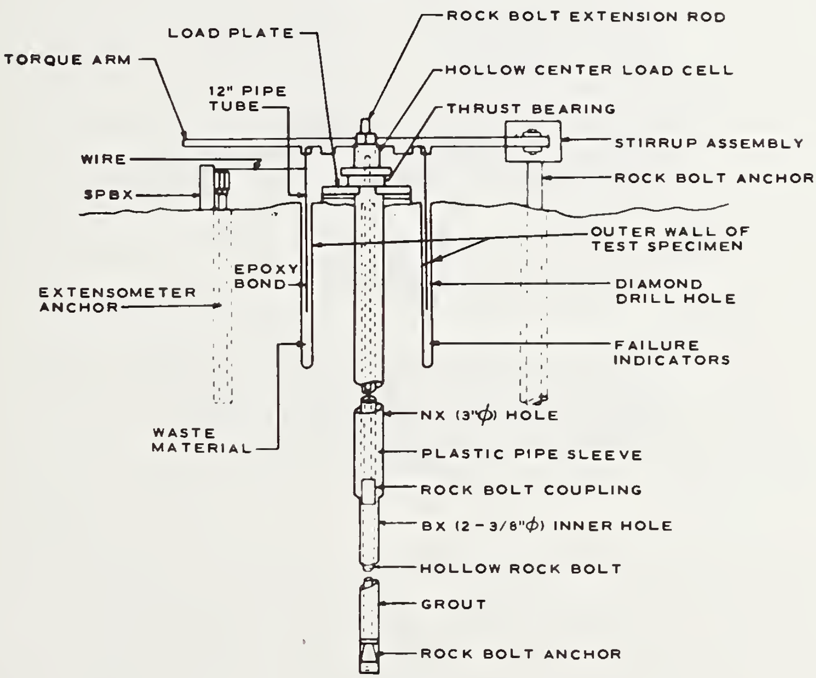
Triaxial testing in situ involves the application of stresses to a sample along three mutually perpendicular axes. The use of triaxial loading allows the sample to fail along the weakest plane. As in laboratory tests, three tests must be conducted under different confining pressures to determine ϕ and c for the material.

The torsional shear test is conducted on an annulus of material. The center of the annulus is formed by a BX-size hole and the outer wall is formed by a 12-in. or larger overcoring bit. A normal load is applied by tensioning a rock bolt anchored at the bottom of the inner hole. A torque is applied to the annulus through the rotation of a steel cylinder cemented to the outer wall (Figure 35). During the tests, torque, normal stress, and circumferential movement are measured and recorded. Shear stresses can be computed from these data for each load tested (Zeigler, 1972; LaGatta, 1970; and Teledyne Terrametrics, 1970). When at least three tests are conducted under different normal loads, the conventional shear strength versus normal stress plot can be constructed and ϕ and c determined. The apparatus shown in Figure 35 was not designed for tests in compacted shale fills; therefore, modifications will probably be required. The direct shear test and the triaxial shear test both require the preparation of several specimens in a test pit or trench. This requirement restricts the areas of testing of a roadway embankment to the slopes and the median although the direct shear test has been used within a calyx hole (Figure 36). Such use could enable testing at greater depths within the embankment. The torsional shear test would also be confined to the slopes and median areas with the possible advantage of attaining greater testing depth through coring deeper in the same hole and by using a longer torque tube.

The main advantage of a large-scale test is that it tests a large sample where variations in the material are averaged in testing to obtain a strength representative of the entire mass. Disadvantages of



a. PLAN VIEW OF TEST EQUIPMENT



b. SECTIONAL VIEW OF TEST EQUIPMENT

Figure 35. Torsional shear test apparatus (Hartmann, 1966).

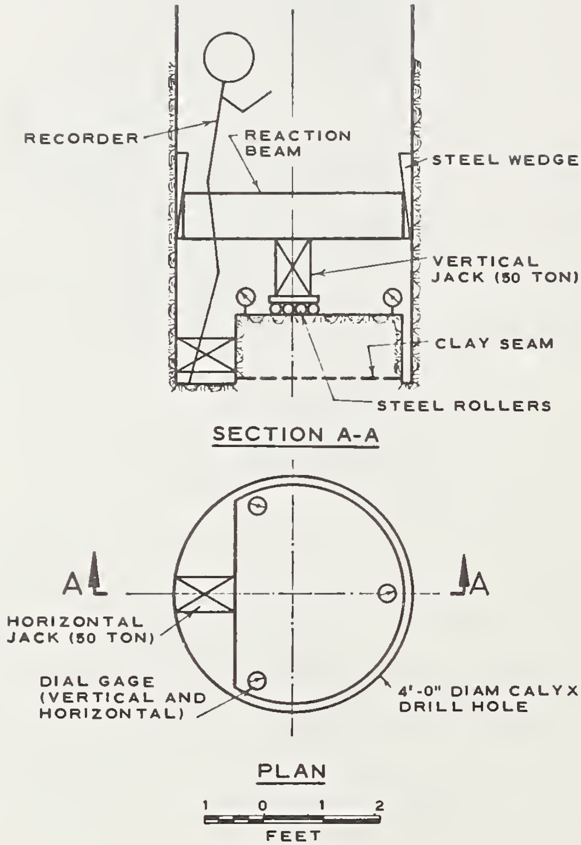


Figure 36. Arrangement of in situ shear test in a calyx hole at Meadowbank Dam (Maddox et al., 1967).

the tests are that the large sample size and time for testing make the tests costly. In an embankment where no weak zone has been defined, a large number of tests are required to provide a reliable estimate of the range of strength properties. Direct shear tests also require that a test specimen be excavated by hand which increases the time for the test and its cost. Large particle sizes may also make the specimen size needed for testing impractical.

Although large-scale shear testing may be economically impractical, it may be the only alternative. When large-scale tests are used, results should be weighed carefully in the light of other test data, previous experience, and good engineering judgment.

Plate loading tests. Compression characteristics are needed in evaluating compacted shale embankments. Plate loading tests may be applicable in determining these characteristics. Plate loading tests are conducted to estimate the bearing capacity of a material in the field. These tests are normally made by loading a circular plate through a jack and anchored beam arrangement and measuring the plate settlements under increasing load (Figure 37). When the allowable settlement, determined by design requirements, has occurred or the capacity of the loading apparatus has been reached, the plate is unloaded. Rebound measurements are recorded during unloading, and test results are plotted as shown in Figure 38. The ultimate soil pressure is taken as the vertical part of the settlement curve or the load at which the allowable settlement occurred, depending upon the test results (Bowles, 1968). Methods of relating deflection of the plate during the load test to settlement of the test area under a particular footing have been presented in the literature (Lambe and Whitman, 1969; Bowles, 1968; Taylor, 1948; and Terzaghi, 1943).

In using the plate loading test to compute the settlement of an embankment, the plate deflection to footing settlement relationships are of little use. A more important property derived from the test is the modulus of elasticity, E_s . This value can be determined from a plot of settlement, S , versus bearing capacity, q_B , from the plate load tests of two or preferably three different size plates (Figure 39).

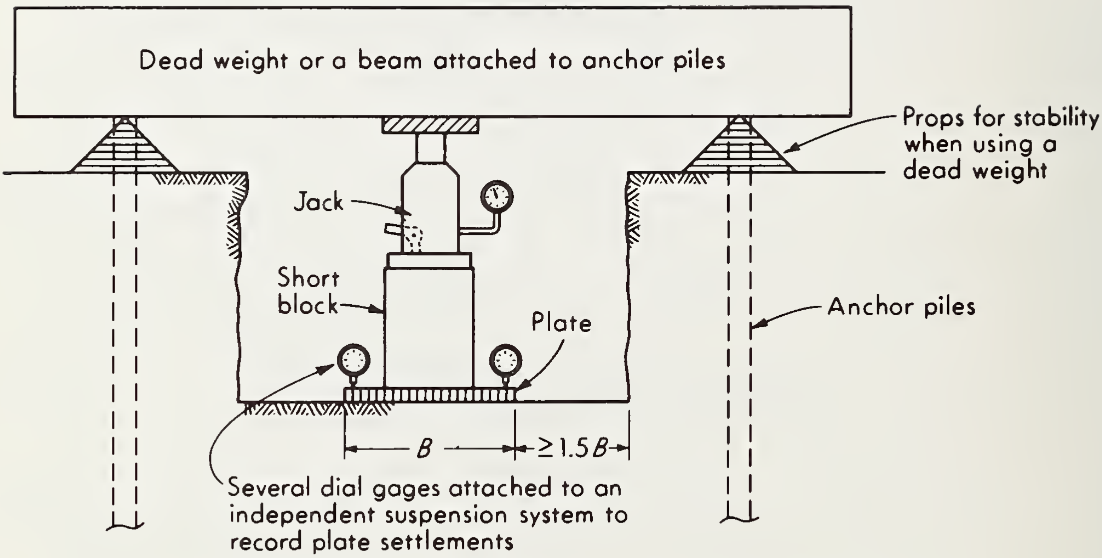
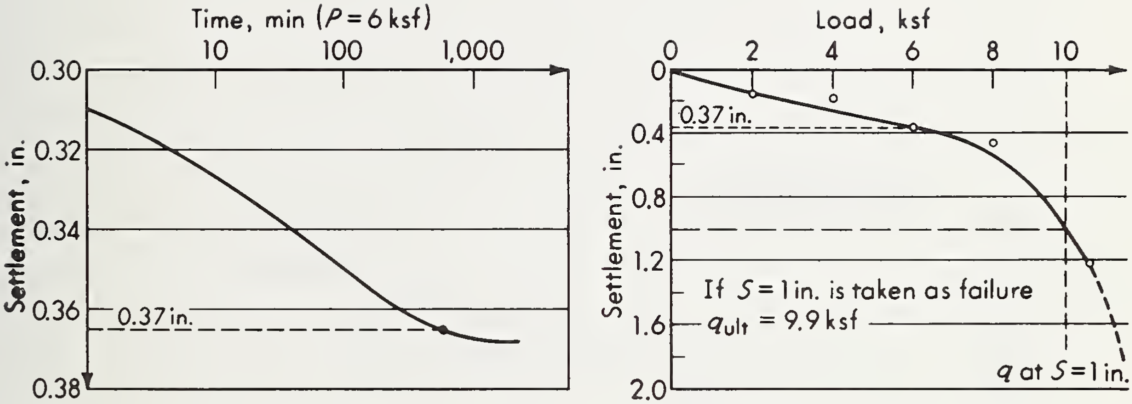


Figure 37. Plate load test apparatus (Bowles, 1968).



a. PLOT OF SETTLEMENT VERSUS LOG TIME TO DETERMINE THE MAXIMUM SETTLEMENT FOR A LOAD INCREMENT (6 KSF)

b. LOAD VERSUS SETTLEMENT PLOT TO ESTABLISH THE MAXIMUM DESIGN PRESSURE.

Figure 38. Plate load test data (Bowles, 1968).

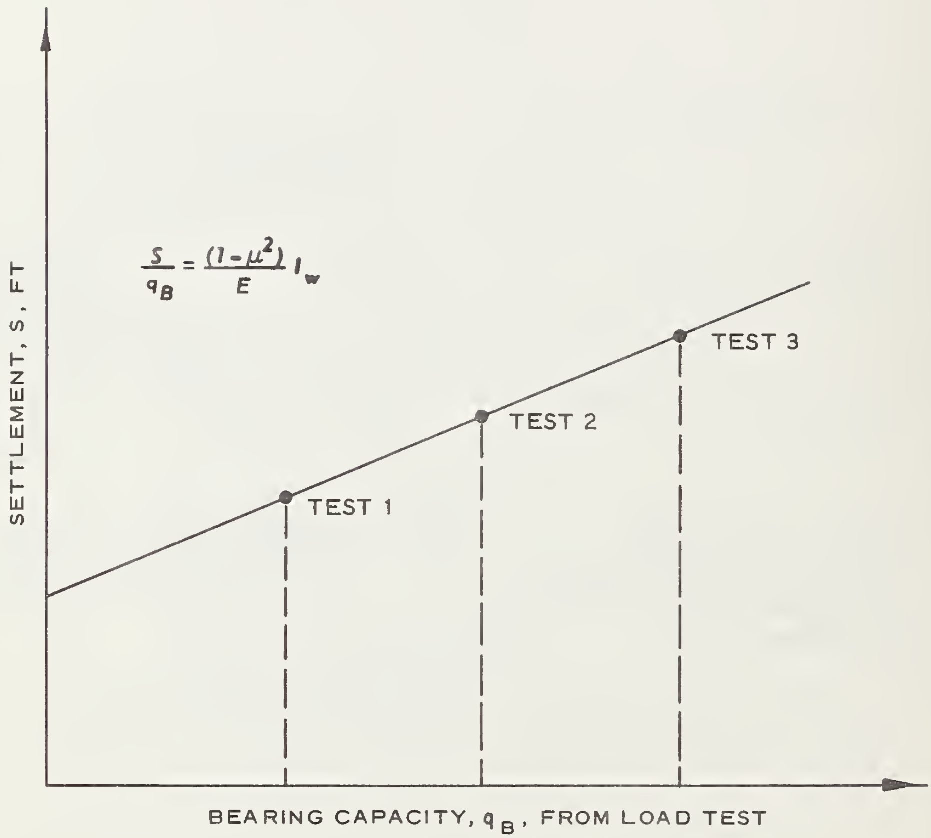


Figure 39. Determination of modulus of elasticity from plate load test (Bowles, 1968).

The equation for the plot is given as follows:

$$\frac{S}{q_B} = \frac{(1 - \mu^2)}{E_s} I_w \tag{2}$$

where

μ = Poisson's ratio

E_s = modulus of elasticity

I_w = an influence factor based on shape and rigidity of the plate

If the modulus of elasticity is to be determined, an assumption must be made for Poisson's ratio. Values of Poisson's ratio may be estimated either from laboratory tests on similar materials or by using values tabulated by others (Table 10). Table 11 gives values of I_w for various shapes of flexible and rigid footings. The value of I_w for a circular plate is 1.0. Therefore, Equation 2 can be solved for E_s from a plot similar to Figure 39. Once E_s is determined, the elastic portion of the stress-strain curve for the material can be constructed. The overburden can be used to compute the stress at a particular depth. The strain associated with that stress is then used to compute settlement for that layer. The relationship proposed by Lambe (Bowles, 1968) is:

$$S_T = \sum_{n=1}^{n} H_n \xi_n \tag{3}$$

where

S_T = total settlement of n layers

n = number of layers

H_n = thickness of layer n

ξ_n = average strain for layer n

For a granular embankment, it has been noted that the modulus of elasticity, E_s, increases with depth due to its sensitivity to lateral pressure (Burmister, 1962 and Bowles, 1968). Therefore, in order to describe the modulus of elasticity for use in settlement computations for a granular embankment, it is necessary to conduct plate load tests at different depths within the embankment or to make measurements of

Table 10. Typical ranges of Poisson's ratio after Bowles, 1968.

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand (dense)	0.2–0.4
Coarse (void ratio = 0.4–0.7)	0.15
Fine-grained (void ratio = 0.4–0.7)	0.25
Rock	0.1–0.4 (depends somewhat on type of rock)

Table 11. Influence factors for footings after Bowles, 1968.

	Flexible			*Rigid*	
Shape	*Center*	*Corner*	*Average*	I_w	I_m
Circle	1.00	0.64 (edge)	0.85	0.88	
Square	1.12	0.56	0.95	0.82	3.7
Rectangle:					
$\frac{L}{B} = 1.5$	1.36	0.68	1.20	1.06	4.12
2	1.53	0.77	1.31	1.20	4.38
5	2.10	1.05	1.83	1.70	4.82
10	2.52	1.26	2.25	2.10	4.93
100	3.38	1.69	2.96	3.40	5.06

settlements at depths beneath the loaded plate. Load tests at depths can be conducted in excavated trenches or in calyx holes. Settlement at depths beneath the footing can be measured using a variation of the plate loading test. The method utilizes an annular plate as a load plate. A hole is drilled through the opening in the plate into the material to be tested and a vertical extensometer is installed. The extensometer is capable of obtaining measurements of movements for limited depths beneath the load plate. From these measurements, a stress-strain curve can be plotted for each layer and a modulus computed for each depth. However, difficulty might be encountered in anchoring the extensometer in a rock fill.

The settlement computed using modulus values from plate loading tests is elastic settlement and occurs upon loading. For a compacted embankment this settlement occurs during construction, and the value computed from a plate load test after construction will not provide insight into predicting future settlements. Therefore, the value obtained for settlement is not as important as the modulus values for each depth tested. A comparison of the modulus values for different depths will indicate the presence of soft zones of weaker materials and possible problem sources. Determination of modulus values at different time intervals will indicate any change in modulus with time and will give an idea of the rate of deterioration of the embankment material.

Plate loading tests have not been used by any of the States contacted in this study. The expense of conducting plate load tests at depths within an embankment may be prohibitive for wide use. The modulus values obtained from plate load tests, however, might be useful when compared with modulus values obtained from other inexpensive methods of in situ testing such as the Menard pressuremeter.

Visual examination. Other than sampling an embankment to determine its strength properties, a qualitative idea of the strength can sometimes be gained by examining the walls of a borehole. Probes containing photographic or television cameras have been designed for use in examining small-diameter boreholes. The probe is lowered into a drill hole and a continuous film record is made or a closed-circuit picture is transmitted to a control console. Directional orientation is provided by a compass. Borehole television devices have been described by Smith (1966) and by Logan (1965), and a guide for use has been presented by Lundgren et al. (1970). Borehole camera devices have been used primarily as geologic aids in determining hole inclination and drift, stratification, and hole orientation. Other uses of the devices have included examination of drill holes prior to grouting, after-grouting surveys, examination of drainage wells, examination of structural foundations, and numerous other applications. Another method of examination of the fill properties by direct observation involves the drilling of a large-diameter borehole (calyx hole), usually 4 ft or more in diameter. After the hole is drilled, an inspection of the walls of the hole is made and pertinent geotechnical features are mapped. However, the drilling of a calyx hole is expensive and the use of this technique should be limited.

Information to be gained from either borehole television or examination of a calyx hole includes particle size, layering, degree of weathering, presence of voids, seepage and drainage characteristics, and location of weak zones. Obviously, the use of either of these methods requires that the holes remain open without the use of casing or drilling mud. Attempts to correlate observations from borehole television or calyx hole examinations are coupled with available data from instrumentation, laboratory testing, and in situ testing to provide a more complete description of the fill under study.

<u>Geophysical methods</u>. Only one State in the study had attempted to use geophysical methods to evaluate a fill. West Virginia obtained data on moisture and density in five cased boreholes by utilizing Schlumberger Well Services. A directional tool for density and an omnidirectional tool for water content logging were first used; however, inconsistent density data resulting from spiraling of the cable led to the use of omnidirectional density equipment. Consistency of the results using this tool has not yet been determined. Moisture-density probes are available commercially. One moisture probe has been evaluated by the Texas Highway Department and found to be fast, efficient,

and accurate (Ehlers et al., 1969). Other geophysical methods such as seismic refraction or reflection, electrical resistivity, and continuous vibration are not known to have been used in evaluating a compacted shale embankment.

Seismic refraction has been utilized in site surveys as a preliminary method of establishing the depth of denser materials; however, it is limited to depths of about 100 ft. Reflection methods have been used to determine depths of rock exceeding 2000 ft (Department of the Navy, 1962). These methods are based upon the fact that the velocity of a shock wave passing through a material is dependent upon the density of the material. Density of the material can thereby be related to travel time of the wave from a source to a receiver. Use of the methods requires close correlation of seismic data with borings (Golder and Soderman, 1963; and Bowles, 1968). Seismic methods are not applicable to the evaluation of a fill since they require sharp transitions in material densities which are not likely in a compacted fill. The seismic method may be of practical use in defining the contact between fill and foundation material when the number of borings is limited.

Electrical resistivity has also been used in site surveys to determine horizontal extent and depth of strata up to 100 ft. Trantina (1962) has also correlated resistivity measurements to failure plane location in investigations of landslides. Resistivity methods involve passing an electrical current of known magnitude through the ground between two electrodes and measuring the gradient between them using potential electrodes. Resistivity can be computed from these measurements and is correlated to material type. Resistivity measurements are highly dependent upon the degree of saturation of the material and the dissolved salts present. As with seismic methods, resistivity measurements must be interspersed with boring data to avoid misinterpretation of data. Resistivity measurements, therefore, will be of little use in evaluating a compacted shale fill.

Continuous vibration methods consist of measuring the travel time of transverse shear waves (generated by a mechanical vibrator) traveling from the source to seismic detectors located at specific distances from the vibrator (Department of the Navy, 1962). Wave velocity can be correlated to material type. The method has been used to determine elastic and shear moduli, Poisson's ratio, principal stresses and shear stresses, trajectories of equal stresses, soil density and viscosity, and strata depths (Bernhard, 1970). Applications to compacted shale embankments should be investigated. Weissman (1970) suggests a method for obtaining soil shear moduli by in situ vibratory techniques, and Bernhard (1970) gives an extensive bibliography on the subject. Discussions of procedures are also presented by Ishimoto and Iida (1937), Maxwell and Fry (1967), and Stevens (1970).

Analysis and Prediction

After gathering data on a problem fill from all practical sources, the engineer must assess the validity of the data and apply it to predict the future of the fill. If continuing settlement is the problem, the cause must be determined and a prediction made of the extent, rate, and amount of future settlements. Should slope stability be in doubt, an analysis to determine the factor of safety of the slope must be undertaken. At times, a failure will have already occurred and remedial treatment will be necessary. For these cases, strength properties must be determined for use in the design of remedial treatment measures. The following paragraphs will describe these evaluations and predictions with reference to the application of data accumulated through instrumentation, sampling and laboratory testing, and in situ investigations.

Settlement. The compacted shale embankment can vary from a soillike fill to a rock fill (Shamburger et al., 1975). In soillike fills total settlement is usually related to the consolidation of both the fill and the foundation. When soils are compressed under a load, consolidation occurs in three phases: initial, primary, and secondary Initial consolidation is caused by a squeezing out of the air or gases in the voids of the soil and occurs almost immediately. Primary compression is caused by dissipation of pore pressures induced by loading.

Secondary compression occurs due to a change in void ratio after the dissipation of pore pressures. Methods of computing ultimate settlement based on consolidation tests on undisturbed samples have been presented in the literature (Taylor, 1948; Department of the Navy, 1962; Terzaghi and Peck, 1967; Bowles, 1968; and Lambe and Whitman, 1969).

The theory of consolidation was first expressed by Terzaghi in the 1920's. Assumptions made in the theory are stated by Taylor (1948) as:

a. Homogeneous soil.

b. Complete saturation.

c. Negligible compressibility of soil grains and water.

d. Action of infinitesimal masses no different from that of larger representative masses.

e. One-dimensional compression.

f. One-dimensional flow.

g. Darcy's law valid.

h. Constant values for certain soil properties which actually vary somewhat with pressure.

i. An idealized pressure versus void ratio relationship.

From these assumptions it is apparent that the theory does not apply to settlement of a compacted shale embankment, or strictly speaking, to the foundation of the embankment. Two- and three-dimensional consolidation solutions have proven useful in practice (Lambe and Whitman, 1969).

Finite element solutions are also capable of predicting deformations of embankments and their foundations based on their stress-strain characteristics and the theory of elasticity (Chang et al., 1972a). However, the application of finite element solutions to compacted shale embankments is complicated by the need for obtaining stress-strain characteristics. The stress-strain characteristics can only be obtained from laboratory tests of undisturbed samples which for compacted shale may be difficult or impossible to obtain.

For rock fills, settlement does not result from consolidation or elastic deformation but from migration of fines from between points of contact and from crushing of the contact points between rock particles (Sowers and Sally, 1962). Migration of fines causes a reorientation of rock particles into a denser structure and results in settlement. Crushing of contact points permits movement of the rock particles, settlement, and possibly continued crushing and more settlement. The saturation of rock particles has been found to accelerate their settlement in laboratory compression tests on graywacke, granite, and sandstone (Sowers et al., 1965). The problem of estimating settlement in compacted shale embankments is further complicated by a time dependent reduction in strength due to deterioration of the shale particles. This reduction increases the likelihood of crushing between particles and, therefore, the probability of excessive settlements.

Large-scale compaction and triaxial testing has been performed for use in design studies of a number of CE dams (Shamburger et al., 1975). Large-scale testing has also been conducted for highway embankments (Hall and Smith, 1967). Application of large-scale consolidation tests for use in predicting settlement may also be possible. Samples have been compressed using 1-ft-sq shear boxes in England for investigating settlement of colliery spoil heaps. Extrapolations of test results on 3/4-in. material have provided reasonable estimates of expected settlements (Taylor, 1975).* However, the maximum load that can be applied with the device is 1 tsf, limiting the device to tests for embankment heights of less than 16 ft. More sophisticated testing for higher embankments might also be justified depending upon the cost of the testing in relation to the cost of the project and also the potential for savings based on more accurate material properties (Penman, 1971).

Continued settlements of compacted shale embankments, whether soil-like or rocklike, can best be predicted by monitoring settlements using profiling methods or settlement markers or pins. Settlements should be plotted versus time on a semilog plot. The plot will aid in comparing the actual settlement with the theoretical shape of the settlement curve for soils (Figure 40) and field and laboratory test curves for rock (Figures 41 and 42). Both the primary and the secondary

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* R. K. Taylor (1975), Personal communication, Senior Lecturer in Engineering Geology, University of Durham, England.

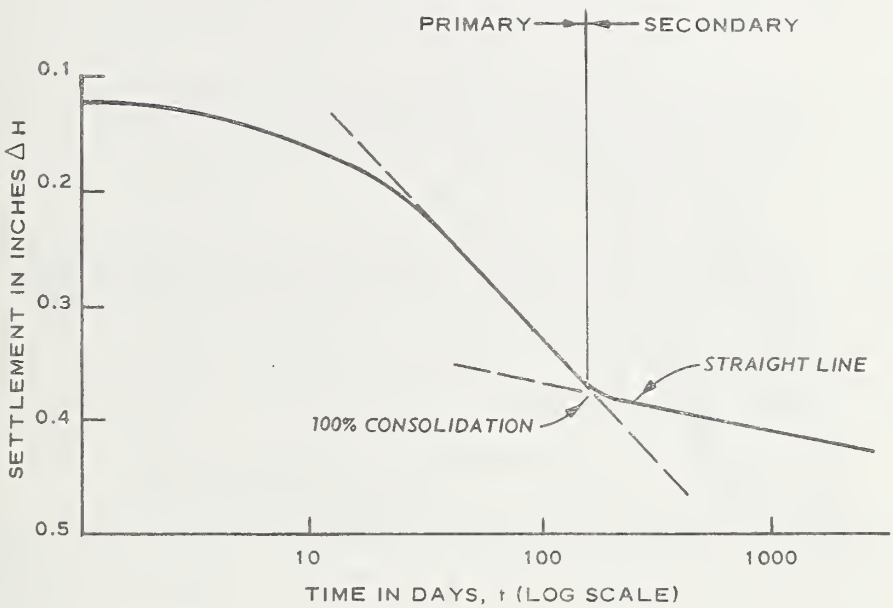

Figure 40. Settlement versus log time for soil.

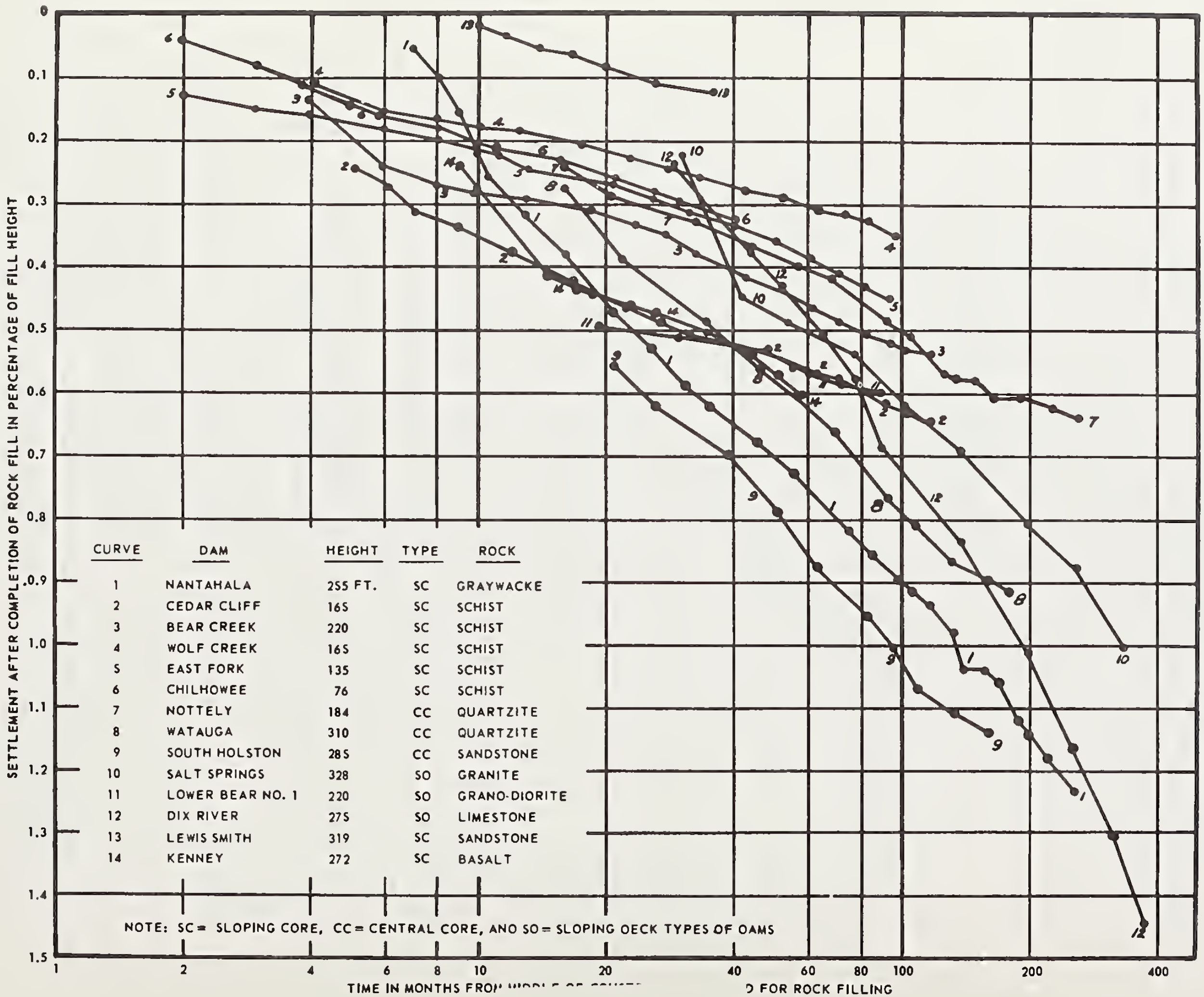

Figure 41. Observed settlement of rock-fill dams after completion of construction (Sowers, Williams, and Wallace, 1965).

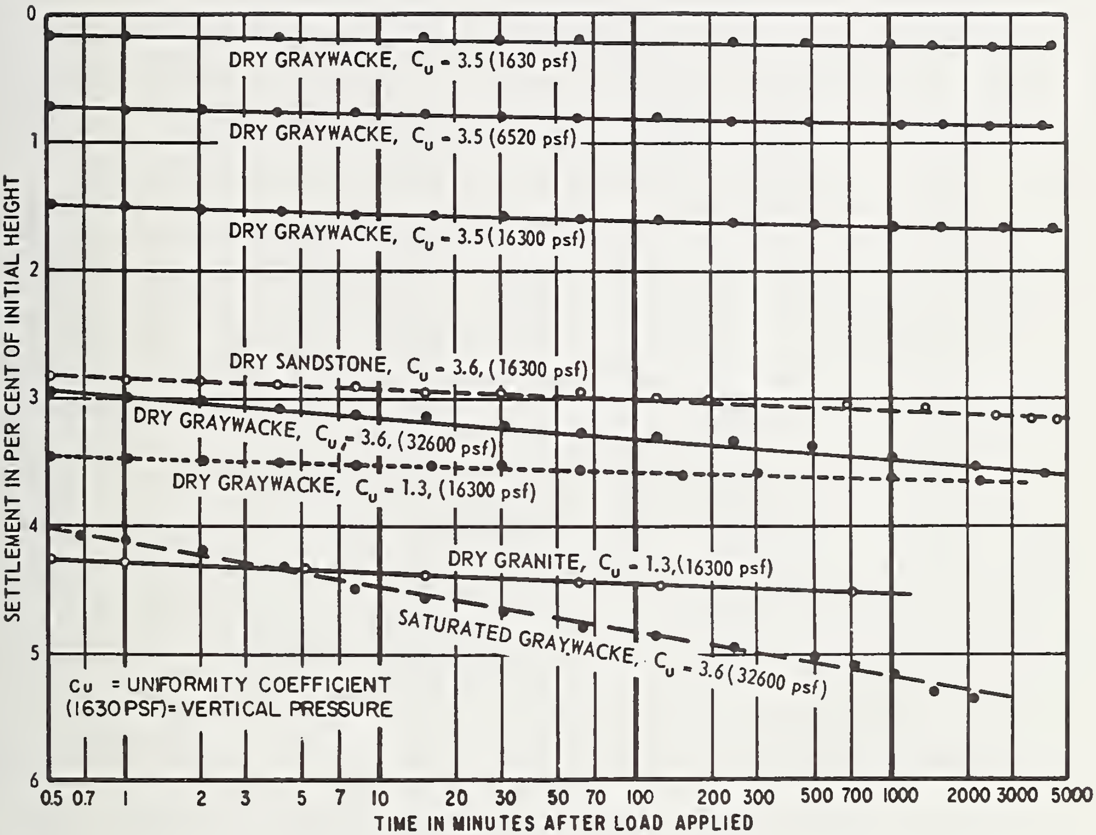

Figure 42. Settlement versus log time curves for laboratory confined compression tests of broken rock for constant vertical pressures applied in increments (Sowers, Williams, and Wallace, 1965).

consolidation portions of the curves for soil should be relatively straight lines with the rate of settlement during primary consolidation normally greater than that during secondary compression. Rock-fill settlement data from field observations on dam embankments (Figure 41) and from laboratory tests (Figure 42) indicate a straight line relationship similar to the secondary compression in soils (Sowers, Williams, and Wallace, 1965). The extrapolation of plotted observations should reasonably predict future settlements.

Slope stability. The determination of the factor of safety of an embankment slope against sliding can be made using laboratory strength data from tests on undisturbed samples or strength data from in situ tests. Numerous computer solutions are available. Wright (1969)* describes selected stability analysis procedures according to equilibrium conditions satisfied and the shape of the failure surface (Table 12). A classification of selected procedures for stability analyses is shown in Table 13. An evaluation of these stability analysis procedures by Johnson (1975) is shown in Table 14.

The Simplified Bishop analysis is the most widely used and the most suitable for evaluating embankment behavior. Numerous comparisons of results using the Simplified Bishop method and more rigorous analyses have shown it to give compatible results (Johnson, 1975). Detailed explanations of stability analysis procedures for various methods are given in the references listed in Table 13 and in USAE, OCE (1970b); Lambe and Whitman (1969); Taylor (1948); and numerous other publications. It is not only important to use a satisfactory method of analysis but also to select the proper strength properties for input into the analysis. To do this, the situation existing in the embankment must be evaluated and proper strengths assigned. The data available on the condition of the fill will determine the necessary strength values and the type analysis. Figure 43 is a greatly simplified example for two extremes of fill material. For claylike materials, the safety factor

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* S. G. Wright (1969), "A Study of Slope Stability and the Undrained Shear Strength of Clay Shales," Ph. D. Thesis presented to the University of California at Berkeley.

Table 12. Equilibrium conditions satisfied by various procedures of stability analysis after Wright, 1969.*

PROCEDURE OF ANALYSIS	EQUILIBRIUM CONDITIONS SATISFIED						SLIP SURFACE
	Overall			Individual Slices			
	Moment	Vertical Force	Horiz. Force	Moment	Vertical Force	Horiz. Force	
(ϕ = 0) Method	Yes	(Yes)**	(Yes)	Not a Slices Procedure			Circular Arc
Logarithmic Spiral	Yes	(Yes)	(Yes)	Not a Slices Procedure			Log Spiral
Plane Shear Surface, Culmann	Yes	Yes	Yes	Not a Slices Procedure			Plane
Friction Circle Method	Yes	Yes	Yes	Not a Slices Procedure			Circular Arc
Frohlich	Yes	Yes	Yes	Not a Slices Procedure			Circular Arc
Bell	Yes	Yes	Yes	Not a Slices Procedure			General Shape
Ordinary Method of Slices - Fellenius	Yes	No	No	No	No	No	Circular Arc
Petterson	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape†
Fellenius Rigorous Graphical	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape†
Raedschelders	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape†
Modified Bishop	Yes	(Yes)	No	No	Yes	No	Circular Arc
Bishop Rigorous	Yes	(Yes)	(Yes)	Yes	Yes	Yes	Circular Arc
Nonveiller	Yes	(Yes)	(Yes)	(Yes)	Yes	Yes	General Shape
Spencer	Yes	(Yes)	(Yes)	Yes	Yes	Yes	General Shape†
Morgenstern and Price	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape
Janbu et. al. - Horiz. Side Forces	No	(Yes)	(Yes)	No	Yes	Yes	General Shape
Lowe and Karafiath	No	(Yes)	(Yes)	No	Yes	Yes	General Shape
Corps of Engineers - Modified Swedish Method	No	(Yes)	(Yes)	No	Yes	Yes	General Shape
Janbu Generalized Procedure of Slices - (CPS)	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape
Seed and Sultan	No	(Yes)	(Yes)	No	Yes	Yes	Two Sliding Wedges
Corps of Engineers - Sliding Block	No	(Yes)	(Yes)	No	Yes	Yes	Three Sliding Blocks

* S. G. Wright (1969), "A Study of Slope Stability and the Undrained Shear Strength of Clay Shales," Ph. D. thesis presented to the University of California at Berkeley.

** Parentheses indicate that this condition of equilibrium is implicitly satisfied as a result of the direct consideration of other equilibrium conditions.

† The original presentation of this procedure was for a circular shear surface only.

Table 13. Classification of selected procedures for stability analyses after Johnson, 1975.

Procedure	Computation by Hand	Computation by Computer
Class I: Approximate procedures (as good as rigorous procedures for ϕ small or zero)		
Ordinary method of slices--Swedish method (USAE, OCE, 1952)	Yes	Yes
Class II: Intermediate procedures (generally work well for design purposes)		
Simplified Bishop (Bishop, 1952 and Bishop, 1955)	Yes	Yes
Taylor--Lowe (Taylor, 1949 and Lowe, 1967)	Yes	Yes
Taylor--CE (Taylor, 1949; USAE, OCE, 1952; and USAE, OCE, 1970b)	Yes	Yes
Class III: Rigorous procedures		
Janbu (Janbu, 1973)	Yes	Yes*
Morgenstern and Price (Morgenstern and Price, 1965)	No	Yes*
Spencer (Spencer, 1967)	No	Yes*
Sarma (Sarma, 1973)	Yes	Yes

* Possible convergence difficulties in some cases.

Table 14. Evaluation of stability analyses procedures (Johnson, 1975).

Characteristic (1)	Remarks (2)
1. Results achieved	(a) For c large and ϕ small: All methods give about same result, including ordinary method of slices, except that Taylor-Lowe and Taylor-CE occasionally give slightly high safety factors
	(b) For c small and ϕ large: Ordinary methods of slices are too conservative. Others give about same results
	(c) For circles extending beyond toe: Taylor-Lowe give slightly high safety factors; Taylor-CE must assume horizontal interslice forces beyond toe
2. Practicality of hand computation	Approximate order of preference: Ordinary method of slices; Simplified Bishop, Taylor-Lowe, or Taylor-CE
3. General suitability for most purposes with circular surfaces and computer solution	Approximate order of preference: Use any method except ordinary method of slices; Simplified Bishop, Taylor-Lowe, or Taylor-CE are suitable
4. For evaluating proposed methods	Use Morgenstern-Price, but recent program improvements make it suitable for routine use if desired

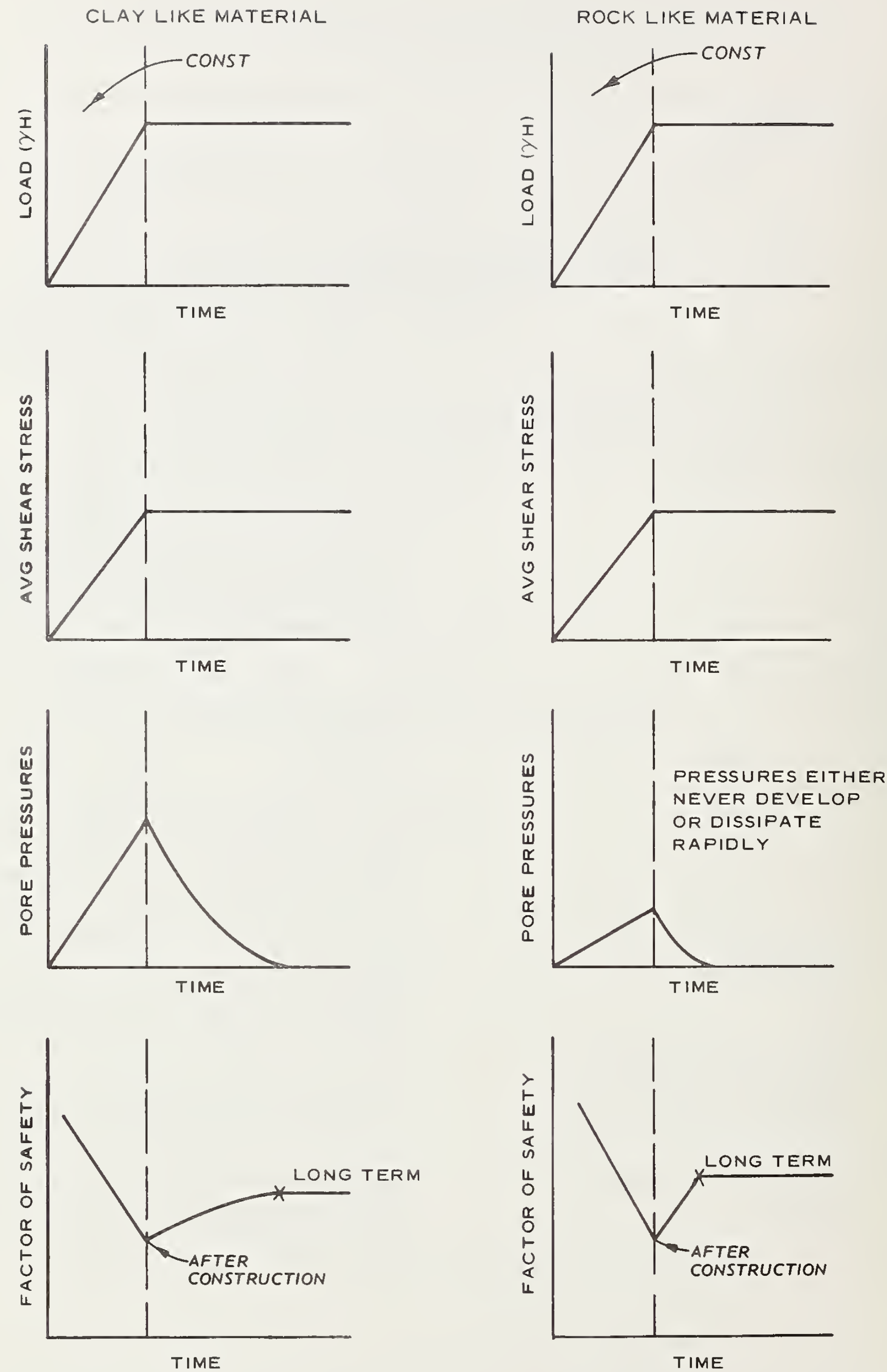

Figure 43. Changes in shear stress, pore pressures, and safety factor during and after construction for claylike material and rocklike material.

decreases as the embankment height increases; whereas upon completion of construction, dissipation of construction-induced pore pressures causes an increase in the safety factor which continues until pore pressures have dissipated and the long-term safety factor is reached. For rock-like materials, either the development of construction pore pressures does not occur or the pressures are dissipated much more rapidly. Thus, the long-term case is reached more quickly and sometimes is identical to the after-construction case. It is apparent that the key parameter in the determination of the factor of safety at any time is pore pressure. Piezometers must be used to define the pore pressures within the embankment under study. If piezometer data are not available, an estimation of the pore pressure condition must be made using water elevations in boreholes or engineering judgment. Two methods of analyses are possible: total stress analysis and effective stress analysis. Table 15 lists the conditions to be analyzed and the preferred method of analysis. In general, the effective stress method is considered the best method of analysis by many geotechnical engineers. Pore pressures are estimated or obtained from field instrumentation, and strength data are obtained from laboratory consolidated-drained or consolidated-undrained tests with pore pressures measured. In situ strengths determined from field tests can also be used in analyses when proper correlations to field conditions are available.

Back analysis. When a slope failure occurs or a shear plane is found from inclinometer observations, remedial measures are necessary. Evaluation in these cases consists of studying the available data to determine causes of failure and arrive at strengths for use in remedial designs. One method of obtaining the needed strengths is through a back analysis of the failed or failing slope.

A back analysis of the failed slope makes use of the fact that the factor of safety (FS) at failure is equal to or slightly below 1.0. To perform a back analysis it is necessary to:

a. Describe the existing failure surface and slope geometry (plane, circular, wedge, or combination).

b. Describe the internal and external forces acting on the fill at failure.

Table 15. Total stress method versus effective stress method after Lambe and Whitman, 1969.

Situation	Method
1. End-of-construction	
a. Saturated material	Total stress with ϕ and c from unconsolidated-undrained tests.
b. Partially saturated material	Either total stress with ϕ and c from unconsolidated-undrained test or effective stress with ϕ and c from consolidated-drained plus estimated pore pressures.
2. Long-term stability	Effective stress with pore pressures from groundwater conditions.
3. Stability at intermediate times	Effective stress with pore pressures estimated or from actual field measurements.

c. Set the expression for factor of safety for the described conditions equal to 1.0.

d. Solve for the family of strength parameters that will satisfy the expression.

Since there are an infinite number of combinations of ϕ and c that will satisfy the back analysis, an assumption is sometimes made regarding one of the variables. Typical assumptions are that either ϕ or c is equal to 0 depending on the soil type and pore water pressures.

The failure surface can be described when data from slope inclinometers are available as previously discussed and shown in Figure 20. Field measurements can also locate the failed surface at the crest of the embankment and the area of bulging at the toe. When subsurface data from inclinometers, shear strips, or other devices are not available, the shape of the failure surface between the crest and the toe can be estimated considering boring logs, and locations of soft or weak zones or layers.

Internal loading is the pore pressure. Piezometric data, if available, can be used to estimate pore pressures. Otherwise, an assumption of the pore pressures at failure must be made.

The solution of the expression for factor of safety using computer techniques usually involves successive trials since most programming has been set up to search for the critical failure surface for given values of ϕ and c.* Each trial requires changing ϕ and/or c until a factor of safety near 1.0 is obtained for a failure surface conforming to the actual failure surface. When this is done, one particular set of values for ϕ and c is defined that could exist at equilibrium (FS = 1.0) for the failure surface and pore pressures analyzed. An alternate method involves an analysis using a particular ϕ and c for the failure surface, division of $\tan \phi$ and c by the resulting safety factor, plotting c/FS versus $(\tan \phi)/\mathrm{FS}$, and repeating for different values of ϕ and c .

* The Morgenstern-Price method has provisions for determining the factor of safety for a given failure plane and different values of ϕ and c .

The individual points on the plot (Figure 44) will fall in a straight line provided the definition of factor of safety is expressed as:

$$FS = \frac{\text{available shear strength}}{\text{shear stress required for equilibrium}} = \frac{c + \sigma \tan \phi}{\tau} \qquad (3)$$

The resulting plot contains a complete description of all possible combinations of ϕ and c on the failure plane at failure.

The values obtained from a back analysis can be compared with laboratory data and in situ test data to aid in the selection of strength properties for reconstruction. By additional analyses, the effect of fluctuation of the water table upon the strength required for stability can also be determined. Back analyses correctly performed and interpreted can provide valuable information for design of remedial measures.

The use of back analysis by States contacted in this study for determination of strength properties for design of remedial measures was limited to three States. The highway departments of Indiana, Kentucky, and Tennessee have all used a back analysis method to some extent to determine strength properties; however, the type analyses used and the assumptions made were somewhat different.

In back analyzing SLIDE-2 on I-74,* the Indiana State Highway Commission used the scarp and limits of the slide observed in the field to describe a wedge-type failure surface. Prior experience with similar materials led to the assumption of a friction angle of 0. Cohesion was then calculated from the geometry of the problem, assuming an equilibrium condition existed (FS = 1.0). The cohesion calculated from the back analysis was used in the design of remedial measures. The Kentucky Department of Transportation has used back analyses in several cases for determining strength properties of materials involved in embankment failures (Girdler and Hopkins, 1973; and Hopkins, 1972). The method of analysis used by the Kentucky Department of Transportation

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* See footnote on page 55.

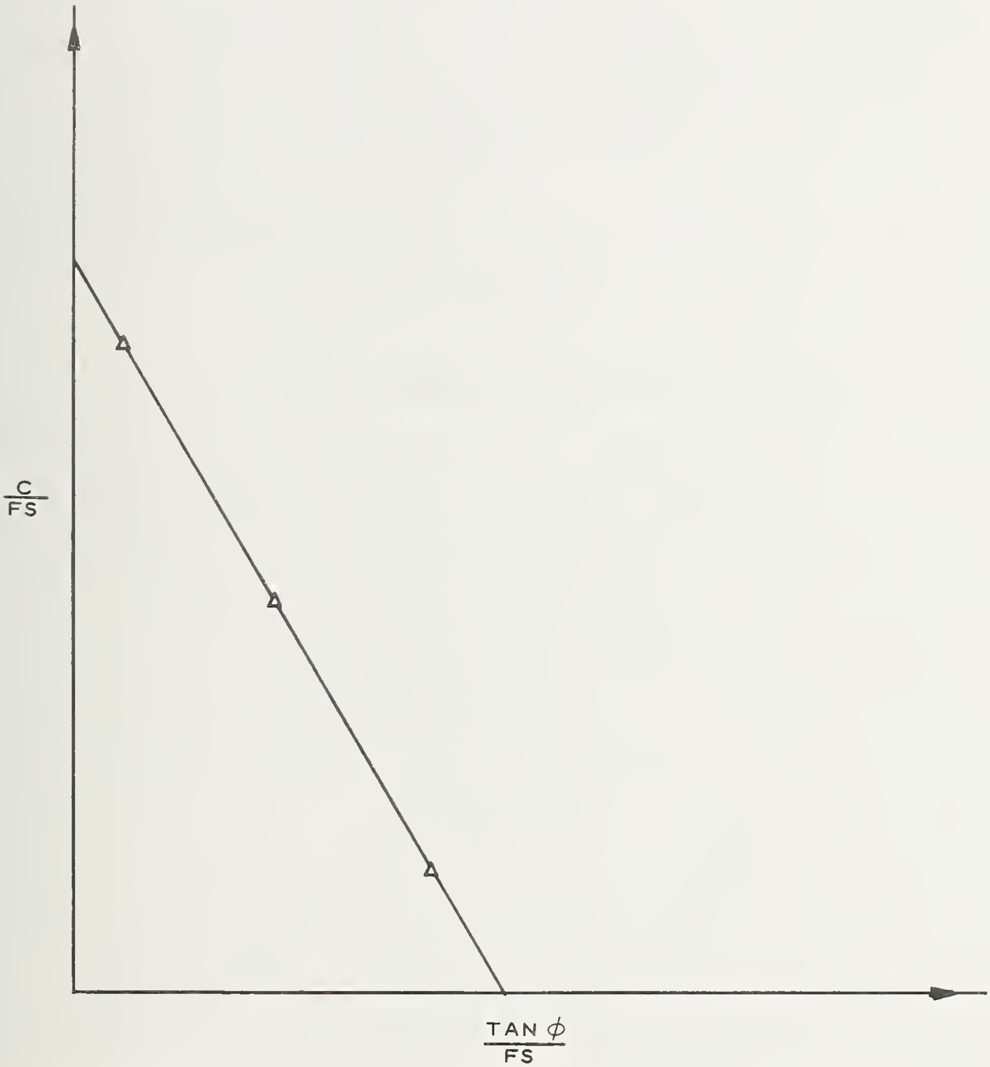

Figure 44. Plot of c/FS versus $\frac{\tan \emptyset}{FS}$ for description of $\emptyset$ and c obtained from back analysis.

employs a computer solution using the Simplified Bishop method of slices. The known slope configuration is used to determine strength properties which will produce a computer solution having a failure surface similar to that observed in the field and a factor of safety close to 1.0. The cohesion is generally assumed equal to 0, and a trial-and-error procedure is used to arrive at the effective friction angle that produces a factor of safety of 1.0. Different water surface elevations are assumed or observed surfaces are used in the analyses. Results of back analyses are generally compared with consolidated-undrained triaxial tests with pore pressures measured. Experience is used in selecting strengths for design of remedial measures based on the laboratory data or results of the back analyses.

In evaluating one of three fills on I-75 in Tennessee, Law Engineering Testing Company (1972)* used a procedure which involved a computer analysis of the slope based on the ground cross sections obtained after a failure. These cross sections identified a probable failure surface which was used in the analyses. Results of the analyses based on total stress indicated a representative strength of 500-psf cohesion with a friction angle of 35 deg. These strength properties were used to predict the future safety of the failed slope and to assess the behavior of the unfailed portion of the slope during remedial repairs. Interestingly, this strength compares with those for unsaturated compacted shale listed in Table 14 of Volume 1 (Shamburger et al., 1975). Undisturbed sampling had been attempted but only poor quality samples were obtained. In situ testing had also been attempted using the Menard pressuremeter but drilling difficulties led to its abandonment. Therefore, back analysis was the only means available for determining the strength properties of the embankment.

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* See footnote on page 10.

IV. REMEDIAL TREATMENT

The following techniques were investigated to determine their applicability in remedial treatment of distressed or failed shale embankments:

- a. Pavement overlay.
- b. Drainage systems.
- c. Slope flattening, berms, and buttresses.
- d. Retaining walls (including reinforced earth).
- e. Lime, cement, and chemical stabilization.
- f. Reconstruction.

Experience with the above methods, their advantages, and disadvantages are discussed in the following paragraphs.

Pavement Overlay

Excessive settlement and cracking of highway pavements is routinely corrected by overlaying the existing pavement with an asphalt mixture. Periodic pavement overlay has been required on many compacted shale embankments. Although the procedures have kept lanes open to traffic in distressed areas, it is recognized that the method may prove only a temporary measure. Experience indicates that settlement of shale embankments is often followed by slope failure as discussed in Part II. For example, several shale embankments along I-75 in Tennessee have experienced slope failure preceded by a 5- to 10-yr period of pavement cracking and subsidence requiring intermittent pavement overlay. Asphalt overlay at sta 950+00 on I-75 in Tennessee is shown in Figure 45. The embankment has not failed although settlements are continuing.

The common persistence and eventual magnification of shale embankment distress indicate the need for early evaluation and treatment of embankment problems. Experience suggests that in many cases pavement overlay should be supplemented by more permanent remedial measures in an effort to prevent complete slope failure and its costly and time-consuming repair.

Figure 45. Pavement overlay, sta 950+00, I-75, Tennessee (courtesy of the Tennessee Department of Transportation).

Drainage Systems

As discussed in Part II, the infiltration of surface and subsurface water can lead to degradation and loss of strength in shale fill. Embankment distress and failure are most prominent during rainy periods when saturation and increased pore water pressures further weaken embankment and foundation materials. Water has been the major cause of slope instability or failure whether occurring in natural, cut, or embankment slopes. Consequently, drainage measures have been employed for many years as both a remedial and a preventive technique (see for example, Ladd, 1928).

Drainage techniques investigated were surface treatment, horizontal drains, vertical drains, pumped wells, trench drains, toe drains, drainage blankets, and bench drains. Although discussed separately, these measures are commonly used in combination and supplement other types of remedial treatment techniques. Combined usage will be illustrated in examples. Drainage systems should be designed not only to provide adequate discharge capacity, but also to ensure filter protection to prevent adjacent soil from piping or clogging the sytem. Theoretical and empirical methods for predicting the effects of individual or combinations of drains are contained in textbooks by Cedergren (1967; 1974) and in the Materials Manual (Volume VI), California Department of Transportation (1973). However, many field applications have been based largely on past experience or trial and error. In general, application of drainage measures involves engineering judgment based largely on experience. Drainage installations should be checked (and altered as necessary) by monitoring drainage discharge rates and changes in groundwater table elevations and/or piezometric heads.

Surface treatment. Infiltration of surface runoff may contribute to gradual shale degradation. In addition, shear stresses will increase as water fills cracks within the slope. Consequently, measures should be applied to prevent surface runoff from entering shale embankments.

Several surface treatment measures can be applied to distressed shale fills. Median and side ditches should be paved and existing paved ditches inspected and repaired as necessary. Curbing can be installed to direct pavement runoff away from the embankment slope. Cracks in the highway pavement or embankment slopes should be sealed. Low areas on the fill slopes should be filled (or graded) to prevent ponding of water. Infiltration on the fill slopes can also be reduced by treating the surface with chemicals (e.g. lime treatment) or sealing the surface with an impervious membrane. Types of membranes used to prevent wetting of expansive soils are reviewed by Snethen et al. (1975). Surface chemical treatment or membranes will also hold moisture in the embankment; thus when they are used on sidehill fills, additional drainage techniques are needed to provide an outlet for subsurface seepage from the foundation into the embankment.

Toe drains. The toe areas of shale embankments are subject to erosion and infiltration by surface runoff, inflow from adjacent streams, and in many cases subsurface seepage from sidehill foundations. These conditions can cause a loss in toe support through rapid shale degradation, piping, and surface sloughing in the toe area. Toe drains, such as those commonly associated with earth dams, can provide some protection. These drains can be constructed adjacent to the toe of the embankment slope or buried as shown in Figures 46 and 47, respectively. The installation shown in Figure 46 is advantageous in remedial treatment since it can be rapidly constructed and does not require significant excavation in the toe area. In remedial treatment of a shale embankment, the excavation required to construct a buried drain as shown in Figure 47 could cause further instability. However, the buried drain would be most appropriate for collecting seepage through sidehill fills and could be applied as a type of preventive technique with construction carried out during a dry season. The Utah State Highway Department has used deep toe drains cut into the natural ground adjacent to embankments constructed of the Mancos shale. Utah State Highway Department soils engineers consider these toe drains to be a primary factor in the satisfactory performance of those embankments.

Horizontal drains. The use of horizontal drains to stabilize highway slopes was initiated by the California Division of Highways in 1939

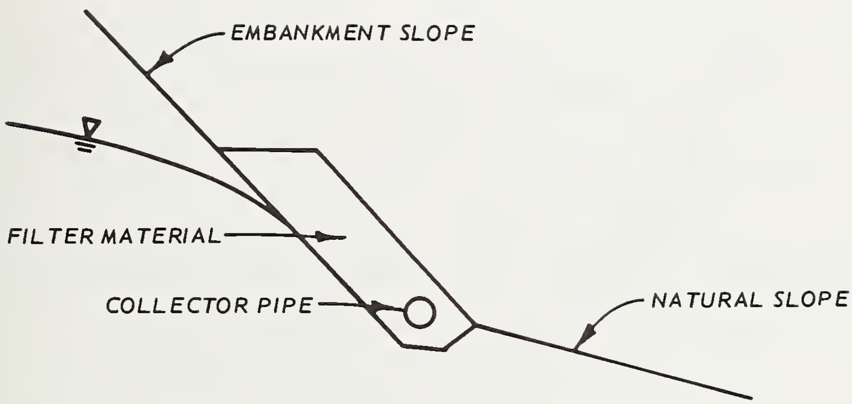

Figure 46. Toe drain adjacent to the slope.

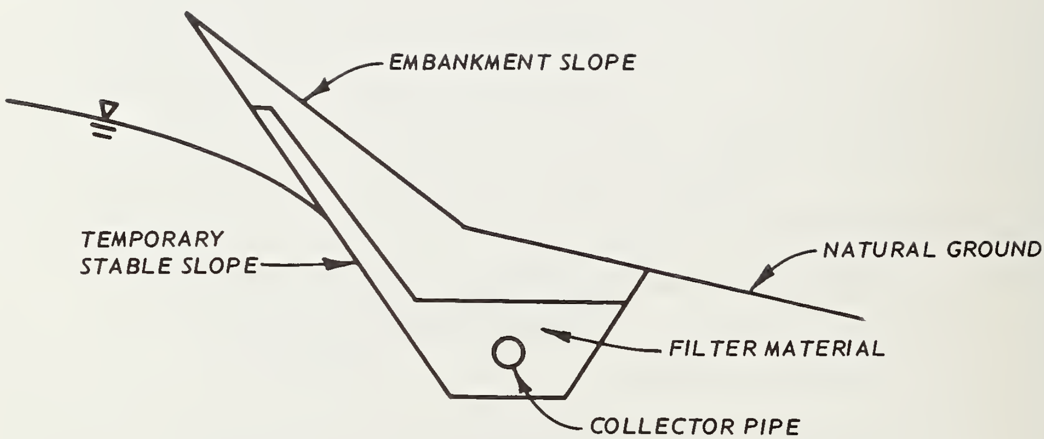

Figure 47. Buried toe drain.

and has since gained much popularity. A horizontal drain consists of a perforated pipe (usually 1-1/2 to 2 in. in diameter) inserted into a borehole inclined upward at a 3 to 20 percent grade to allow gravity drainage. Horizontal drains, often combined with other drainage measures, have successfully stabilized many slopes in California (Smith and Stafford, 1957; Smith, 1962; Smith and Cedergren, 1962). Experience in California has shown that horizontal drains are effective under a variety of soils and geologic, topographic, climatic, and groundwater conditions.

In several cases horizontal drains have been incorporated into remedial treatment of compacted shale embankments. Their purpose has been to intercept subsurface seepage from the natural ground adjacent to sidehill fills and carry the water through the embankment. Horizontal drains in combination with a rock buttress were used to repair three embankment slides along I-75 in Tennessee (Royster, 1973). The original, the failed, and the repaired embankment at one location is shown schematically in Figures 48-50, respectively. Some of the drains were nearly 600 ft in length, and initial flows up to 400 gal/hr were observed. Drains were installed through the lower portion and base of the fill at spacings of 10 or 20 ft. Horizontal drains were also incorporated into repair of one of the three slides along I-74 in Indiana (SLIDE-2, location given in Figures 7 and 8). Ten horizontal drains were used to supplement a large berm and shear key. The drains (from 124 to 223 ft in length) were separated into three groups, each group arranged in a fan pattern and draining into a separate manhole. Discharge from the horizontal drains was approximately 900 gal/hour (Craveiro, 1974).

Craveiro (1974) describes the procedures used to install the horizontal drains in repair of the I-74 fill in Indiana. A variable torque drill was used to drive 10-ft sections of 2-3/4-in.-diam hollow steel drill pipe. A tricone drill bit was used which was cooled by jetting water through the drill pipe. After reaching the desired penetration depth, the drill was reversed to detach the drill bit which remains in the borehole. The drain pipe consisting of 1-1/2-in.-diam perforated

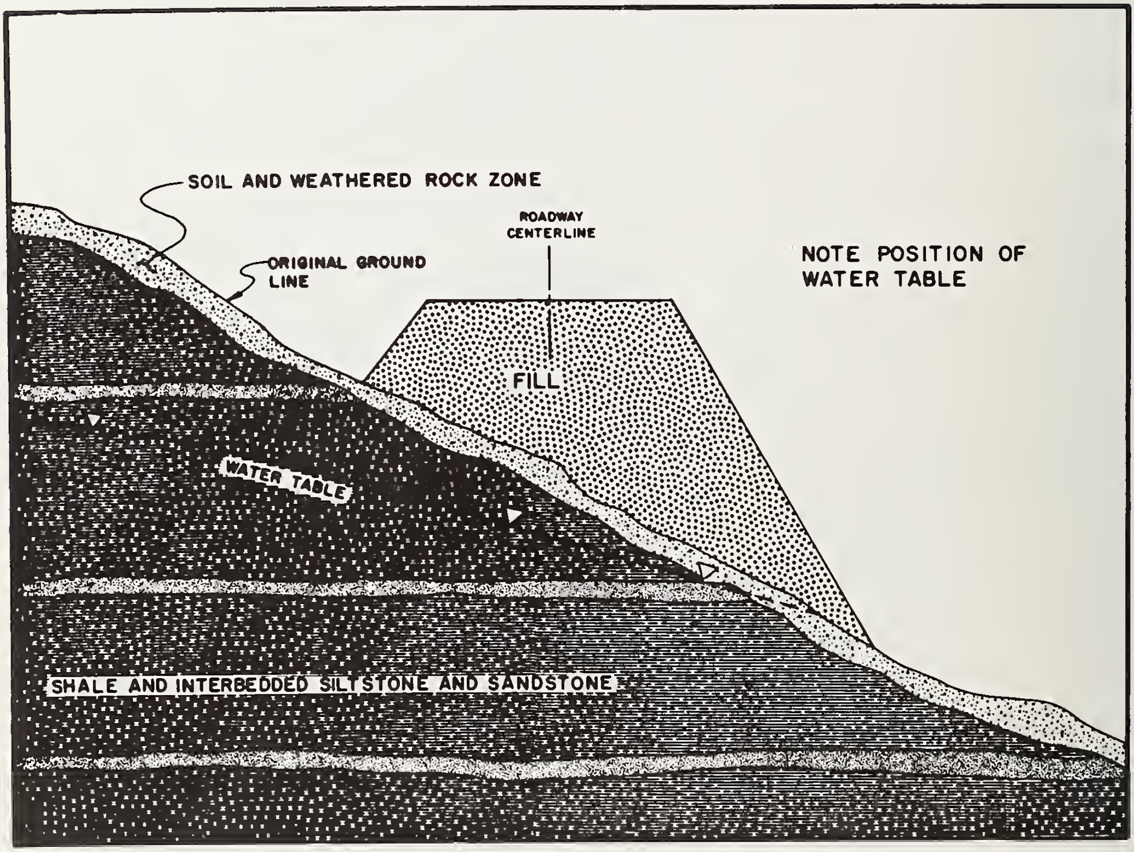

Figure 48. Sta 1464+00, I-75, embankment as constructed (Royster, 1973).

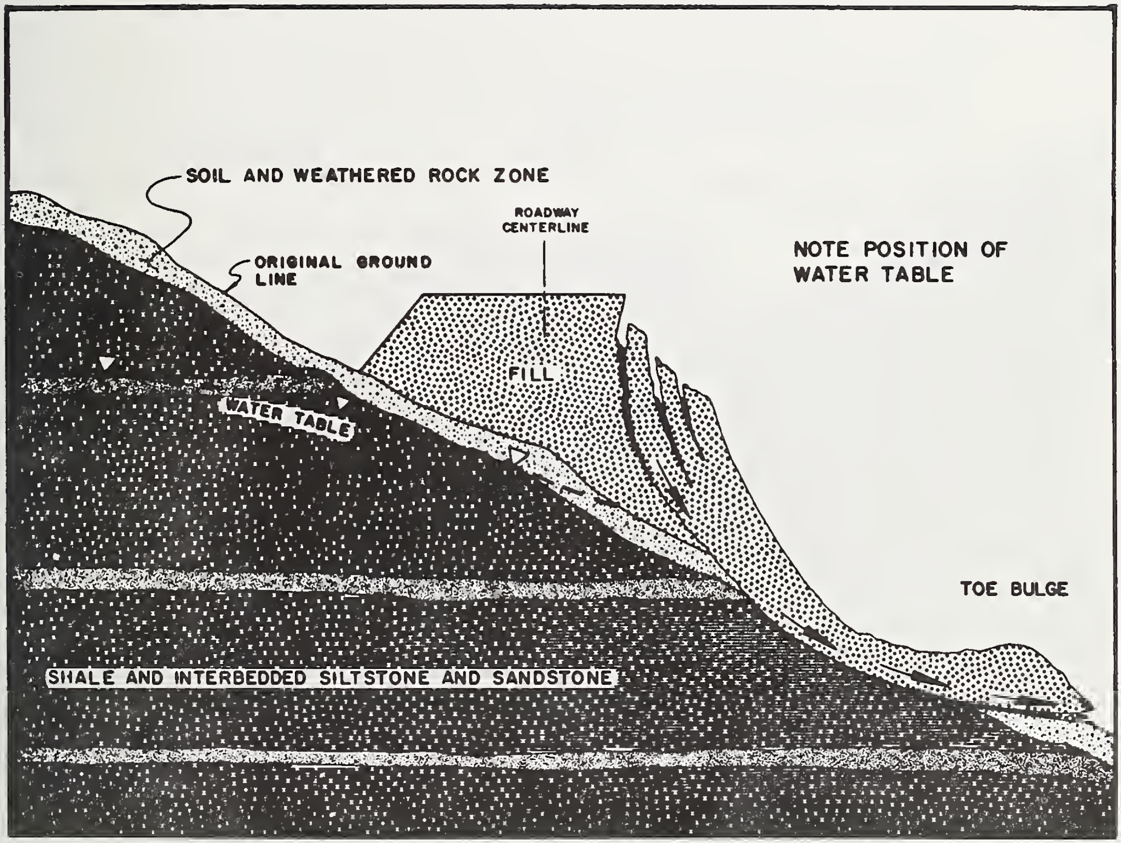

Figure 49. Sta 1464+00, I-75, embankment at failure (Royster, 1973).

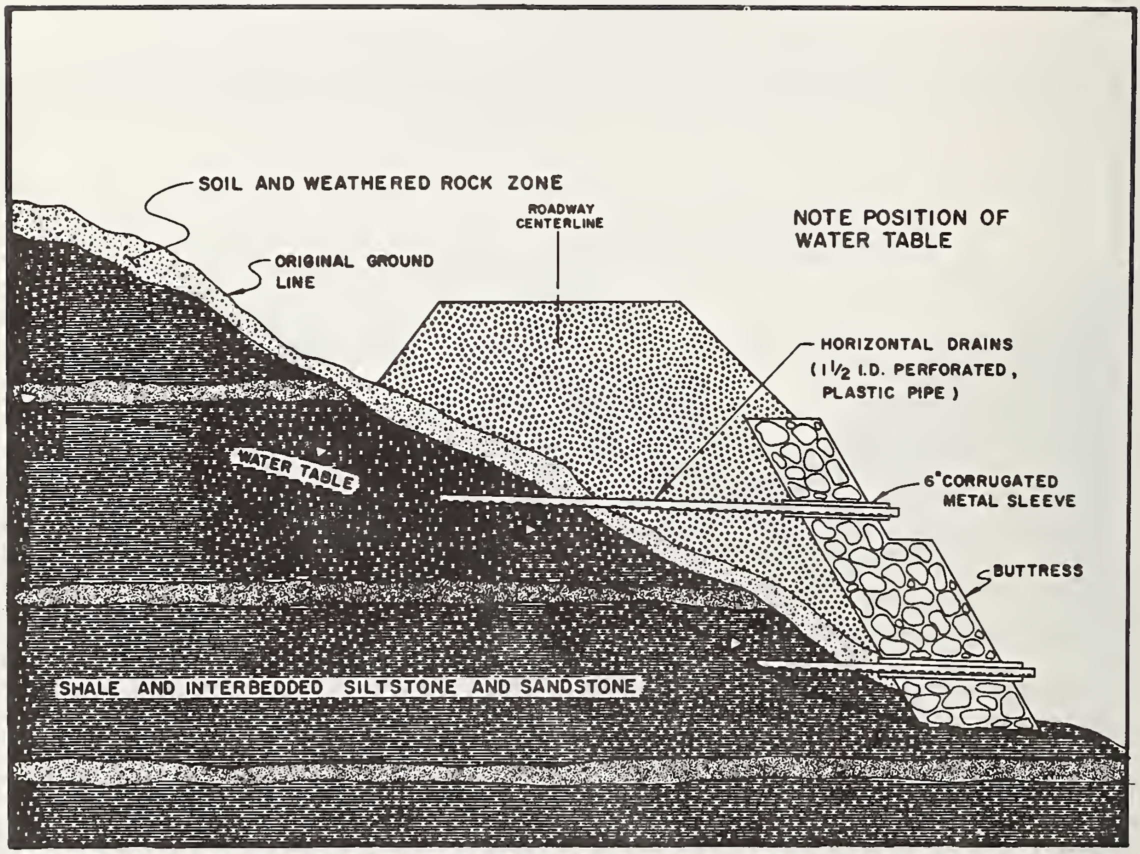

Figure 50. Sta 1464+00, I-75, embankment after repair (Royster, 1973).

PVC pipe was then inserted within the hollow drill pipe. The drill pipe was then removed while holding the PVC pipe in place. This procedure was similar to that used in Tennessee and has the advantage of eliminating the possibility of borehole caving prior to insertion of the drain pipe.

The application of horizontal drains prior to significant deterioration and deformation of the embankment may prevent complete slope failure. Location and number of horizontal drains required will be largely dependent on the groundwater and topographic conditions and the permeability of the subsurface materials. Piezometer data and subsurface investigations including vertical and horizontal borings should be used in defining the groundwater pattern and characteristics of water bearing materials. The horizontal borings can serve as trial horizontal drains to delineate drilling problems and the potential effectiveness of horizontal drains at different locations.

Horizontal drains must be initiated at some point accessible to the drilling equipment and the upper portion of the drain must intercept the subsurface water which is contributing to the unstable condition. Efforts can be made to penetrate along aquifer layers such as sandstones or fractured limestone; however, the locations, orientations, and thicknesses of such layers will likely be variable and difficult to determine. Deviation of the borehole during drilling will also make it difficult to follow a particular aquifer layer.

Horizontal drains are often installed at different elevations. In treatment of sidehill fills, drains could be placed through the fill or in the natural ground above or below the fill depending on the particular groundwater conditions (Figure 51). Experience indicates that in most cases, the drain spacings at any particular elevation should be about 30 ft or less with the closest spacings at the lower elevations such as immediately below the toe of the fill. In areas of rapid groundwater recharge or low permeability, spacings of 10 to 15 ft or less may be required. Some drains may be located entirely within the fill; however, most drains should penetrate deep within the adjacent natural ground to intercept seepage before it reaches the fill.

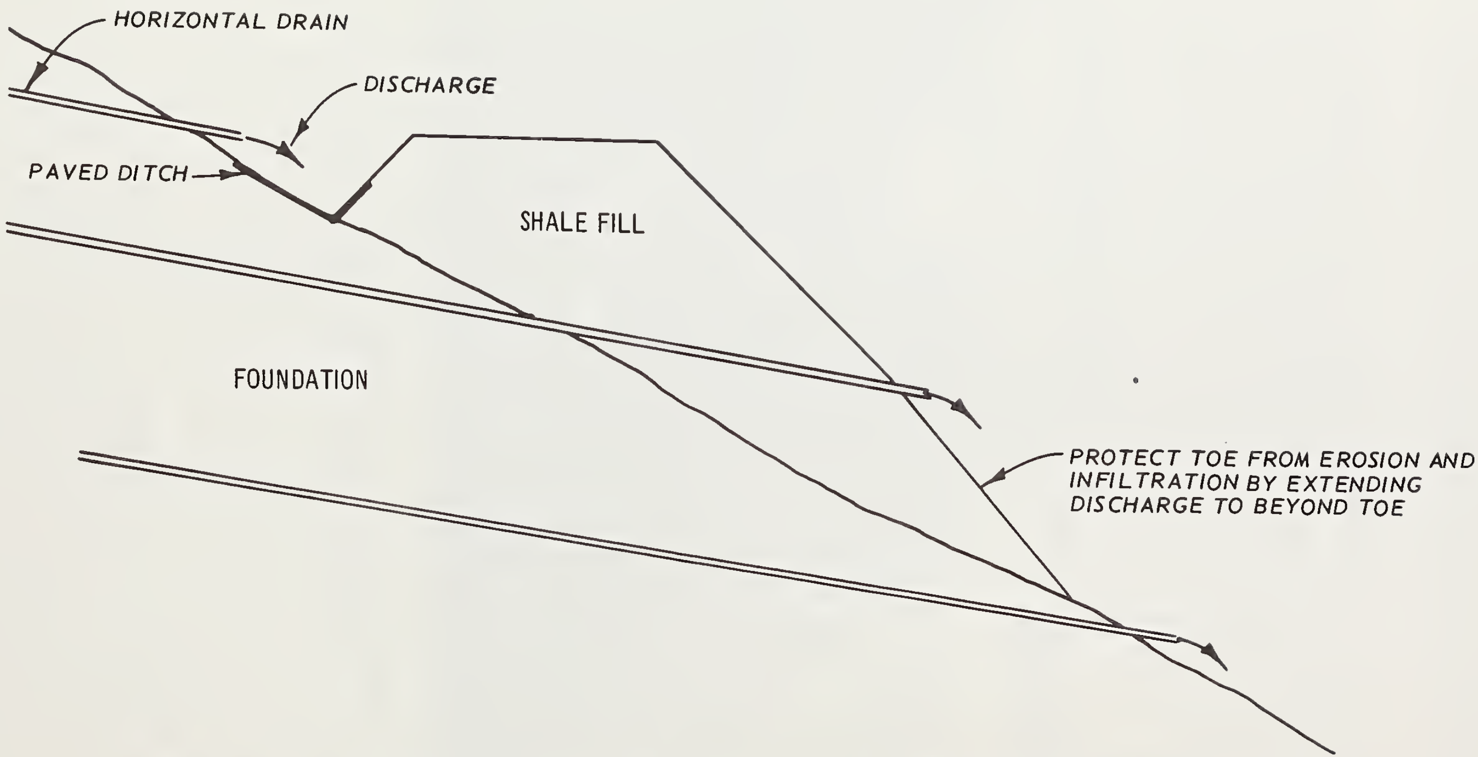

Figure 51. Application of horizontal drains to stabilize sidehill fills.

Horizontal drains (or portions thereof) located within the unstable fill are most susceptible to damage due to ground movements. Damaged drains within the fill could increase instability by allowing seepage to easily penetrate but not pass through the fill. Where feasible, most drains should be located above or below the fill.

Water intercepted by horizontal drains should be discharged away from the distressed fill. Paved ditches or 8- to 12-in.-diam collector pipes should be satisfactory, although both should be periodically inspected and cleared or repaired as necessary. Where freezing temperatures are encountered, collector systems should be buried to prevent ice blockage near the outlets.

Horizontal drains are not without problems. Horizontal drains often perform poorly in low permeability materials and/or areas with irregular seepage patterns (see for example, Montana State Highway Commission report: Williams and Clark, no date). This problem is expected since these drains must intersect dominant seepage zones and are dependent on gravity drainage. Horizontal drains are also susceptible to blockage caused by penetration of roots or infiltration of fine soils. Easy access to the drains will allow for periodic maintenance. Horizontal drains should be inspected routinely and cleared to maintain their free draining capability. California experience has shown that maintenance requirements can be greatly reduced by using nonperforated galvanized pipe to a distance of 20 ft from the drain exit (Smith and Stafford, 1957). This procedure minimizes blockage caused by root growth through the perforations. Smith and Stafford (1957) suggest that horizontal drains be thoroughly cleaned every 3 to 10 yr by water jetting and penetrating with a drill bit small enough to rotate freely inside the drain. Blockage of horizontal drains by infiltration of surrounding soil may be decreased by wrapping the perforated pipes with filter cloth before installation. Types of filter cloth and their general applications are discussed by Calhoun (1972) and are presented in Engineer Manual EM 1110-2-1609 (USAE, OCE, 1973a).

Vertical drains. Vertical drains offer the advantage of intercepting flow from water-bearing strata which are separated by impervious layers. Installation of vertical drains is relatively simple and is not likely to cause further instability since there is no lengthy, deep, open excavation involved. Such wells may be ideal in sidehill fill foundations. Seepage from interbedded shales, limestones, and sandstones can be collected in large-diameter vertical drains prior to entering the shale embankment. In California practice, vertical drains are drilled to 3 ft in diameter, spaced on 5-ft centers, and interconnected by belling out the bottom of each well.* In certain cases, hand mining has been used to interconnect vertical drains. The interconnected drains are filled with permeable material to form a continuous curtain to intercept groundwater. Collected groundwater is then removed through a system of horizontal drains. On the average, one connecting horizontal drain has been used for every 10 interconnected vertical drains. The horizontal drains are drilled toward the belled out zones between the drains. The drain curtains have been successfully intersected with horizontal drains drilled up to distances of 600 to 700 ft.

In some cases, it may not be practical to interconnect vertical drains. In such instances, horizontal drains or large-diameter pipe drains can be used to intersect and drain each well individually as shown in Figures 52 and 53. However, horizontal borings are likely to deviate during drilling making it difficult to intersect individual vertical drains over long distances. Ray Forsyth* indicated that the California Department of Transportation has not attempted to intercept individual vertical drains with horizontal drains and that borehole drift would make this procedure extremely difficult at distances much greater than 100 ft.

To intercept vertical drains with horizontal drains, it would be advantageous to survey the horizontal drain borehole during drilling and make orientation corrections as necessary. A brief check was made to determine if there existed commercially available equipment capable of surveying horizontal drain boreholes to a distance of 100 to 200 ft.

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* Ray Forsyth (1975), Personal communication, Transportation Laboratory of the California Department of Transportation.

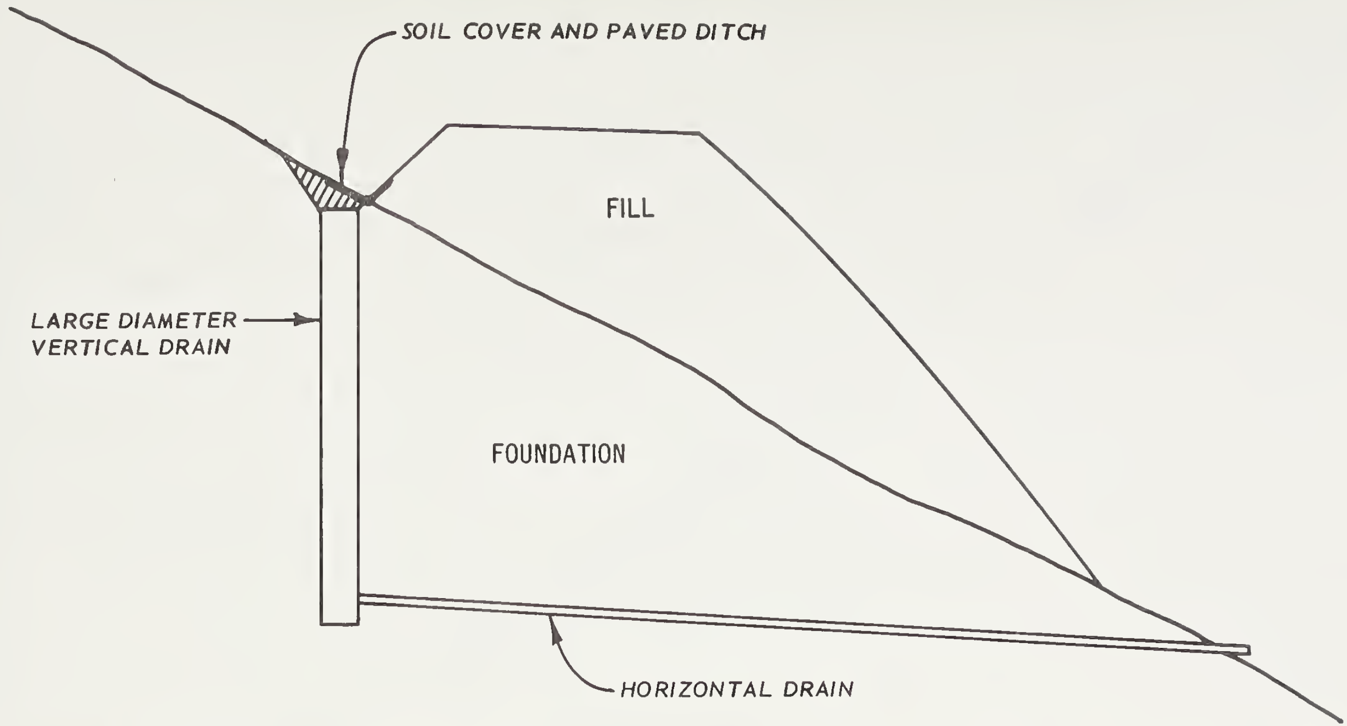

Figure 52. Application of large-diameter vertical drains to stabilize sidehill fills.

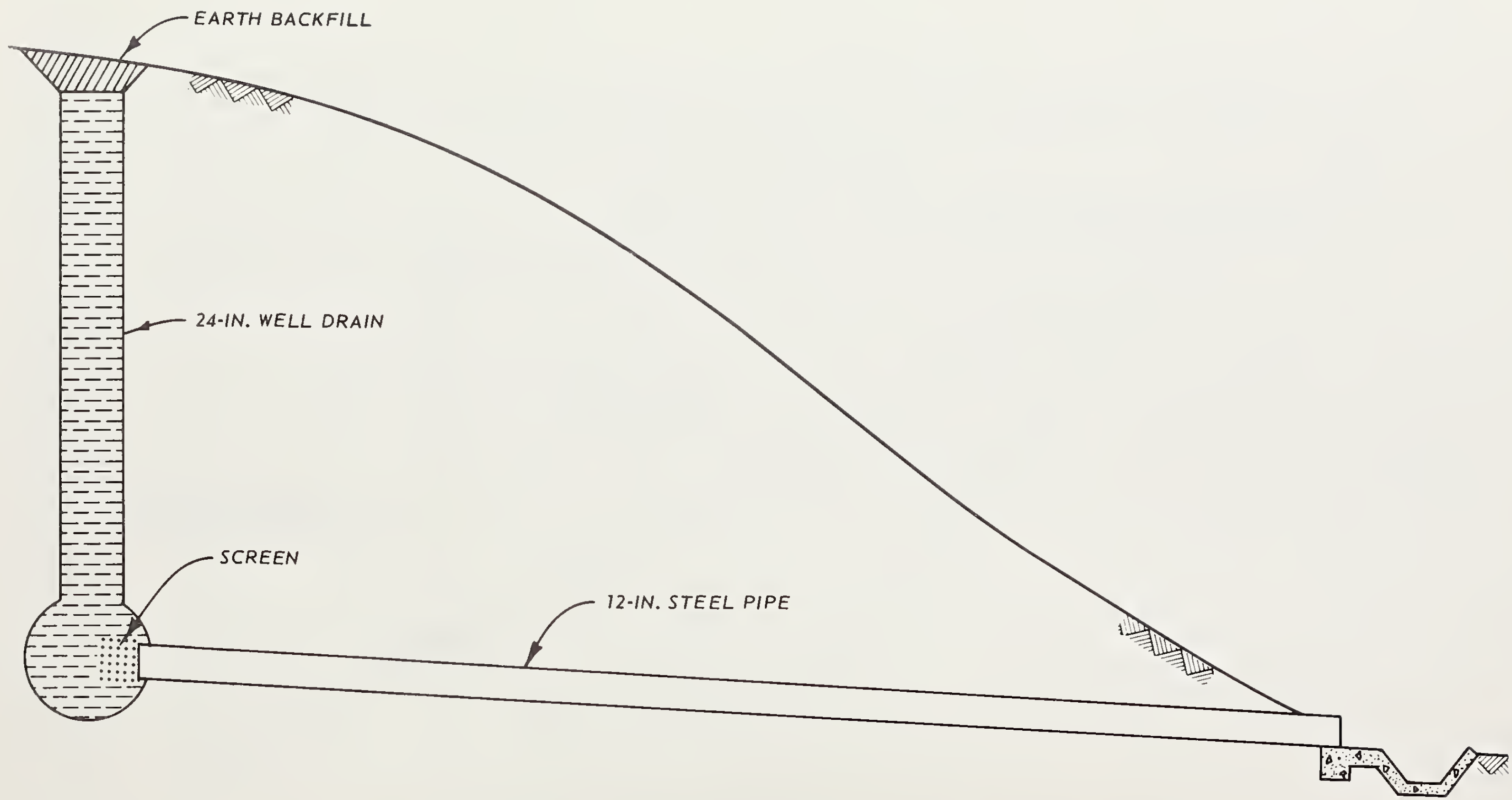

Figure 53. Well drains used to stabilize landslides in glacial till (after Palmer et al, 1950).

There is an instrument available which can be used to survey an existing PVC lined (but not steel lined) horizontal drain hole with a minimum diameter of 1-1/2 in.* This capability is advantageous in that horizontal drains could be installed on a trial basis. If the horizontal drains were inadequate because thin water bearing layers separated by impervious materials were not intercepted, the existing horizontal drains could be surveyed and vertical drains installed to intersect them. Field experience is needed to better define the applicability and practicality of surveying horizontal drain holes.

Vertical drains can also be used to drain surface water. For example, short sections of vertical drains spaced intermittently along the shoulder may be necessary to remove pavement runoff where curbing is used, especially along lengthy, gradual grades. When vertical drains are used in connection with shale fills, collected surface water should not be allowed to infiltrate the embankment but must be removed through pipe underdrains. Design of vertical drains for collection of highway surface water is discussed in detail by Smith et al. (1969).

Pumped wells. Rapid removal of subsurface water may best be achieved by using vertical pumped wells. These wells can be installed on a permanent basis and activated during rainy periods or applied as a temporary measure to maintain stability while applying other remedial measures. Guidance for the planning, design, construction, and operation of pumped wells is contained in Technical Manual TM 5-818-5, Departments of the Army, Navy, and Air Force (1971).

Vertical pumped wells (100 ft deep and equipped with automatically actuating pumps) were applied in remedial treatment of the embankment slide at sta 840+00, I-75, Tennessee (failed slope shown in Figure 2). The wells aided in providing temporary stability of the northbound lanes during reconstruction of the failed slope (Royster, 1973). These wells penetrated the sidehill foundation and discharged into the median ditch as shown in Figures 5 and 6. Discharge into the median or anywhere on a

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* Ed Hern (1975), Personal communication, Eastman Whipstock, Inc., Houston, Tex.

fill should be avoided unless it is certain that the water will not reenter the embankment.

Trench drains. Trench drains are commonly used to intercept subsurface seepage. These drains are usually constructed parallel to the slope crest and in such instances are termed interceptor trenches. In construction, perforated pipe is commonly laid along the trench bottom prior to backfilling with a filter material. In natural slopes, interceptor trenches are graded to outlets sometimes consisting of intermittently spaced transverse trench drains commonly oriented perpendicular to the slope crest. These drains have been successful in stabilization of clay slopes. For example, Abrams and Wright (1972) describe a case in which a clay slope was stabilized by constructing several rows of interceptor trenches parallel to the slope crest. Each trench was approximately 6 ft deep and 2-1/2 ft wide. Water was drained from the interceptor trenches through several transverse trench drains.

Interceptor trenches have been used in attempts to stabilize sidehill shale fills; however, in most cases instability had been attributed to foundation movements. The trenches were located within the natural slope near road level to intercept subsurface seepage into the embankment and residual soil foundation as shown in Figure 54. These drains were generally ineffective probably due to their shallow depths (generally 10 ft or less). Depth of trenching is limited by the need to insure stability of the trench walls which, for sidehill fills, include the natural slope above the embankment. To reach sufficient depths, excavation may require the removal of hard rock beneath the natural slope.

Shallow trench drains constructed on the shale embankment slope (either parallel or perpendicular to the slope crest) may effectively reduce infiltration of surface runoff. However, in the case of sidehill fills, the drains may not adequately lower the water table within the embankment. Deep trench drains are likely to be required and their installation would involve considerable amounts of excavation and embankment reconstruction. The California Department of Transportation (1973) recommends that deep trench drains be a minimum of 8 to 12 ft wide at

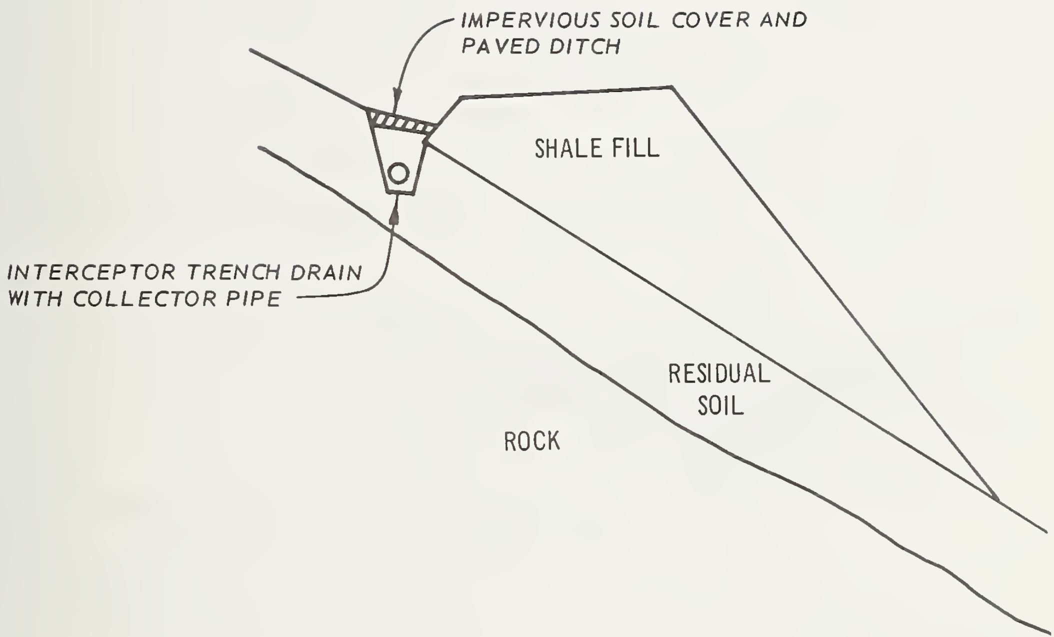
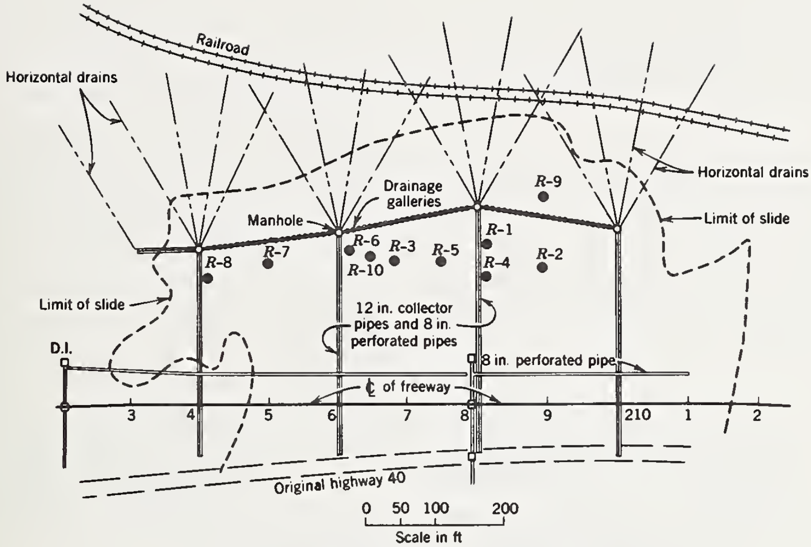


Figure 54. Interceptor trench used to stabilize shale fill.

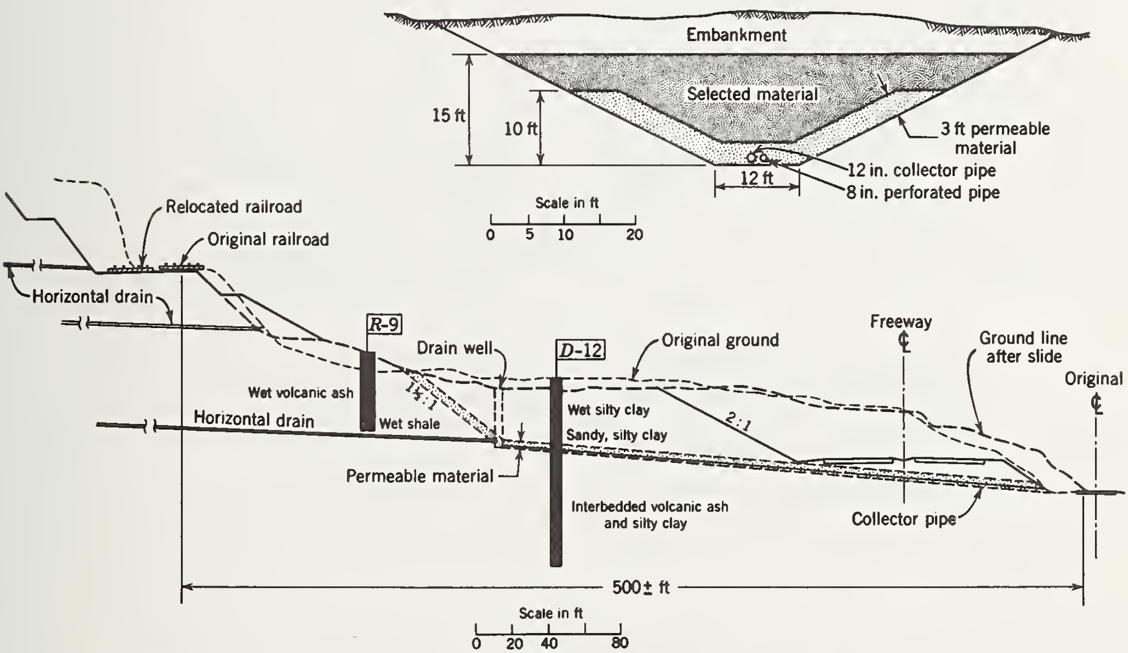
the bottom with side slopes as steep as can be maintained during excavation and backfilling operations, and a blanket of permeable material (minimum thickness of 3 ft) should be placed adjacent to the trench bottom and walls.

Construction of deep trench drains can be extremely difficult in weak unstable materials. Cedergren (1967) describes a case in which pumped vertical wells were required to stabilize the slide material to allow safe construction of the trench drains. The overall drainage scheme that stabilized the slide was elaborate. Horizontal drains, vertical drains, and deep transverse trench drains constructed perpendicular to the slope were used as shown in Figure 55a. Horizontal drains were installed in a fan pattern extending from the upper end of each of the transverse trench drains. The vertical drain wells (30 in. in diameter) were drilled on 8-ft centers and interconnected at the bottom to form a continuous drainage curtain. Four transverse trench drains averaging 30 ft in depth and 12 ft in width at the bottom were constructed at 200-ft intervals. A 3-ft-thick blanket of permeable material was placed along the trench bottom and wall slopes (Figure 55b).

Trench drains are most conveniently installed prior to initial construction of the embankment. The example in Figure 56 shows an interceptor trench and connecting transverse trench to be constructed under a proposed sidehill embankment (Root, 1958). As shown in Figure 56, weak soils should be stripped before construction of the trench drains to provide a firm foundation and direct contact with aquifer layers. To avoid total reconstruction of a lengthy embankment, deep transverse trench drains can be spaced intermittently along the length of the embankment or shear trenches can be used. Shear trenches oriented parallel to the slope crest and commonly used beneath berms (Figure 64, page 140) or incorporated into embankment slopes (Figure 84, page 189) serve as interceptor trench drains to facilitate drainage of sidehill fills and foundations. As shown in Figures 64 and 84, a perforated pipe should be placed at the inside toe of the trench to collect water and carry it to the trench outlets. Although the trench backfill normally consists of coarse, nondegradable rock fill, a drainage blanket



a. PLAN OF DRAINAGE MEASURES



b. PROFILE AND TYPICAL SECTION OF DEEP TRAVERSE TRENCH DRAIN.

Figure 55. Remedial treatment of a slide near Towle, California (Cedergren, 1967).

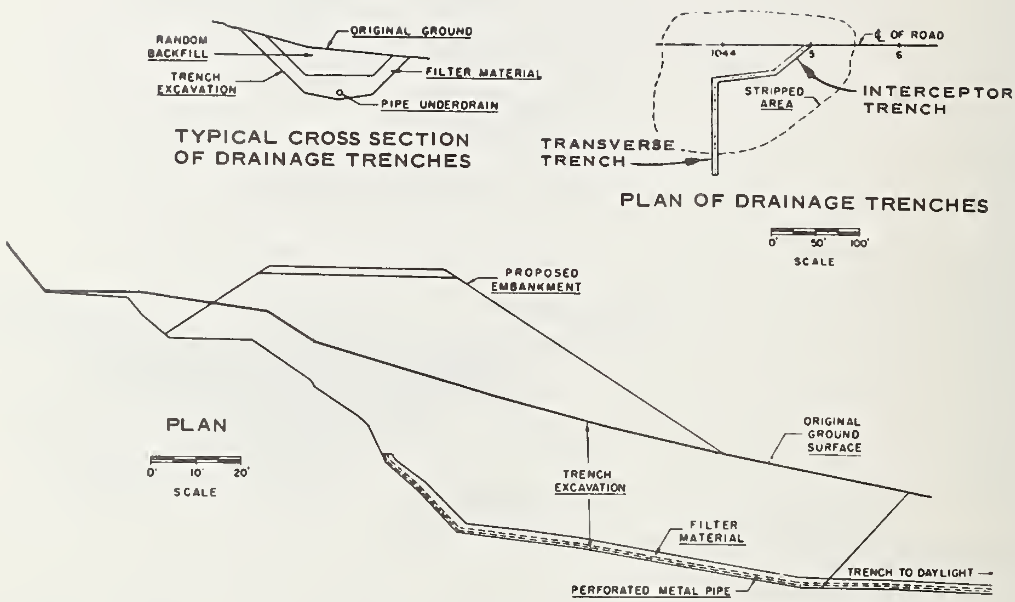


Figure 56. Plan and cross section of preventive treatment consisting of stripping unsuitable soil and constructing drainage trenches (Root, 1958).

(minimum thickness of 3 ft) should be placed along the trench bottom and backslope to prevent future clogging of the backfill and/or collector pipe. Shear trenches are normally cut into firm foundation materials. This is particularly important in preventing differential settlements which may damage collector pipes.

Drainage blankets. A drainage blanket is commonly used to collect subsurface seepage beneath a sidehill fill. The drainage blanket usually consists of a layer of coarse, free-draining material placed along the embankment foundation contact. If large flows are expected (e.g. from natural springs), a drainpipe may be included within the blanket. Root (1958) recommends that weak foundation soils be removed and the blanket be placed on a firm foundation as shown in Figure 57.

The use of a drainage blanket in remedial treatment generally requires that a portion of the unstable slope material be removed. A 3-ft-thick drainage blanket was placed beneath a sidehill fill constructed in repair of a landslide which occurred in 1955 near Baxter, California (Smith, 1962). In this case, the slide material consisted of a saturated mass of tuffaceous silty clay overlying serpentine. Horizontal drains were installed above and below the failed slope to provide temporary stability during reconstruction. Horizontal drains at road level remained as part of the final correction scheme. Weak surface material was stripped to depths of 30 ft prior to placement of the drainage blanket and collector pipe shown in Figure 58.

Several cases were reported in which remedial treatment of shale fills, consisting of the removal of unstable materials followed by placement of a drainage blanket and reconstruction at the original slope angle, had not been adequate. This inadequacy may have been caused by insufficient blanket depth, thickness, and/or clogging of the drains with infiltrated fines from the foundation and embankment. In past practice blankets were often 1 ft thick, placed at shallow depths, and filter layers were not provided. Drainage blankets should be placed directly adjacent to aquifer layers. This placement generally requires that residual foundation soils be removed along with the failed embankment section. It is also important to provide filter layers adjacent to

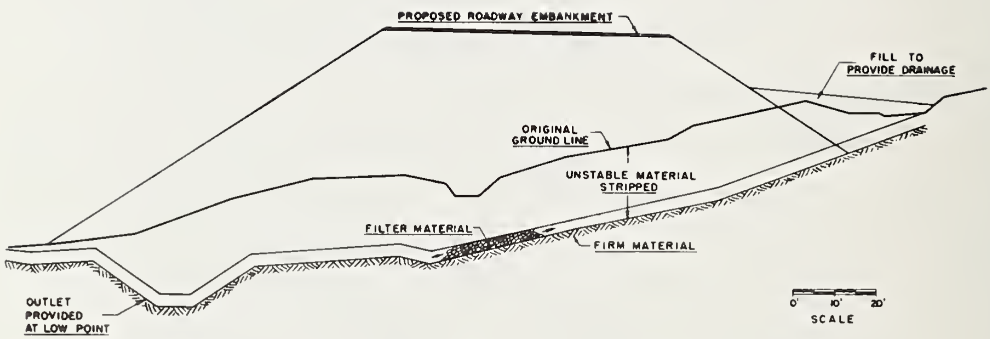


Figure 57. Typical cross section of Redwood Highway in Humboldt County, California, showing stripping of unstable material and placement of drainage blanket before constructing embankment (Root, 1958).

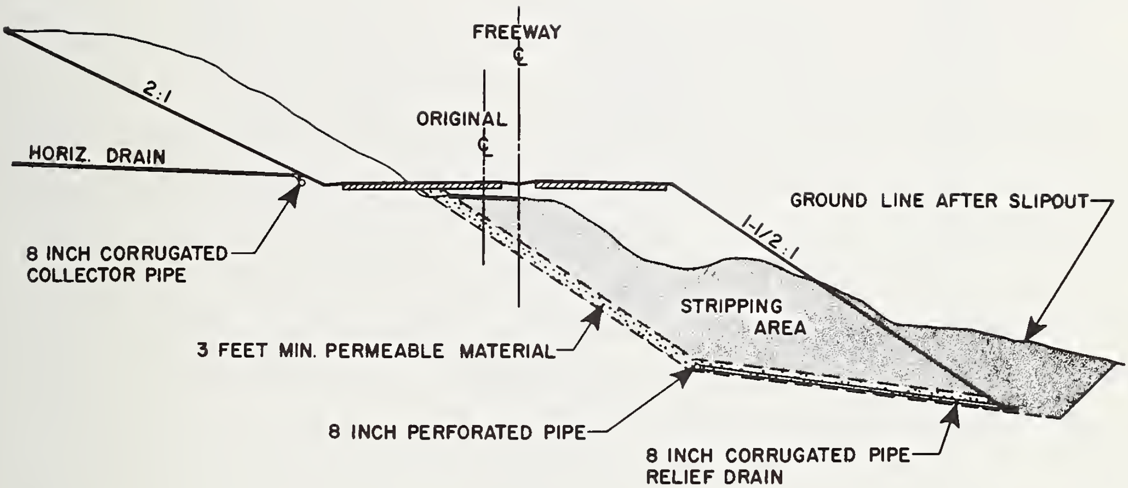


Figure 58. Drainage blanket used beneath a sidehill fill constructed as part of a remedial treatment of a landslide near Baxter, California (Smith, 1962).

the embankment and foundation to ensure long-term operation of the drain.

Drainage blankets have been used successfully when placed directly on the failed slope and overlain by a berm. Stabilizing berms and drainage blankets used in repair of SLIDES-1 and -2 on I-74, Indiana, are shown in Figures 59 and 60, respectively. The location of the ground line after failure indicates that only a small portion of the original embankment material was excavated prior to construction of the drainage blankets and berms. It is intended that the drainage blankets prevent subsurface seepage from the existing embankment into the newly constructed berm.

Bench drains. Remedial treatments involving reconstruction of shale fills should include removal of unstable materials (including residual soils) and benching into the underlying rock formation. Bench drains in the form of partial or fully covering drainage blankets should be placed along the benches. West Virginia utilizes a fully covering blanket as shown in Figure 61. It is advantageous to keep the blanket on the rock line as far up the slope as possible without exposing the drain to excess penetration by surface runoff. The collection of surface water by drainage blankets should be avoided since any problems with the drain will be greatly magnified by the additional volume of water. If drainage blankets must be used for drainage of surface water, special care must be exercised in design to provide sufficient capacity and protection from clogging with fines washed from the adjacent slope.

In repair of SLIDE-3 on I-74, Indiana, unstable fill material and residual soils were completely removed and the embankment reconstructed with a flatter slope. Drainage blankets and collector pipes were placed on the foundation bench slopes as shown in Figure 62. Where feasible, the blanket drain should extend across the benches to avoid seepage along the fill-foundation contact.

Slope Flattening, Berms, and Buttresses

Flattening of embankment slopes often with the addition of a soil or rock berm (or buttress) has been a common remedial technique applied

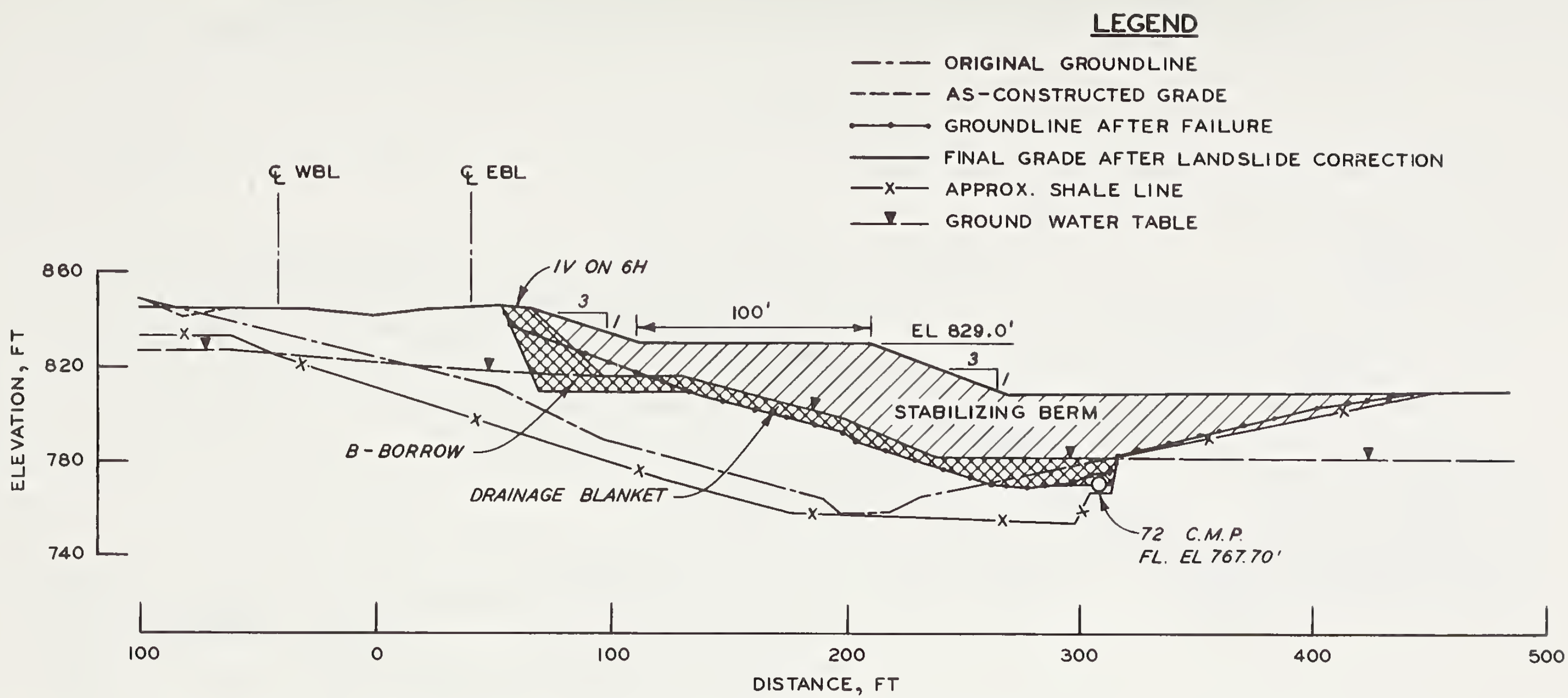


Figure 59. Remedial treatment of SLIDE-1, I-74, Indiana. Location shown in Figures 7 and 8 (courtesy of the Indiana State Highway Commission).

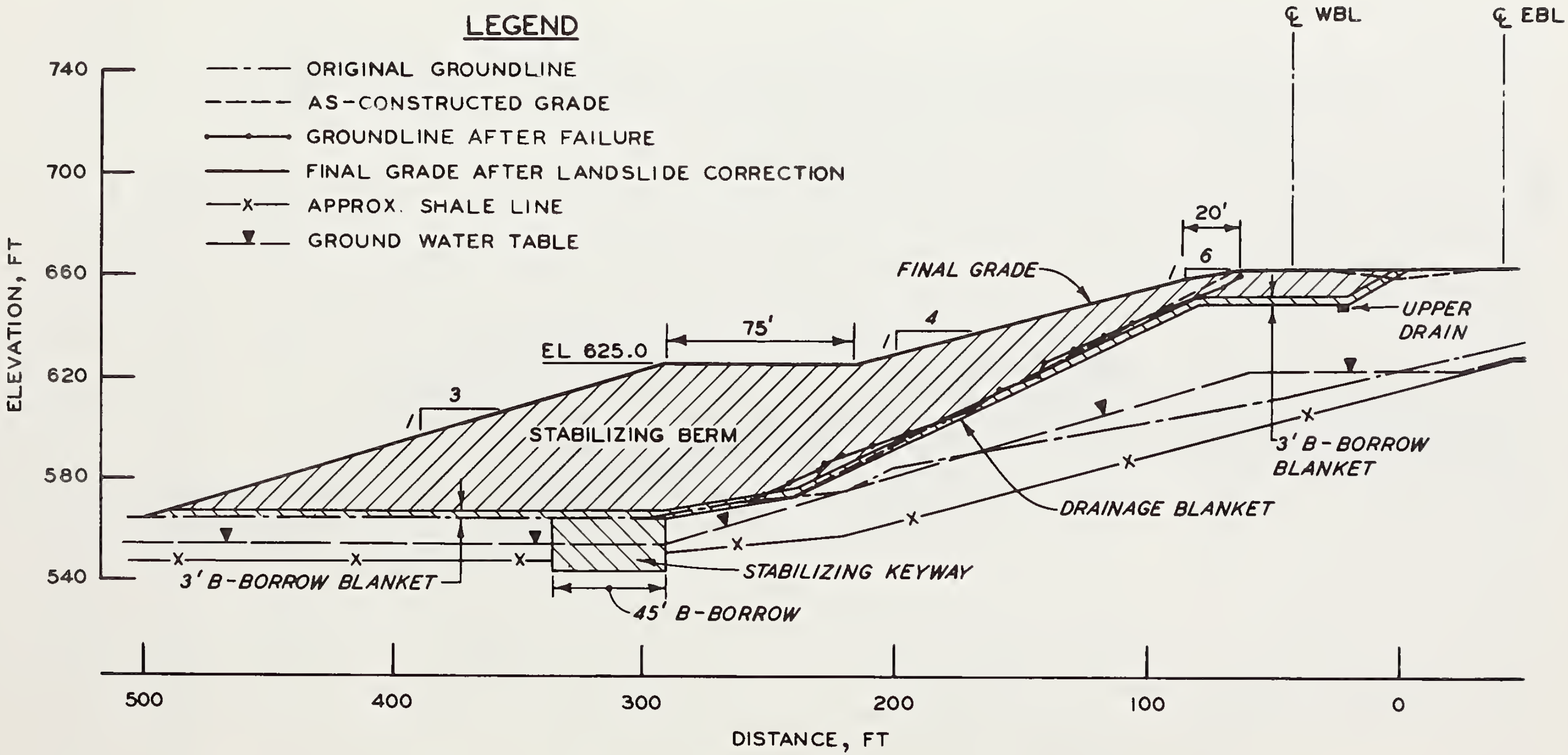


Figure 60. Remedial treatment of SLIDE-2, I-74, Indiana. Location shown in Figures 7 and 8 (courtesy of the Indiana State Highway Commission).

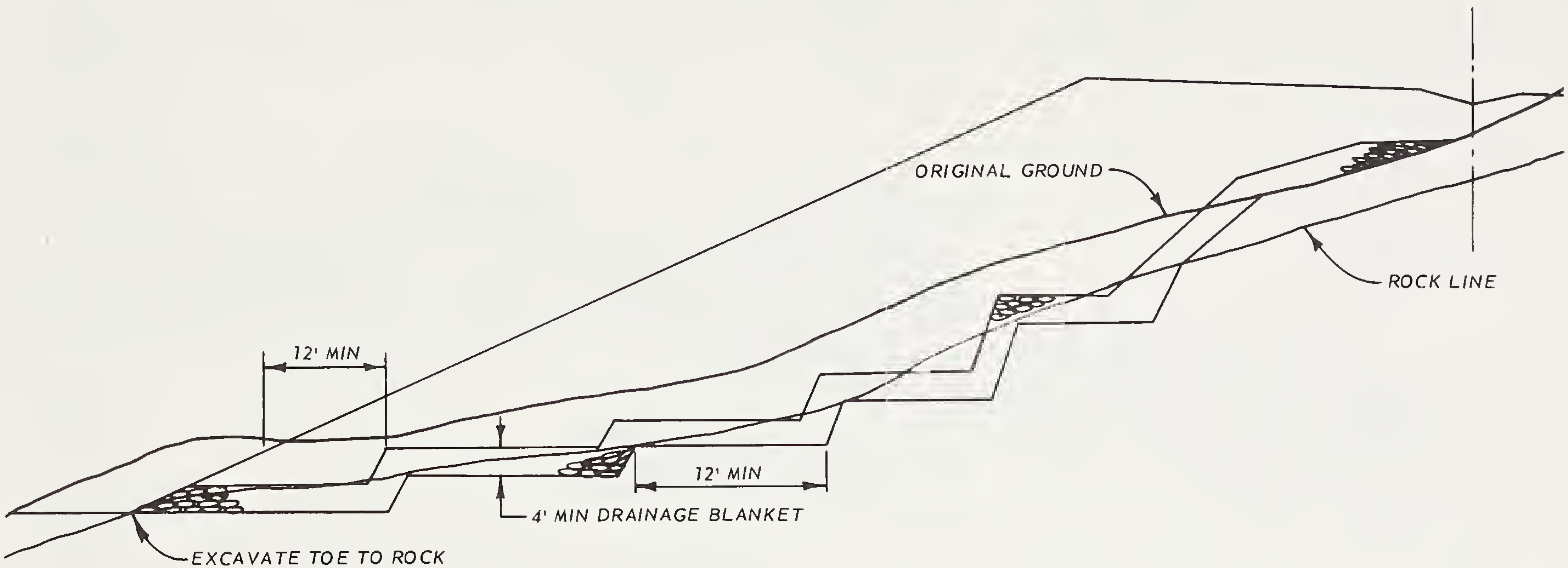


Figure 61. Drainage blanket on foundation benches as used in West Virginia (courtesy of the Department of Highways, West Virginia).

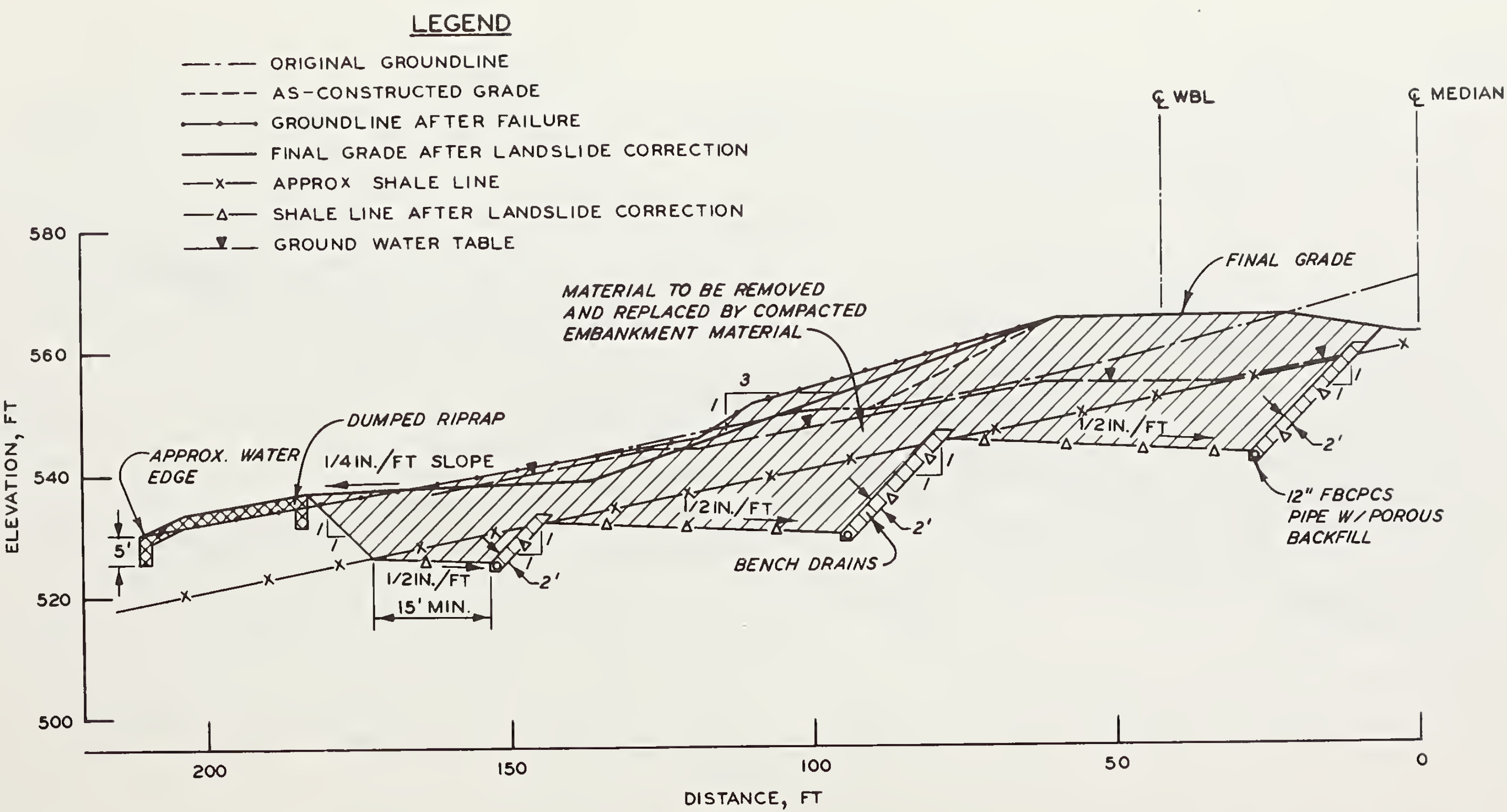


Figure 62. Remedial treatment of SLIDE-3, I-74, Indiana. Location shown in Figures 7 and 8 (courtesy of the Indiana State Highway Commission).

to control shear failure of shale fills. Shale embankment slopes have normally been flattened by placing fill on the distressed slope as shown in Figure 63.* This method provides additional resisting shear forces due to the added weight on the lower portion of the embankment. However, some of its effect is lost as the fill also adds driving shear force to the upper portion of the slope. Consequently, the addition of a berm constructed on the lower portion of the embankment is generally more advantageous than an overall slope flattening. If the same material is used in flattening the slopes and constructing the berm, the entire fill is sometimes referred to as a stabilizing berm (Figure 60). Design of stabilizing berms and buttresses should be based on experience and slope stability analyses (discussed in Part III) in which existing and potential failure surfaces can pass through or beneath the berm fill.

Slope flattening and berms generally require the acquisition of large amounts of additional borrow material and right-of-way. For example, the stabilizing berm shown in Figure 60 is extended to a distance of 260 ft beyond the toe of the original slope. Slope flattening and berms may not be feasible on sidehill fills unless the natural ground is relatively flat near the toe of the unstable embankment slope. In some cases, berms could be extended from the embankment slope to an opposing hillside. In these instances or any situation where natural drainage ditches or small streams paralleling the embankment are blocked, drainage structures will be required (for example, see Figure 59).

Berm (or embankment) slopes adjacent to rivers and streams must be protected from erosion due to currents and wave action. These slopes may also be subjected to submergence and subsequent drawdown. These conditions should be considered in stability analyses. If necessary, these portions of berms (or embankment) slopes which will be submerged should be constructed of a nondegradable free-draining rock fill.

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* Slope flattening can also be accomplished by excavating the slopes and embankment crest. This procedure has been limited to only a few cases since it generally requires shifting the roadway alignment into the median or a cut slope and complete reconstruction of the roadway.

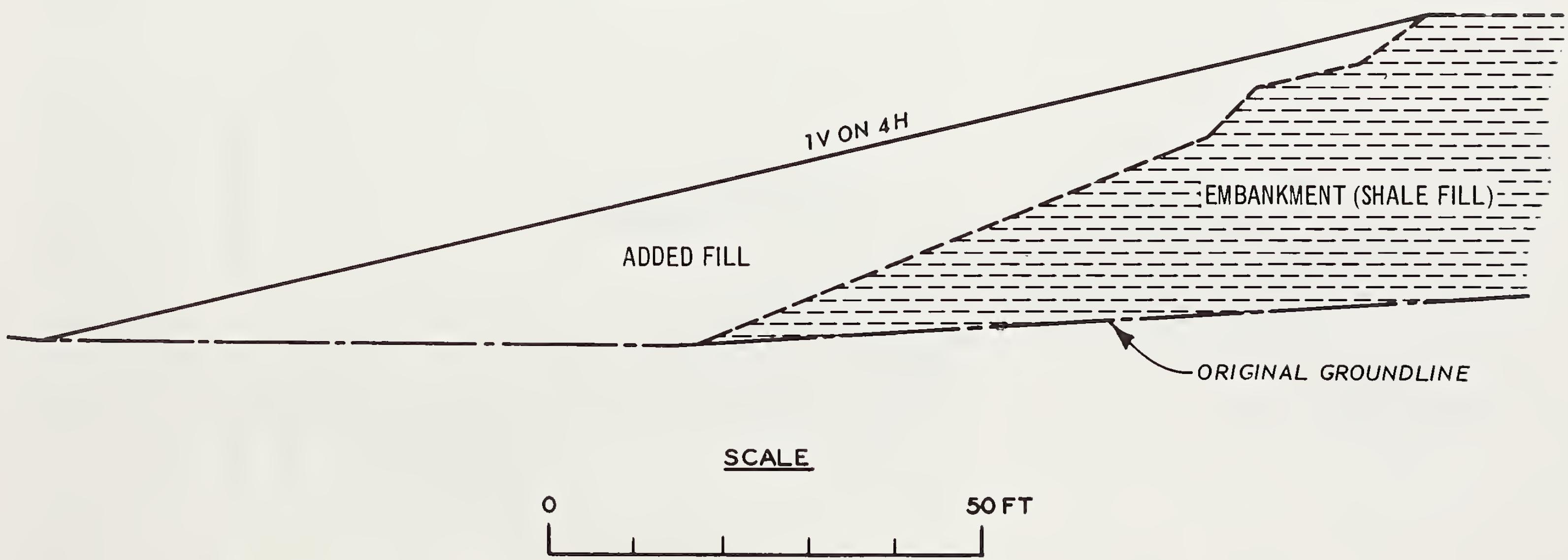


Figure 63. Recommended slope flattening to correct creep movements within shale fill at milepost 179.9 (SBL), sta 693+50, I-75, Kentucky (courtesy of the Kentucky Department of Transportation).

Berms may not be practicable where natural flow channels are significantly constricted. In one case, an alternative to berms considered for remedial treatment of two facing bridge abutments included flattening the abutment slopes by excavation. This treatment was to require the extension of two twin bridges and construction of piers at the present abutment locations (Hopkins, 1973; Hopkins and Yoder, 1973). Bid estimates were in excess of $1 million; consequently, other alternatives are presently being sought.

Berm length can be decreased somewhat by using a retaining wall at the end of the berm as discussed in the next section; however, in many cases the berm is extended well beyond the toe of the embankment to counteract a known or potential shear failure within the foundation. Berms are often supplemented with shear keys (or shear trenches) to prevent foundation movements as shown in Figures 60 and 64. The trench is keyed into firm foundation materials and normally filled with coarse nondegradable rock fill. The shear trench should be designed to serve as an interceptor trench drain. A drainage blanket (minimum thickness of 3 ft) should be constructed along the trench bottom and backslope to prevent future clogging of the backfill and/or collector pipe (see discussion, page 126).

Berm (or buttress) fills should be constructed of coarse, nondegradable rock fill whenever feasible. The free-draining rock fill will enhance drainage of the embankment and foundation, especially when accompanied by a similarly constructed shear trench as shown in Figure 64. Clay soils or shale materials should be compacted in thin lifts when placed as berm fills. Benching, moisture control, and special efforts to break down shale pieces may be required to achieve proper compaction. Drainage blankets will also be required to promote subsurface drainage and prevent buildup of pore water pressures within the berm, original embankment, and foundation. The stabilizing berm used in remedial treatment of the I-74, Indiana, embankments (Figures 60 and 61) were constructed of shale materials broken down and compacted in thin lifts. A similar treatment shown in Figure 65 was recommended for repair of an embankment at milepost 92 along the Western Kentucky Parkway.

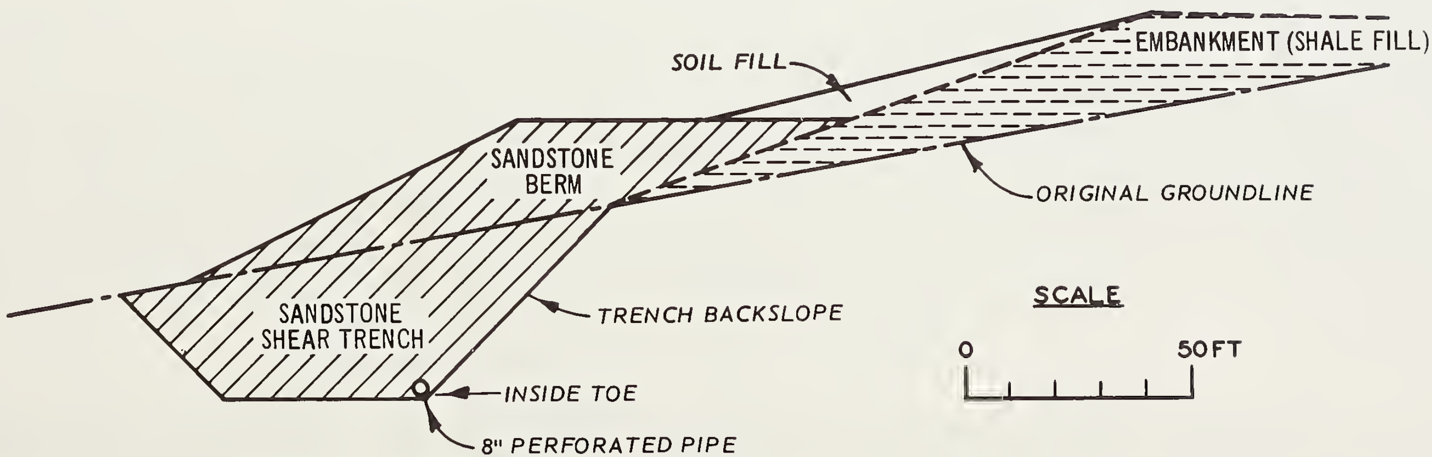


Figure 64. Recommended treatment of failed slope in shale fill at milepost 92, sta 6192+50, Western Kentucky Parkway, Kentucky (courtesy of the Kentucky Department of Transportation).

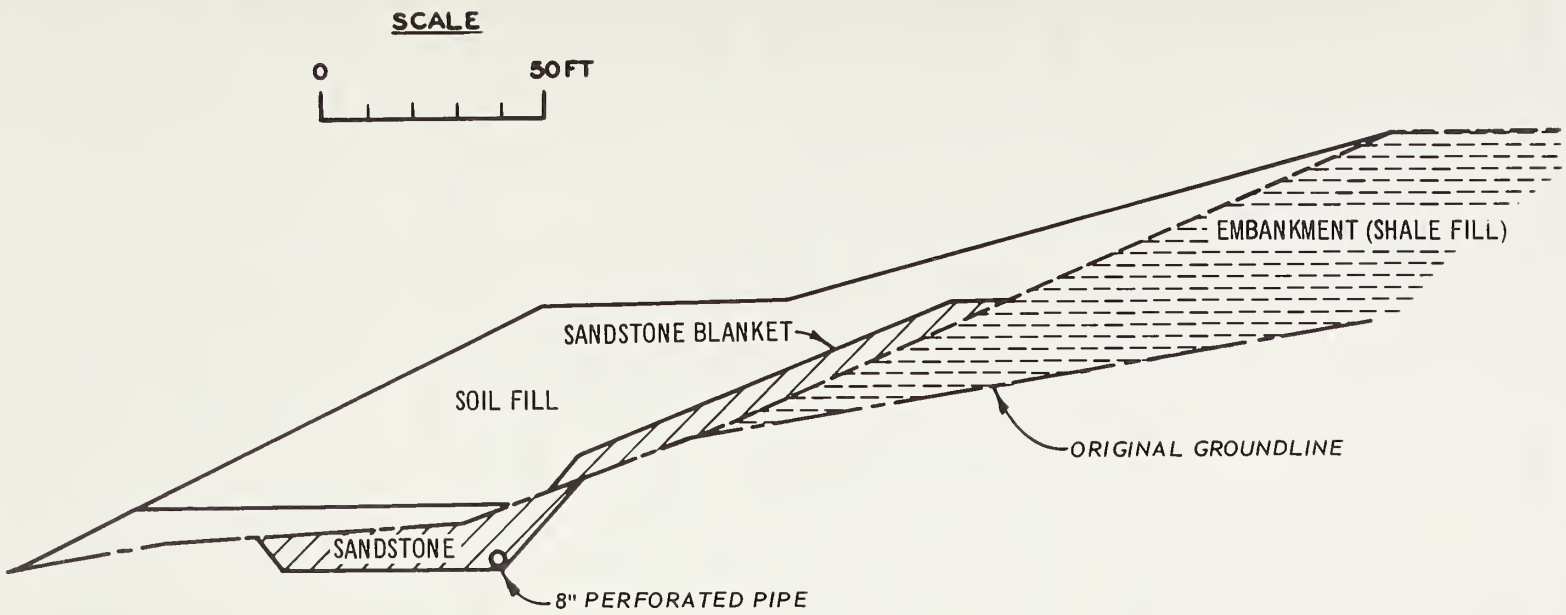


Figure 65. Recommended treatment of failed slope in shale fill at milepost 92, sta 6199+00, Western Kentucky Parkway, Kentucky (courtesy of the Kentucky Department of Transportation).

Retaining Walls

The following types of retaining walls are reviewed separately in subsequent sections:

a. Concrete walls.

b. Crib walls.

c. Gabion walls.

d. Pile walls.

e. Anchored walls.

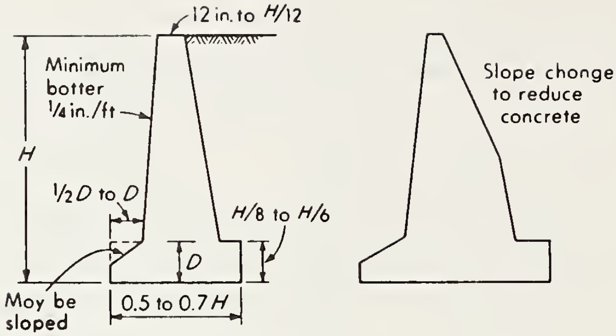
f. Reinforced earth walls.

General considerations. Retaining walls designed to support earth and rock masses have often been used to maintain stability of natural and embankment slopes. A retaining wall is usually constructed parallel to the adjacent slope crest (one exception is a wing wall abutment). In remedial treatment of embankments, retaining walls can be placed anywhere along the slope face or at the end of berms (or buttresses). Location of the wall on the upper portion of the slope adjacent to the roadway can result in an overall slope flattening, but much of the benefit may be lost since the wall and supported roadway fill would act as a surcharge on the remaining fill. Retaining walls are likely to be most advantageous when used to support berm (or buttress) fills. Retaining wall costs have usually been justified only in cases where right-of-way is limited. Their application in remedial treatment of distressed slopes is hindered by the time required for wall construction. Where slope failure is imminent, temporary stabilization by drainage techniques or partial placement of berms may be necessary prior to retaining wall construction.

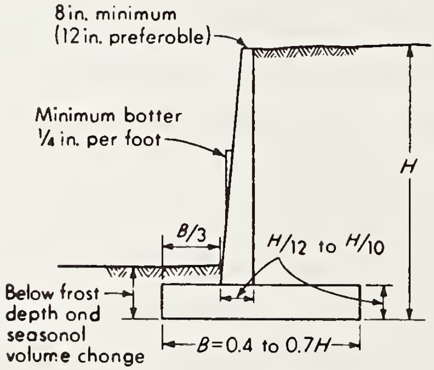
Design principles and behavior of many types of retaining walls have been discussed in several references. Comprehensive reviews are contained in Bowles (1968), Huntington (1957), Terzaghi and Peck (1967), and Tschebotarioff (1973). Retaining walls are designed to ensure safety against (a) structural failure, (b) overturning, (c) sliding along or near the wall-foundation contact, (d) foundation bearing capacity failures, (e) excessive differential settlement, and (f) large-scale shear failure of the supported fill where the sliding surface passes beneath or possibly through the wall. Forces exerted on retaining walls can be derived from earth pressure theories by applying graphical techniques, the method of slices, or semiempirical methods (see for example Terzaghi and Peck, 1967; Spencer, 1975).

Retaining wall failures have generally been initiated within the wall foundation (Terzaghi and Peck, 1967), which may partially account for the infrequent application of retaining walls in remedial treatment of distressed shale fills. The removal of weak residual soils has normally been required to provide adequate foundations; however, excavation in the toe area of a distressed embankment can lead to further instability. Even where residual soils are removed, there exists the potential of sliding of the wall along weak bedding planes (or other discontinuities) within the sedimentary rock foundation. An alternative to complete removal of residual soils is to construct a pile foundation through those soils. Piles can also add to the total horizontal resistance of a retaining wall. A valuable discussion of design and potential problems concerning retaining wall pile foundations through soft clay is contained in Tschebotarioff (1973). An important consideration is the development of bending moments within the pile as a result of lateral deformation caused by consolidation or rotational shear failure within the clay. To provide the necessary flexural resistance, Tschebotarioff (1973) recommends the use of steel H-piles or pipe piles.

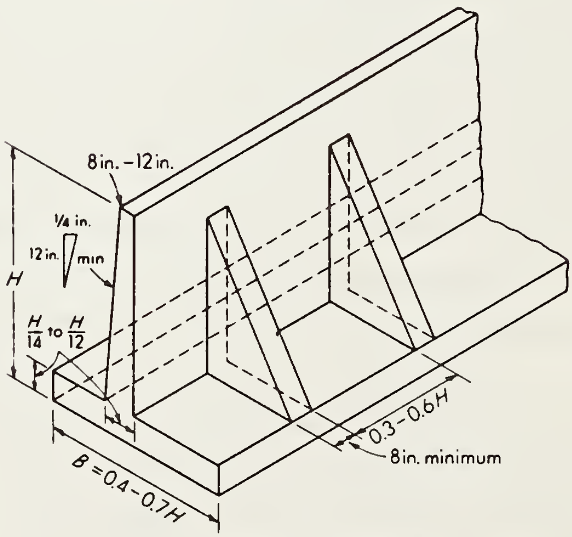
Concrete walls. Three common types of retaining walls constructed of concrete are the (a) gravity wall, (b) cantilever wall, and (c) counterfort wall. The gravity wall depends on its weight for stability. The wall is generally trapezoidal in shape and its massive proportions often allow construction with low strength concrete (Figure 66a). The cantilever wall utilizes a cantilever action caused by the weight of the soil on the heel portion of the base slab (Figure 66b). The same is true for the counterfort wall except that counterforts are set at intervals behind the wall to reduce bending moments and shear stresses within the wall (Figure 66c). The cantilever and counterfort walls are heavily reinforced.



a. GRAVITY WALL



b. CANTILEVER WALL



c. COUNTERFORT WALL

Figure 66. Tentative design dimensions for typical concrete retaining walls (Bowles, 1968).

The economy and safety of concrete retaining walls are largely dependent on the implementation of adequate drainage provisions to prevent the accumulation of water behind the wall. Proper design requires that the water not only be effectively removed, but that the increase in lateral pressure due to seepage toward the drains and the effects of frost action adjacent to the retaining wall be minimized. Methods of draining retaining wall backfills are discussed by Terzaghi and Peck (1967).

A concrete retaining wall has not been used or considered for remedial treatment of a shale embankment. In the few cases where a retaining wall was considered, crib, gabion, or reinforced earth walls were preferred. These techniques are less expensive than concrete walls and offer other advantages as discussed in the following sections.

Crib walls. Crib cells are constructed of interlocking units of either metal, precast reinforced concrete, or wood. These cells are linked together and filled with crushed rock and sand to form a crib wall as shown in Figure 67. The crib wall is advantageous since it permits free drainage of the crib fill and retained material. The crib fill should be of sufficiently large grain size and properly graded to prevent loss of crib fill materials through the open face. An extensive drainage system behind the crib wall should not be necessary. However, some granular backfill should be placed to significantly lower the water table immediately behind the wall and to provide a filter zone to prevent piping of fines into the wall.

The bin wall is similar to the crib wall in that it is constructed of soil-filled interconnected cells; however, each cell of the bin wall is closed on all four sides. This type of construction allows the fill to be composed primarily of finer soils such as clean sands. However, drainage systems are required behind the wall, and weep holes are needed to drain the cell fill.

Crib walls have been used for many years. Root (1958) and Baker and Marshall (1958) have presented photographs of several successful crib wall installations. Seelye (1956) and Tschebotarioff (1973) recommend that the crib cells be filled prior to backfilling and that individual structural members be designed based on pressures computed by

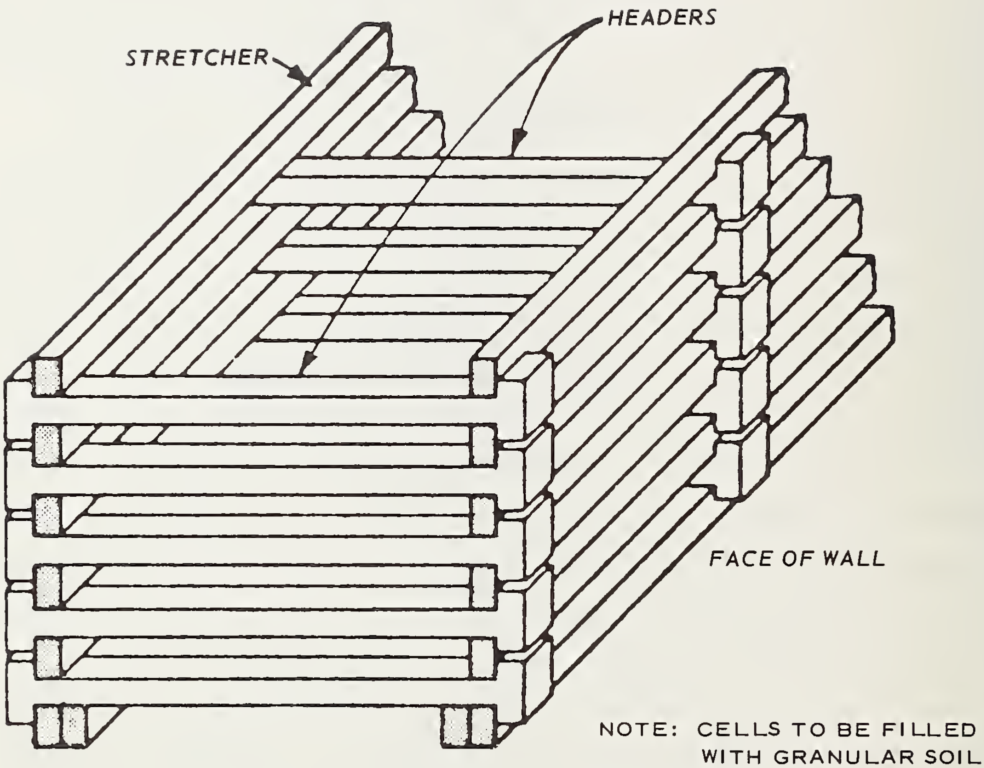


Figure 67. Crib wall (Bowles, 1968).

methods similar to those used for silos and bins. The crib wall is analyzed as a gravity retaining structure. Seelye and Tschebotarioff note that the crib wall can withstand considerable differential settlement. However, high walls must have adequate width to protect against excessive lateral deformation. Tschebotarioff (1973) indicates that an 8-ft-wide wall should be limited to a height of about 16 ft. In an example cited by Tschebotarioff, safety against sliding or shear through the wall was checked by assuming that the shear strength of the wall was equal to that of the contained fill.

To improve their stability, high crib walls are commonly battered toward the fill and can be supplemented by cribs of varying heights placed behind the main wall. The Department of the Navy (1962) recommends that crib walls higher than 4 ft be battered toward the backfill a minimum of 2 in. per ft of height and suggests that supplemental cribs may be necessary for walls higher than 12 ft.

Crib walls recommended for use in stabilizing shale embankment slopes are shown in Figures 68 and 69. In these cases, the large buried portion of each wall acts as a shear key. The benefits of burying a large portion of a crib wall should be carefully determined before applying the technique. Excavation of a trench will be required as shown in Figure 69. Backfilling the trench with a granular material may provide the needed additional shear resistance, thus foregoing the expense of crib materials and construction. Of course, other factors must be considered; for example, excavation of a significant depth of soft materials may be required to reach a firm foundation for the crib wall. Also, a short section of exposed crib wall can conserve a large section of right-of-way where proposed slopes are relatively flat. In any case, trench excavation near the embankment toe could cause further distress or complete slope failure. Such excavation would probably have to be supplemented by drainage measures. Even if construction is deferred to a dry season, bracing of the trench may be necessary.

Gabion walls. The gabion wall is constructed of interconnected rectangular wire-mesh containers filled with coarse nondegradable rock. The mesh consists of steel wire (U. S. Gauge No. 11) woven in a hexagonal

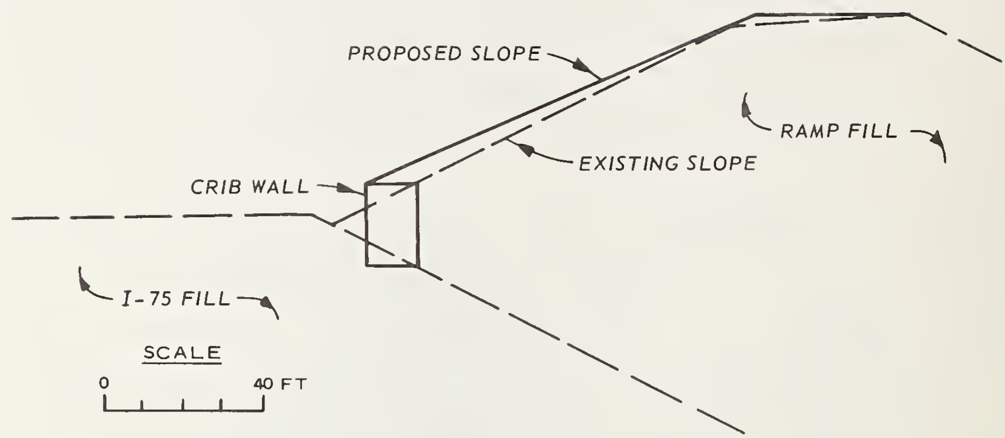


Figure 68. Proposed remedial treatment at the I-75 - I-275 interchange, Kentucky (courtesy of the Kentucky Department of Transportation).

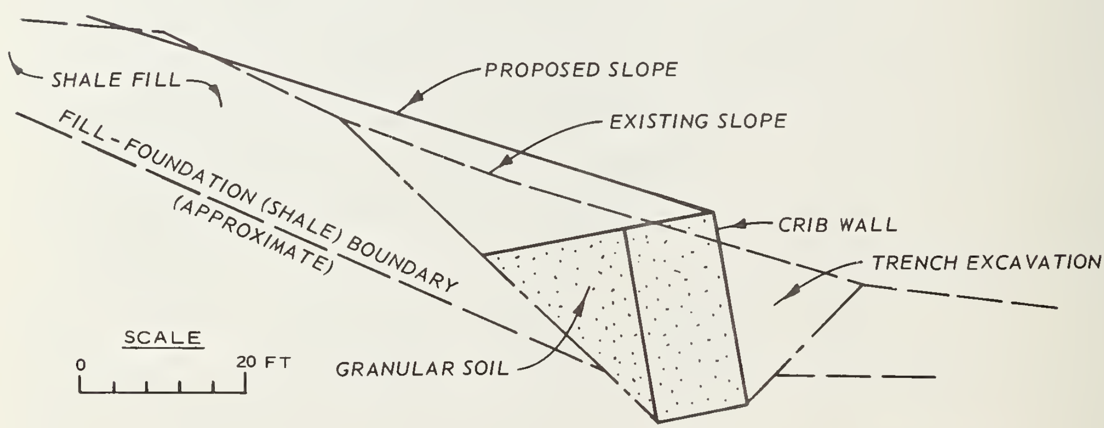
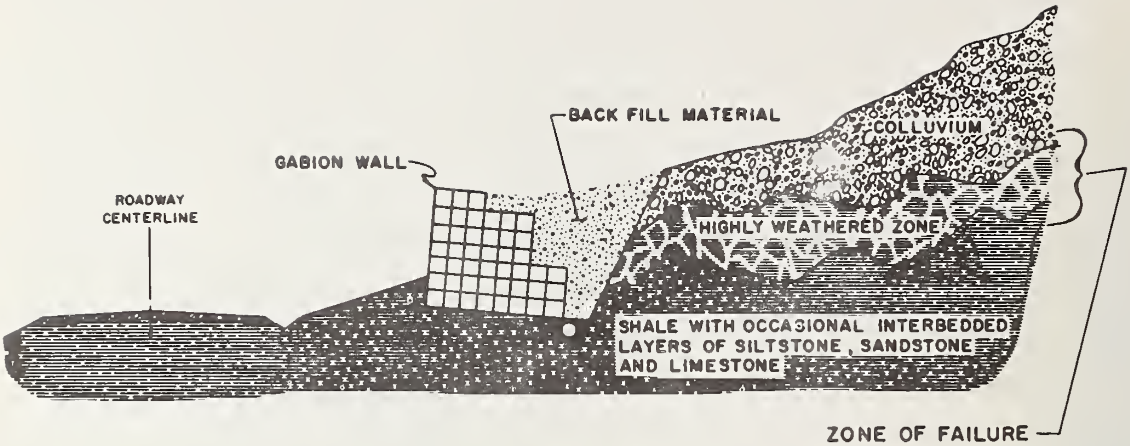


Figure 69. Alternate correction scheme, Blue Grass Parkway, milepost 21, Kentucky (courtesy of the Kentucky Department of Transportation).

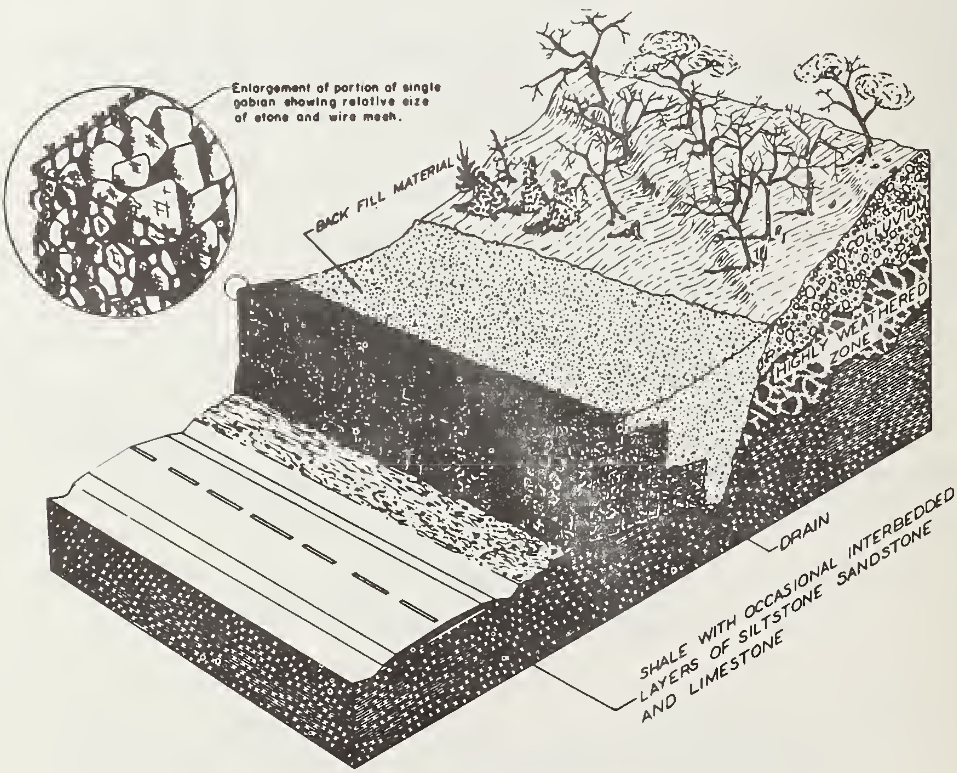
pattern with openings of about 3 to 4 in. Figure 70 depicts the manner in which several gabion walls as high as 30 ft were used in remedial treatment of slides along I-40 near Rockwood, Tennessee (Royster, 1973). The massiveness of the gabion structures and large particle size of the rock fill (4- to 8-in. pieces) are shown in Figures 71-73. The gabion wall offers advantages similar to those of the crib wall. It is free draining and can withstand large differential settlements. At Rockwood, the cost of the gabion walls was similar to or less than that of alternately proposed crib walls.

An extensive drainage system behind a gabion wall should not be necessary; however, some granular backfill should be placed to lower the water table immediately behind the wall. A filter zone or filter cloth should also be used to prevent removal of fines through the highly permeable wall. A failure involving the upper portion of one of the gabion walls at Rockwood occurred apparently because of saturation of the backfill (Blackburn, 1973). The slide occurred during a rainy period, and saturation of the backfill was suspected to have been caused by infiltration from springs covered by the backfill. Vertical failure scarps standing unsupported in the backfill indicated an excessive content of fines, which was later verified by sampling and excavation within the backfill. Specifications had called for backfill stone grading from 1 in. to not greater than 5 percent less than 0.1 in. These specifications have since been changed to stone grading from 2-1/2 in. to not greater than 5 percent less than 1/4 in. with placement by end-dumping.

Gabion walls have not been used extensively in the United States; however, much experience has been gained in Europe. Literature concerning past experience, design, and construction can be obtained from companies selling the gabion wire containers. Gabion walls are designed as a gravity retaining wall. These walls are susceptible to lateral differential movement, and it is suggested that shear through the wall be checked by assuming the shear strength equal to the contained rock fill. Gabion walls have been used often in erosion control and might serve as excellent protection of remedial slopes and berms adjacent to or intruding into rivers and streams.



a. 2-DIMENSIONAL SKETCH



b. 3-DIMENSIONAL SKETCH

Figure 70. Gabion wall used in remedial treatment of slides along I-40 near Rockwood, Tennessee (Royster, 1973).

Figure 71. Gabion construction, I-40 near Rockwood, Tennessee (courtesy of Law Engineering Testing Company).

Figure 72. Gabion construction, I-40 near Rockwood, Tennessee (courtesy of Law Engineering Testing Company).

Figure 73. Gabion construction, I-40 near Rockwood, Tennessee (courtesy of Law Engineering Testing Company).

Pile walls. Vertical piles have often been installed to provide additional shear resistance along existing and potential shear surfaces. Sufficient pile length is provided to cross the zone or surface of shear failure and anchor the pile in stable material. In treatment of highway embankments, the piles are generally placed in rows near the shoulder or toe of the embankment as shown in Figure 74. The spacing between piles is commonly limited to less than a few feet to effect a continuous wall. Root (1958) and Baker and Marshall (1958) reported that most pile installations have been unsuccessful in providing permanent stabilization; however, more recently Abrams and Wright (1972) reported that the Texas State Highway Department has successfully applied the technique in stabilizing numerous cut slopes and at least one embankment slope.

Wood, steel, and reinforced concrete piles can be used. Wood piles driven in place are disadvantageous in that difficulties may be encountered in obtaining a firm anchorage without damaging the piles. Steel piles such as I-beams, H-piles, or pipe piles can be driven in place although a more secure anchorage can be obtained by placing them in predrilled holes that penetrate stable rock strata and backfilling the holes with concrete. Cast-in-place reinforced concrete piles less than 2 ft in diameter were used in Texas (Abrams and Wright, 1972).

In over half of the cases reported by Abrams and Wright (1972), a retaining wall was constructed by placing wood planks, wood piles, steel guard rails, or even a cast-in-place concrete wall against the upper portion of the vertical piles. The walls were sometimes left exposed to support berms or flatter slopes; however, in most cases the walls were buried within the slope to provide additional resistance to sliding along shallow shear surfaces and to inhibit the movement of materials between the vertical piles. Collector pipes and granular filter material were placed behind the retaining walls. Slopes treated ranged in height from 15 to 30 ft and usually consisted of a plastic clay, clay shale, or marl.

General application of the pile method by the states surveyed in this study is unknown; however, the technique has received little use in repair of shale embankments. In the few cases where piles were used,

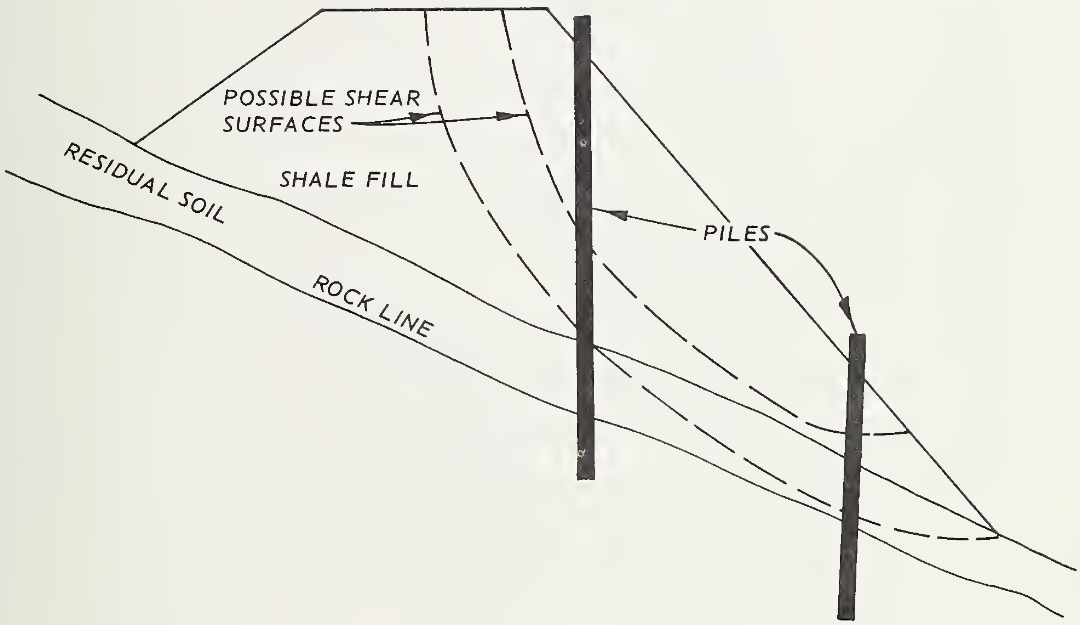


Figure 74. Piles located to provide lateral restraint against shear failure.

they provided only temporary support. Problems with the technique were attributed largely to the practice of installing piles as remedial maintenance without prior analyses to estimate their capabilities. Problems also occurred where wood lagging or old guard rail was placed against exposed sections of the piles to form retaining walls which were backfilled to support shoulder areas. Although this practice may be necessary to support the roadway, the backfill in the shoulder area adds driving force which can offset the beneficial effects of the piles.

A method of analyzing the stabilizing effect of piles is presented by Baker and Yoder (1958) in Highway Research Board Special Report Number 29. The piles act as cantilever members and are susceptible to failure by (a) overturning, bending, or shear through the piles; (b) shear failure beneath the piles; and (c) shear between and past the piles. These possible modes of failure are considered in the analysis presented by Baker and Yoder (1958). It is particularly important to space the piles close enough to prevent shear around the piles. If the piles are too far apart, their strength will not be mobilized, making little difference as to whether they are wood or high-strength steel piles. Baker and Yoder (1958) use the Swedish method of slices (Fellenius, 1927, 1936) to compute earth pressure forces on the pile wall for an assumed circular shear surface. More rigorous methods of slices have since been developed and should be used in place of the Fellenius technique. Abrams and Wright (1972) describe a method for determining earth pressures on the buried pile supported retaining walls discussed above. A circular failure surface is assumed and the method of slices is applied based on assumptions made by Spencer (1967).

Pile walls can also be constructed of large-diameter ($\geq$4 ft) reinforced concrete cylinder piles. In several cases 4- to 5-ft-diam cylinder piles spaced on 6-ft centers were installed to stabilize cut slopes during construction of interstate highways through Seattle, Washington (Andrews and Klasell, 1964). The slopes were cut into overconsolidated layered silt and clay soils. In one case (Figure 75) the cylinder piles helped maintain slope stability during construction of a tunnel adjacent to the toe of a cut slope. The cut slope was previously

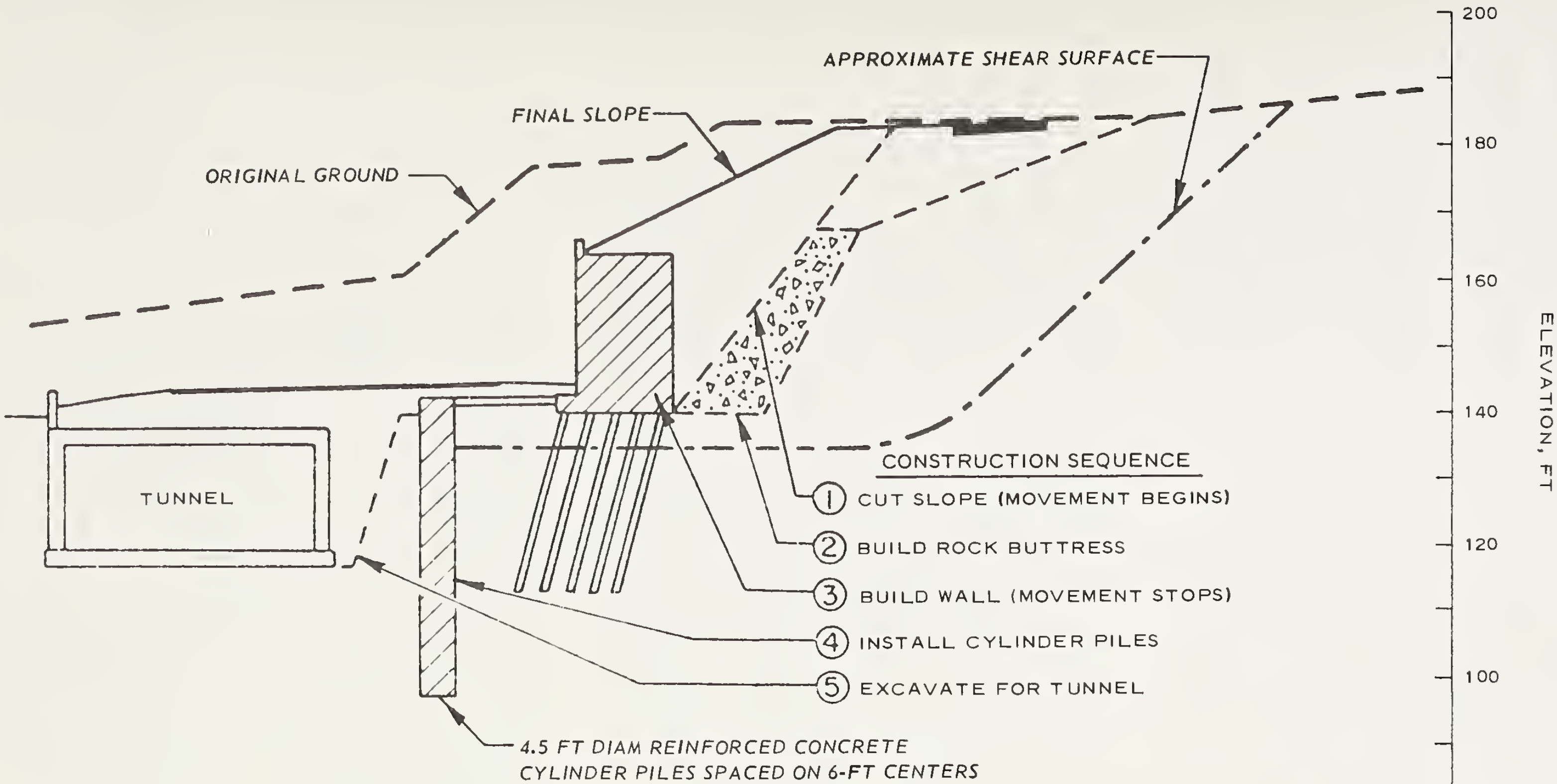


Figure 75. Stabilization of landslide by construction of a rock buttress, concrete gravity wall, and cylinder piles (Andrews and Klasell, 1964).

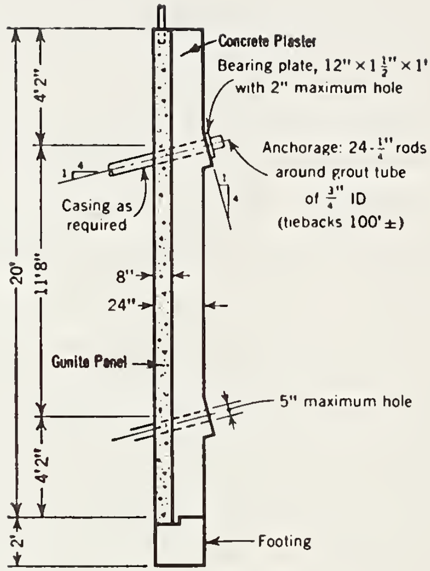
stabilized by construction of a rock buttress and a conventional gravity retaining wall as shown in Figure 75. Design criteria for the pile walls were set forth by Peck (1963). The design procedure consists mainly of the determination of the applied and resisting earth pressures. Each pile was designed as a cantilever member with deflections at the top of the pile dependent upon (a) magnitude of the applied loads, (b) flexural rigidity of the cylinder pile section, and (c) soil resistance developed due to displacement of the cylinder pile. Certain piles were instrumented, and preliminary studies showed that bending moments in the range of 50 to 100 percent of design values were activated (Gould, 1970).

In remedial treatment of shale embankments, cylinder piles can be constructed along the embankment shoulder or side slopes and based within the underlying foundation rock. This treatment may be particularly advantageous in stabilizing sidehill fills. If movement is suspected within thick deposits of residual soils or even the underlying rock, construction of cylinder piles in the toe area may be most beneficial. Cylinder piles are constructed one at a time; thus, toe support will not be lost due to excavation. Wilson (1974) indicated that drilling and casting of cylinder piles is relatively simple even under wet conditions. An advantage of these piles and smaller diameter piles is the capability to install them through temporary or permanent berm (or buttress) fills.

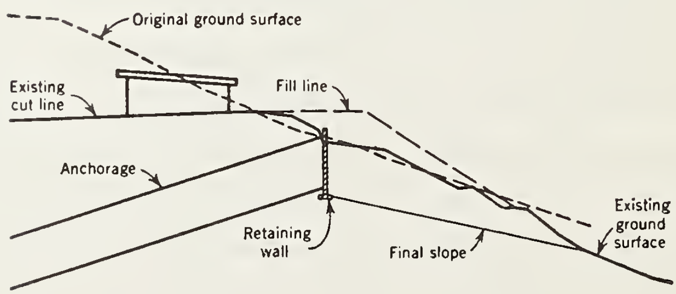
The less expensive smaller diameter piles will likely be used in most cases. They can be installed rapidly without prior excavation and therefore can provide emergency temporary support. The most efficient pile spacing, depth, and design can be determined by estimating the location of the failure surface from slope inclinometer data and by using back-calculated shear strengths in applying the method of slices for determining forces to be resisted by piles.

Anchored walls. A method of increasing the stabilizing capability of a retaining wall is by means of anchor ties placed in holes drilled beyond the locations of existing or potential failure surfaces. An example discussed by Reti (1964, 1965) and Gould (1970) illustrates the technique. In the Hollywood Hills community of Los Angeles, several homes founded on a horizontal cut section bordered by a sidehill fill were being threatened by continued movement of the slope. Various types of retaining walls (including crib walls) were considered, as were earth buttresses. The method considered most advantageous was an articulated retaining wall anchored within the hillside rock. One of the main advantages of the wall was that a retaining force could be applied immediately after construction. The wall is 20 ft high, 260 ft long, and consists of 13 independent panels, each 20 ft long. The wall panels were constructed of 8-in.-thick gunite and were reinforced as one-way slabs spanning between two vertical stiffening pilasters (Figure 76). The pilasters project 16 in. from the wall, are 20 in. wide, and are reinforced as vertical beams spanning between anchor points. At each pilaster, two tie rods, both 100 ft in length, were anchored in rock (Figure 76) through a 4-3/4-in.-diam drill hole. Each anchor consisted of 24 steel wires (1/4 in. in diameter) and was designed to be loaded in tension to 100 kips. To install the anchors, the wires were placed around a grout pipe (3/4 in. in diameter) with spacer plates between the wires. Each anchor was loaded to 150 kips for a period of 10 min. The load was then released, and the design load of 100 kips was applied. During the 150-kip test the anchors elongated about 2-1/2 in. and the wall panels moved about 3/4 in. toward the slope.

A similar treatment can be used for retaining wall supported berms or buttresses used to stabilize sidehill shale fills. Anchorage can be obtained deep within the sedimentary rock beneath the sidehill fill as shown in Figure 77. In certain cases it may be feasible to construct the retaining wall by placing lagging between piles. Piles have often been used in remedial treatment of distressed slopes by installing them across the suspected failure zone as discussed in the previous section. Piles placed in the toe area to prevent foundation movement can be used for convenient construction of a retaining wall to support a berm. The retaining wall can be anchored in a fashion similar to that used to support vertical excavations (e.g., Larson et al., 1972). An installation



a. CROSS SECTION OF GUNITE WALL PANEL, CONCRETE PILASTER, AND ANCHORAGES



b. CROSS-SECTION SHOWING SLOPE CONDITION BEFORE, AND AFTER FAILURE AND REMEDIAL TREATMENT.

Figure 76. Remedial treatment using an anchored retaining wall (Reti, 1964).

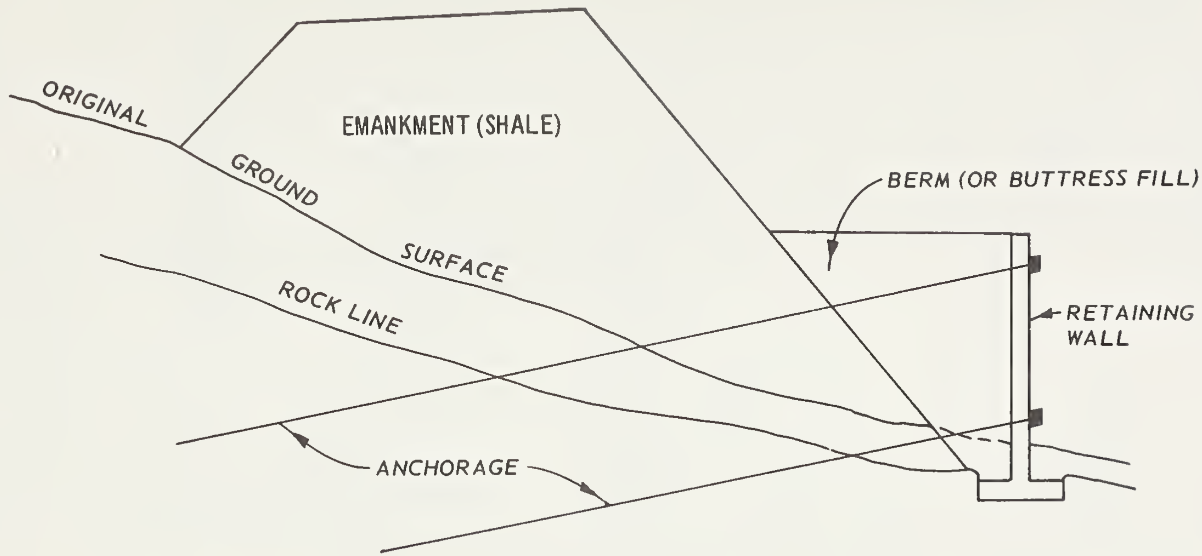


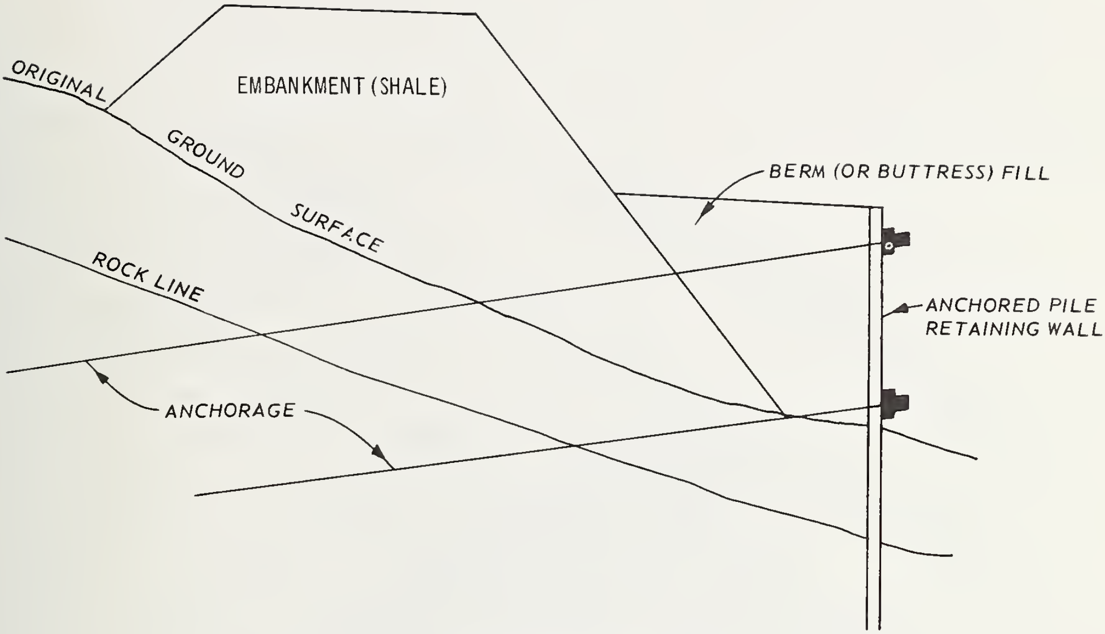
Figure 77. Anchored retaining wall to support a berm (or buttress) fill.

using H-piles and precast reinforced concrete paneling might appear as shown in Figure 78.

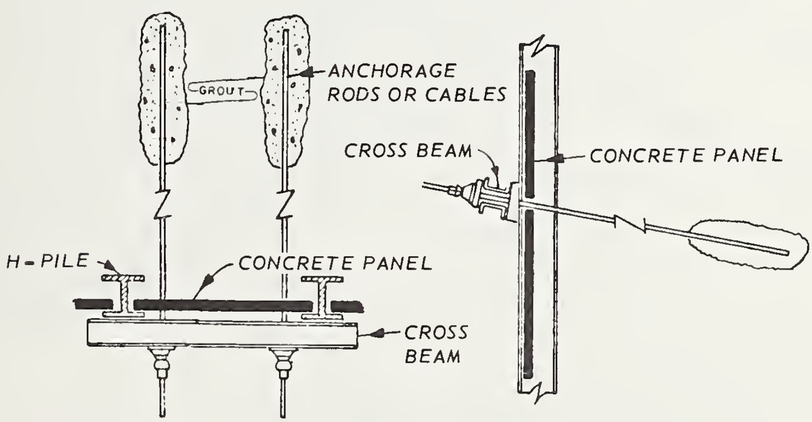
Reinforced earth. Reinforced earth is a construction material composed primarily of soil whose quality and performance have been improved by the introduction of small quantities of other materials in the form of bars, strips, and fibers to resist tensile forces that soil alone cannot resist. Although the concept of reinforced earth is old, it had remained qualitative until 1969 when Henri Vidal developed a rational concept for estimating stresses within a reinforced earth mass (Vidal, 1969). The basic concept assumes that a differential tensile force developed in the reinforcement creates a linkage between soil grains such that the tensile forces are restricted by a frictional force between the reinforcement and soil grains.

The reinforced earth retaining wall, shown schematically in Figure 79, consists of three major components: the backfill material, the reinforcements, and the skin elements. The backfill material usually consists of free-draining material whose strength depends on the frictional forces between the particles rather than the cohesion of the mass. Material with a relatively high angle of internal friction such as sand and gravel with less than 15 percent fines (i.e. material passing the No. 200 sieve) is suitable for backfill. The backfill material is commonly placed in horizontal layers of constant thickness and density. The reinforcements commonly consist of long, thin metal strips placed to provide an additional strength to the backfill material in a manner similar to steel in a reinforced concrete structure. The metal strips should possess high tensile strength (minimum of 36,000-psi yield strength) and adequate resistance to environmental conditions. The skin or facing elements are used to protect the surface of the wall and prevent raveling of the backfill material that may endanger the integrity of the structure. The reinforcing elements are attached to the skin elements by means of brackets, bolts, or hooks.

The reinforced earth retaining wall is analyzed as a gravity structure. Analytical concepts and design considerations are presented in Appendix D and in discussions by Vidal (1969) and Lee et al. (1973).



a. ANCHORED PILE RETAINING WALL TO SUPPORT BERM OR BUTTRESS FILL



b. ANCHORAGE DETAILS

Figure 78. Schematic drawings of an anchored pile retaining wall using H-piles and concrete panels.

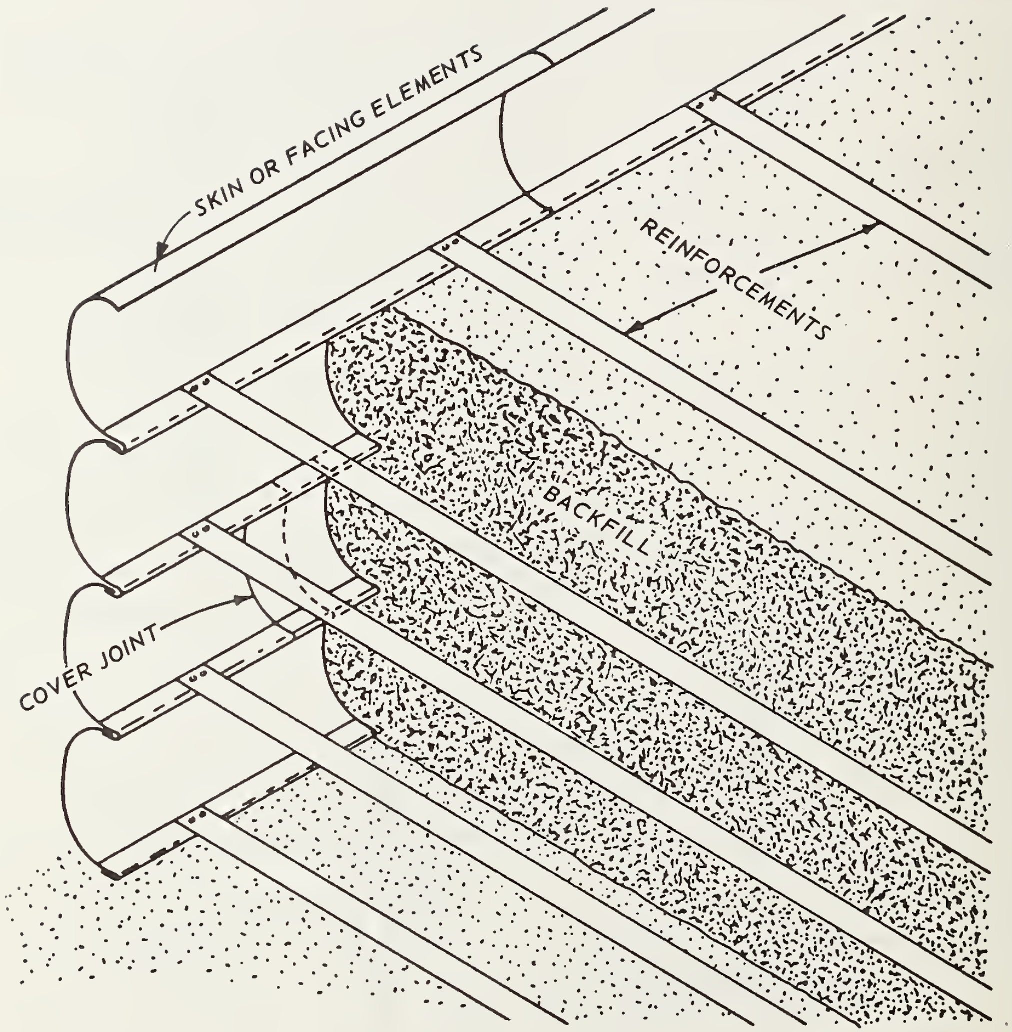


Figure 79. Schematic drawing of a reinforced earth wall (Vidal, 1969).

The reinforced earth wall is flexible, economical, and easily constructed.

Reinforced earth construction began in the United States in 1971 with the establishment of the Reinforced Earth Company, Washington, D. C., as the licensee for the purpose. Reinforced earth retaining walls have since been used on several projects in the United States. A few examples are described below to illustrate their potential applicability in remedial treatment of shale embankments.

Route 39, California. The first reinforced earth wall in the United States was used in the reconstruction of a section of Route 39 near Crystal Lake in the San Gabriel Mountains, Los Angeles County, California (Chang et al., 1972b; Gedney, 1972; Chang et al., 1974). The highway is supported by the reinforced earth wall which is constructed over a random fill embankment founded on slide debris. A toe berm (or buttress) was built at the bottom of the slide debris to support the embankment fill and natural slope as shown in Figure 80. The reinforced earth wall has a maximum height of 55 ft and a length of 528 ft. An extensive drainage system consisting of a 3-ft-thick permeable interceptor blanket placed behind the wall in combination with buried pipe drains was used to remove subsurface seepage as shown in Figure 80.

Consolidation-drained triaxial tests on the embankment material compacted to field density (85 percent relative compaction) gave an angle of internal friction of 44 deg. Shear box tests gave an angle of skin friction of 31 deg between the embankment material and the reinforcing ties. The reinforcing ties and skin elements were constructed of galvanized steel with a yield strength of 37,000 psi, ultimate strength of 50,000 psi, Young's modulus of 28,500,000 psi, and Poisson's ratio of 0.28. The ties had a thickness of 0.118 in. and a width of 2.362 in. and were 22.8 to 46 ft long. The semielliptical skin elements were 10 in. high along the major axis and 3.7 in. high along the minor axis, 0.118 in. thick, and 6.5 to 39.4 ft long.

To monitor the behavior of the completed structure, a comprehensive instrumentation program was planned and implemented under the auspices of the FHWA. The instrumentation included slope indicators to measure

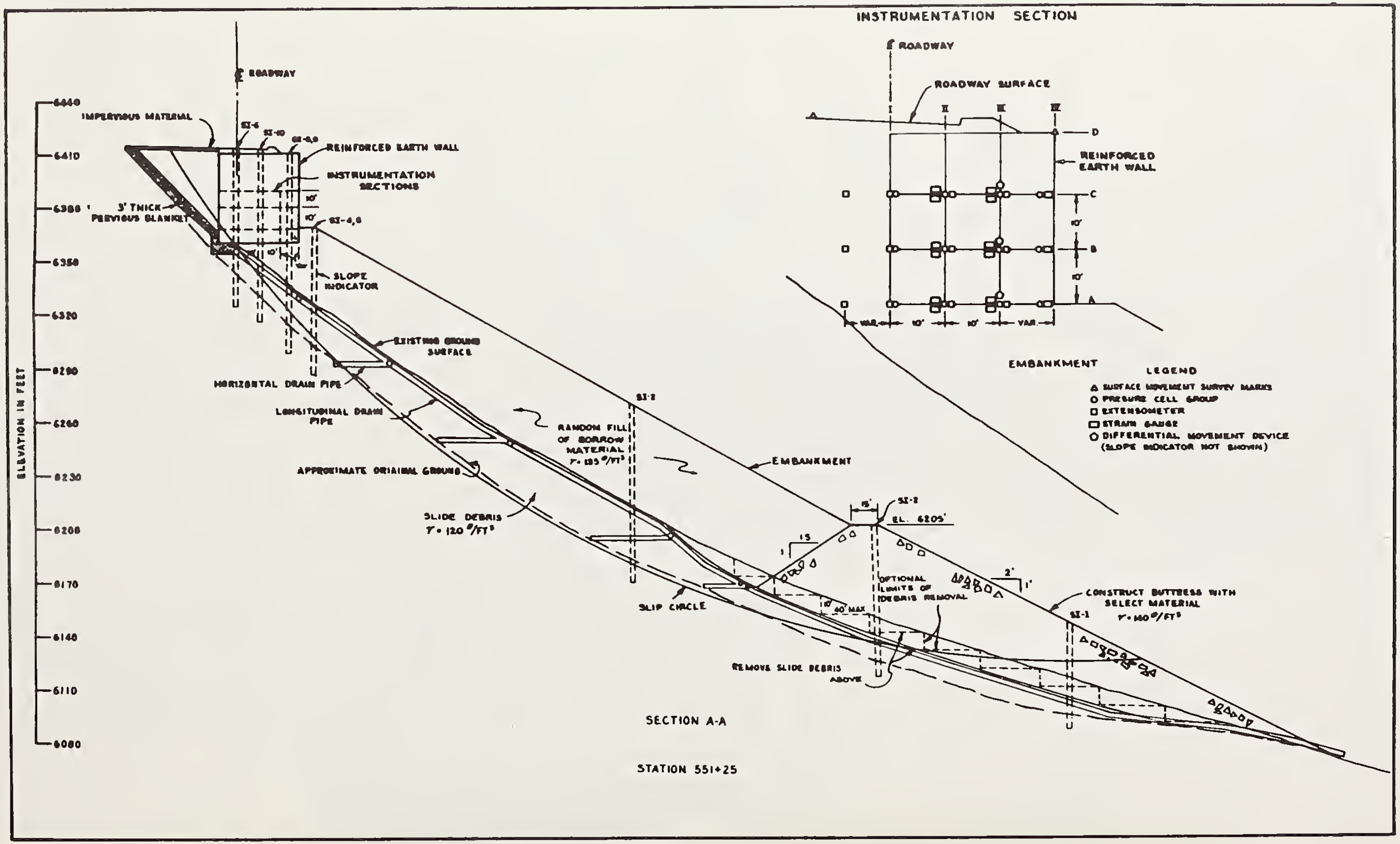


Figure 80. Reconstruction scheme employed on Route 39 near Los Angeles, California (Chang et al, 1972b).

the internal movement of the fill and slide debris, settlement platforms to measure vertical settlements, extensometers to measure soil strains, soil pressure cells to measure soil stresses, strain gages to measure the stresses developed in the reinforcing ties and skin elements, and gage points to measure the deformations of the skin elements and the wall face. Based on the results obtained from the instrumented reinforced earth structure and the analysis of performance, it was concluded that the basic mechanism of behavior could be explained by the Rankine stress theory. For design purposes, it was recommended that the active earth pressure coefficient be used for calculating stresses for the end portions of the reinforcing ties and that the coefficient of earth pressure at rest be used for calculating stresses for the middle portion of the reinforcing ties.

Heart O' The Hills Road, Washington. The Heart O' The Hills Road is located in the Olympic National Park just south of Port Angeles, Washington. In 1970, a major landslide occurred and approximately 200 ft of the road dropped vertically about 30 ft. Remedial treatment of the slide is discussed by Munoz (1974). A comprehensive study of the area showed that the slide material consisted of highly weathered fragments of clay, silt, and sandstone. Several underground springs were also found and were believed to have triggered the landslide.

Three remedial design alternatives were considered:

a. Sidehill embankment with berm.

b. Sidehill embankment supported by reinforced earth retaining walls.

c. Bridge structure.

The earth embankment and berm scheme was least expensive, but the large stabilizing berm would disrupt the park setting and extend into a nearby creek. The bridge structure was found to be the most expensive. The decision was made to use two reinforced earth structures as shown in Figure 81. The lower wall (40 ft high, 256 ft long, and 55 ft wide) was provided to ensure stability of the embankment and natural slope. The upper wall (25 ft high, 378 ft long, and 22 ft wide) was used to carry

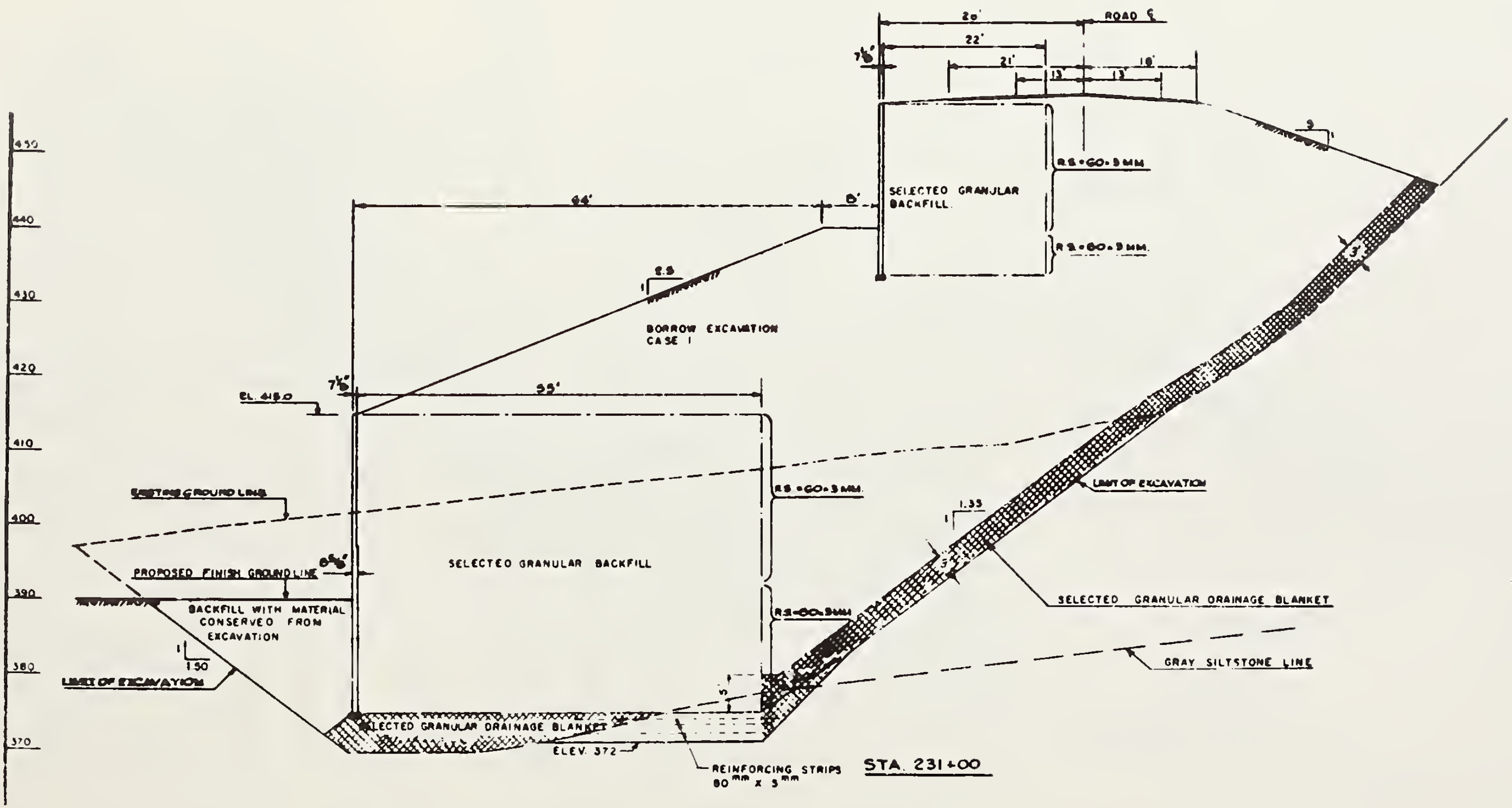


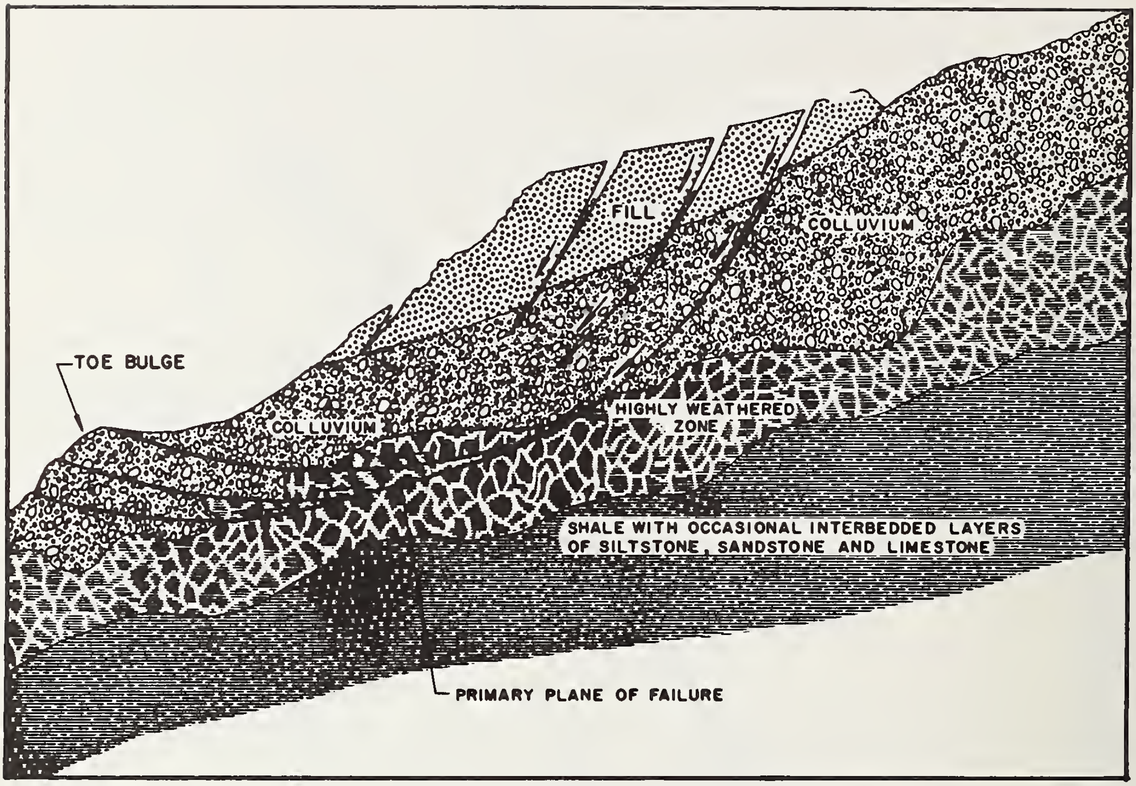
Figure 81. Reinforced earth retaining walls used in reconstruction of the Heart O'The Hills Road (Munoz, 1974).

the road. Pumped wells were installed to reduce the danger of pore water pressure buildup during construction.

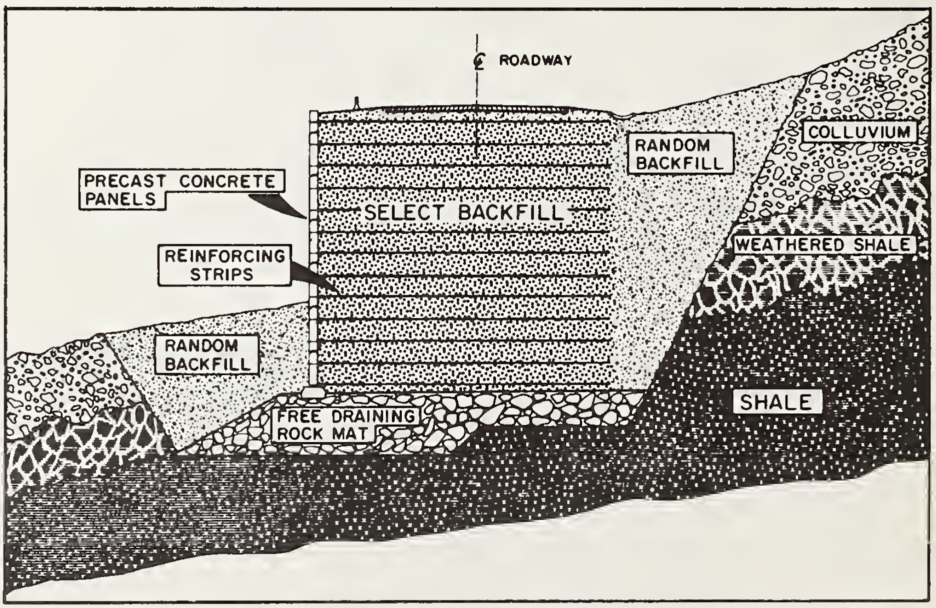
Precast concrete panels approximately 5 ft square were used as facing elements. These panels were placed vertically by a crane working from the top of the excavation. The reinforcing strips were bolted to the panels; and the backfill material was placed in 10-in. lifts, spread, and compacted with a steel-wheel compactor. The project was completed and the road opened to traffic in early December 1973.

I-40 near Rockwood, Tennessee. A reinforced earth retaining wall was constructed by the Tennessee Department of Transportation to correct a shale embankment failure on I-40 in Tennessee (Royster, 1974; Scott, 1974). The affected section of the highway is located in an extremely unstable area where several major landslides have occurred. Failure occurred through the weak colluvium and weathered shale foundation materials. The type of failure and reinforced earth correction scheme are shown in Figure 82. A rock buttress was also considered but was estimated to cost nearly twice that of the reinforced earth scheme. The reinforced earth retaining wall was about 800 ft long and 39 ft high. Cruciform-shaped, precast concrete panels were used as facing elements. Galvanized steel strips 1/8 in. thick, with varying width and length, were used as reinforcing strips. Each row of strips was covered with approximately 30 in. of sand compacted in 8- to 10-in. lifts. The reinforced earth wall was founded on a free-draining rock pad as shown in Figure 82b. The reinforced earth wall was completed in October 1973, and as of May 1974, no appreciable deflections or settlements had been observed (Royster, 1974).

Brunswick, Georgia. As of late 1974, a reinforced earth wall was under construction along the navigable tidal estuary, Academy Creek, in Brunswick, Georgia (Engineering News Record, 1974). The reinforced earth wall is being used in widening a primary state highway and relocating a railroad freight spur. The center line of the railroad will be 14 ft from the vertical face of the reinforced earth wall. The wall is being constructed in the dry in back of a temporary earth dike. The wall facing will be constructed of interlocking precast concrete panels.



a. EMBANKMENT FAILURE



b. REINFORCED EARTH CORRECTION SCHEME

Figure 82. Embankment failure and remedial treatment along I-40 near Rockwood, Tennessee (Royster, 1974).

To deter the corrosive action of the salt water, reinforcing strips of aluminum-magnesium alloy will have a thickness about twice that required by tensile considerations. Since most of the wall bracing will be under water, a filter material is being placed behind the concrete facing panels to prevent the sand backfill from washing out.

I-74, Indiana (alternate correction scheme). An alternate scheme considered for remedial treatment of SLIDE-2, I-74, Indiana, is shown in Figure 83. The reinforced earth wall was designed to support the stabilizing berm and the original shale fill. The wall was to be founded on shale at a depth of 25 ft below the original ground surface. The buried portion of the wall would serve as a shear key to prevent sliding through the residual soils. The remedial treatment used to correct the slide was selected based on cost and is shown in Figure 60.

Research concerning the behavior and strength of reinforced earth walls is continuing. WES has conducted field tests on two instrumented reinforced earth retaining walls. Each wall was 16 ft long, 10 ft wide, and 12 ft high and was surcharge-loaded to failure. The first wall was reinforced by strips made of heavy-duty, neoprene-coated nylon fabric membrane and the other wall was reinforced by galvanized steel strips. Sand was used as backfill material for both walls. A report discussing the results of those experiments is in preparation and tentatively scheduled for publication in 1975 (Al-Hussaini and Perry, 1975).

Lime, Cement, and Chemical Stabilization

The following treatment methods are reviewed separately in subsequent sections:

a. Cement grouts.

b. Chemical grouts.

c. Drill-hole lime.

d. Lime slurry pressure injection.

e. Electrokinetic stabilization.

f. Ion exchange.

Cement grouts. As discussed in Part II, shale embankment problems

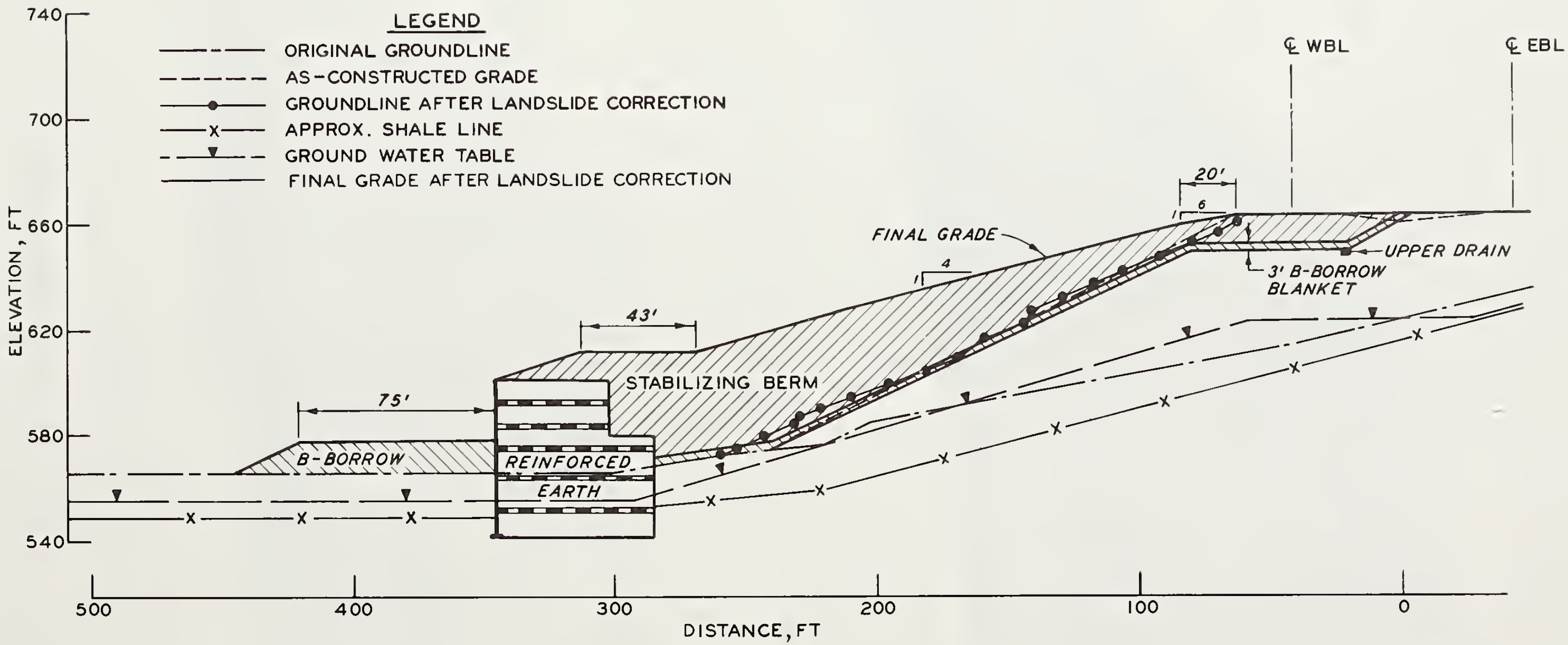


Figure 83. Alternate correction scheme, SLIDE-2, I-74, Indiana (courtesy of the Indiana State Highway Commission).

may be initiated by the presence of large voids which commonly result from either (a) placement of materials in thick lifts (3 to 4 ft) with little or no compaction or (b) prevention of proper compaction due to the presence of larger more durable shale or limestone particles. Cement grouts can fill these voids before significant shale deterioration has occurred. The Ohio Department of Transportation has successfully used cement grouts to stabilize shale bridge approach embankments experiencing excessive settlements. In one case, as presented previously, the abutment bearings were raised approximately 1-1/2 ft over a 4-yr period to compensate for vertical settlement. Accompanying lateral deformation had also forced contact between the abutment and superstructure steel. The abutment was stabilized by injecting 33,000 cu ft of cement grout into the approach fill. At two other locations injected grout volumes of 30,000 and 12,000 cu ft, respectively, were effective in preventing further settlements. In one case, grouting operations were unsuccessful in an attempt to stabilize an approach embankment which had undergone 12 in. of vertical settlement in 4 yr after construction. Ohio Department of Transportation personnel inspected the fill in a test pit 10 ft in diameter and 25 ft deep. The inspection revealed a general lack of grout penetration in the fill although isolated grout masses existed. Shale materials in the embankment were dry and appeared to have been broken down and well compacted during construction of the embankment.

Although penetration of cement grouts is generally limited to application in porous soils such as sands and gravels, the technique has successfully stabilized numerous slides in clay soils including saturated clays. Experience with stabilization by cement grouting has been obtained largely in application by American railways (Smith and Peck, 1955) and British railways (Purbick and Ayres, 1956; Ayres, 1959, 1961; Toms and Bartlett, 1962). A discussion of their practices is given in Duncan (1971). Typically, the grout points are spaced on 5-ft centers in rows 15 ft apart and set parallel to the track. Grout points are located about 3 ft below the shear surface. Approximately 50 cu ft of grout is injected through each point under relatively high pressures.

For example, when grouting at a depth of 15 ft, the first 10 cu ft may be injected under a pressure of 75 psi with the pressure subsequently lowered to 20 psi. Smith and Peck (1955) reported that, in American practice, grouting is sometimes initiated in the toe area to form a stable mass to resist failure under higher injection pressures applied farther up the slope. The above procedures have been shown to be effective. For example, Toms and Bartlett (1962) reported that cement grout had successfully stabilized 140 shear failures during a 9-1/2-yr period. Stability was usually achieved within 2 to 3 weeks after grouting.

Grouted sections have been excavated to determine cement grout penetration. The grout does not penetrate the voids of the clay or fissures in the clay, but penetrates along the failure surface, lifting the mass and forming a solid layer of neat cement when it hardens. Even before the grout is completely cured, an initial stabilization can occur as the grout displaces free water away from the failure surface (Smith and Peck, 1955).

Cement grouts appear to be most advantageous for treating two extreme problem conditions. First, for controlling settlements caused by the presence of large voids within the shale mass and second for treating shear failures in highly deteriorated shales or residual foundation soils where penetration of cement grout will be concentrated along well developed shear surfaces. When grout is applied to sidehill fills, supplementary drainage measures will probably be necessary to prevent new failures from developing below the treated area.

Guidance concerning grouting methods, equipment, and the effects of various mineral or chemical admixtures on the viscosity and penetrability of cement grouts is contained in Technical Manual TM 5-818-6 (Departments of the Army and Air Force, 1970).

Chemical grouts. A multitude of chemical grouts is available for application in engineering projects. The three main types are (a) sodium silicates, (b) chrome-lignins, and (c) organic monomers. The primary advantages of chemical grouts are their low viscosities and greater penetrability than cement grouts and the ability to control setting times by adding appropriate catalysts. For example, Mitchell

(1970) indicates that the organic monomers (often termed resins or polymers) can be useful in medium-sized silts (grain size greater than 0.01 mm). A commonly used resin grout is AM-9 developed by the American Cyanamid Company. The AM-9 grout can have a viscosity from 1 to 3 times that of water and a controlled setting period from 0.1 to 300 min (Caron, 1963).

Most chemical grouting applications have been used to fill voids and decrease permeability; however, strength increases from chemical grouts may be substantial. Erickson (1968) reported tensile strengths of 6,000 to 9,000 psi and compressive strengths of 10,000 to 20,000 psi for various resin grouts. A uniform distribution of these grouts could significantly increase the strength of the treated soil mass. Robnett et al. (1971) summarized results of various investigations which indicated that silicate chemicals may create unconfined compressive strengths of 85 to 350 psi in loess soils and 100 to 500 psi in sands.

Chemical grouts may be beneficial in cases where cement grouts have been unsatisfactory due to the presence of fine-grained low permeability clayey soils and/or the lack of a well developed shear surface; however, chemical grouting can be costly. Robnett et al. (1971) reported that costs per cubic yard of treated material ranged from $25 to $100 for chemical grouting and $8 to $15 for cement grouting. Chemical grouts generally require precise mixing and in some cases may be toxic. It is recommended that a specialist knowledgeable in the behavior and appropriate injection procedure be consulted when attempting to use chemical grouts. Guidance concerning chemical grouting is contained in Engineer Manual EM 1110-2-3504 (USAE, OCE, 1973b).

Drill-hole lime. Lime has been widely and successfully used as a stabilizing agent for reducing the plasticity and increasing the workability and strength of clay soils. The immediate plasticity reduction which improves the workability of the soil is primarily attributed to base exchange and flocculation or agglomeration. Strength gains are the result of pozzolanic cementing compounds being formed through the alteration of the clay minerals in the high pH environment provided by the lime. Lime treatment is advantageous in that it is generally more effective and less costly than cement in reducing the plasticity of clays. However, the effects of lime treatment depend on other soil chemicals; consequently, prior testing to determine soil chemistry and lime-soil reactions is required before applications.

One method of introducing lime into a distressed slope is by gravity flow of lime slurry placed in drill holes (i.e. drill-hole lime). The lime can also be placed dry if natural seepage is present to facilitate migration. A state-of-the-art assessment of this technique is presented by Robnett et al. (1971). The method has been used primarily to eliminate roadway heave caused by expansive clay subgrades; however, there have been several applications for stabilization of landslides in soft clay soils. The main disadvantage of the method is that both lime migration and strength increases are slow, time-dependent processes. Initial strength increases may be obtained due to base exchange and flocculation occurring due to the reaction of lime with soil; however, long-term, strength-producing pozzolanic reactions are time-dependent, often requiring several weeks to achieve most of their strength potential. Laboratory and field experience indicate that lime penetration is limited to within a few inches from the hole where it is placed (Graf, 1974). Generally, drill holes (minimum diameter of 12 in.) are spaced on 3- to 5-ft centers. The cases discussed below illustrate the technique and associated problems.

Handy and Williams (1967) described the drill-hole lime technique as used to stabilize a landslide involving several homes founded on a poorly compacted silty clay fill overlying shale. A portion of the landslide was stabilized by drilling 6-in.-diam auger holes on 5-ft centers down to the shear zone and introducing approximately 50 lb of pelletized quicklime to fill the bottom 3 ft of each hole. Approximately 20 tons of lime was used. Three months after treatment, movement of the treated area essentially ceased. Chemical tests 1 yr after treatment indicated beneficial pozzolanic cementation occurred within a 1-ft radius of each hole. Borehole shear tests conducted 32 months later showed an increase in cohesion from 0.6 to 1.4 psi and in angle of internal friction, ϕ , from 17 to 26.1 deg.

Abrams and Wright (1972) reported four cases in which the drill-hole lime technique was used in an attempt to stabilize slides in slopes cut into plastic clays. In three cases the technique was successful although the lime treatment was used with other measures. Typically drilled holes were 8 to 12 in. in diameter and were placed in staggered rows on 5- to 10-ft centers. Holes were commonly drilled by placing equipment on 12- to 14-ft-wide benches excavated into the slopes. An attempt was made to drill the holes to a depth several feet beyond the known or estimated failure surface. Hole depths have generally been 25 ft or less. In one case approximately 50 tons of lime slurry was used to stabilize a 40-ft-high slope consisting of thin limestone layers alternating with thicker clay layers. Shear failure had developed along a clay-limestone contact. Ten benches, each wide enough to allow a truck-mounted auger to drill 12-in.-diam holes down to the zone of failure, were cut into the slope parallel to the roadway. A total of 122 holes were filled with a lime slurry containing only enough water to readily mix and pump the lime. In addition to the lime treatment, the limestone layer where the failure had developed was fractured with dynamite to lower a perched water table, and in one area a horizontal pipe drain was installed to remove seepage. The slope was surveyed periodically following treatment. Seeps were observed on the slope; however, the slope appeared to have stabilized at the end of 6 months and has remained stable at least during an 11-yr period prior to the preparation of the report by Abrams and Wright (1972).

The Oklahoma Highway Department has reported numerous successful instances of drill-hole lime applications to stabilize clayey subgrades (Rural and Urban Roads, 1963). Typically 9-in.-diam and 30-in.-deep holes have been drilled on 5-ft centers through existing pavements and backfilled with 25 lb of lime and sufficient water to create a slurry. Similar treatment used by the Louisiana State Highway Department to stabilize a clay subgrade was unsuccessful (Higgins, 1969). In these cases, 25 lb of lime was placed in 9-in.-diam holes, 18 to 24 in. deep, and set on 3-ft centers, while 50 lb of lime was placed in 9-in.-diam holes, 36 or 48 in. deep, and set on 5-ft centers. Samples tested

1 yr after treatment showed no significant change in pH, calcium content, or plasticity of the soil mass. No apparent lime migration occurred from the periphery of each hole, and pavement subsidence actually increased following treatment.

The Colorado State Highway Department (1967) has used the drill-hole lime technique to successfully reduce swelling in expansive clay and shale subgrades. Generally, 12-in.-diam holes spaced on 5-ft centers were used with depths ranging from 6 to 20 ft depending upon the extent of treatment desired. After placement of lime slurry the holes were backfilled with sand and gravel. Experience has shown that at least 12-in.-diam holes are needed, and slurry concentration must be limited to 1 lb of lime per gal of water to allow sufficient lime migration. Even under these conditions, several experimental sections have shown that the lime penetration is limited to a distance of 2 to 3 in. from the borehole.

Lime slurry pressure injection. The technique of lime slurry pressure injection (LSPI) was developed to improve lime penetration in clay soils. A state-of-the-art assessment of the LSPI technique is presented by Robnett et al. (1971) and Thompson and Robnett (1975). The LSPI method consists of pumping lime slurry into the soil under pressures up to 200 psi through hollow injection rods tipped with special injection nozzles. In typical applications, the injection rods are hydraulically pushed into the soil and injections are made at approximately 12-in. intervals until the desired depth is achieved. At each interval, slurry (2.5 to 3.0 lb of lime per gal of water) is injected to refusal where refusal can be defined as:

a. Soil will not take any additional slurry.

b. Slurry is exiting from around the injection pipe or out of other injection holes.

c. The slurry has fractured the surface.

Injection points are commonly spaced 3 to 5 ft apart, and a wetting agent is often added to the slurry to assist in migration.

Application of LSPI has been used primarily for stabilization of expansive clay subgrades. Treatment depths have ranged from about

3 to 20 ft. The technique is apparently a suitable alternative to the drill-hole lime method, and Thompson and Robnett (1974) have concluded LSPI to be more effective. Observations made in test pits excavated up to 4 yr after treatment have indicated that the injected lime slurry generally forms a network of thin, near-horizontal seams with vertical migration of less than 2 in. above and below individual seams (Higgins, 1965, 1969; Lundy and Greenfield, 1968). The seams generally extend to a radius of 1 to 5 ft from the injection point although penetration distances of as much as 10 ft have been observed (Wright, 1975).* The existence of fissures seems to be necessary to achieve significant lime penetration. As with the drill-hole lime technique, significant strength increases may not be realized for several weeks following treatment. Following LSPI treatment, it has not been uncommon to experience an initial, but temporary, reduction in strength due to the large volumes of water introduced which can cause additional swelling of clays (Wright, 1973; Thompson and Robnett, 1974, 1975).

The lime slurry pressure injection technique may be suitable for penetration along shear surfaces in the manner observed in cement grout applications as discussed previously. Potential field problems include difficulty in pushing the injection rods through shale fills containing large rock pieces and initiation or magnification of failure resulting from the high injection pressures and the addition of large volumes of water into the failure zone. Field trials or experimentation will be necessary to determine operational details and utility of the LSPI method in treatment of shale embankments.

Electrokinetic stabilization. Electrokinetic stabilization improves soil properties by applying the phenomenon of electroosmosis. If two electrodes are driven into a saturated soil and an electric current is made to flow from one to the other, the water contained in the soil migrates from the positive electrode (anode) toward the negative electrode (cathode). The flow of water produced by the electric

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* P. J. Wright (1975), Personal communication, Woodbine Corporation, Fort Worth, Tex.

current is known as an electroosmotic phenomenon. The method can be used to dewater and consolidate fine-grained soils such as clays by using well points as the cathodes. The dewatering and accompanying consolidation will increase the shear strength of the clay. During electroosmosis many other physicochemical phenomena contribute to alter the soil properties. A case history documented by Bjerrum et al. (1967) showed that the increase in shear strength of a quick clay treated by electroosmosis had significantly exceeded that predicted from consolidation and dewatering alone. Changes in Atterberg limits were also observed. Mitchell (1970) indicates that the following processes may developduring electroosmosis: (a) ion exchange, (b) ion diffusion, (c) generation of osmotic and pH gradients, (d) desiccation from heat generation at the electrodes, (e) mineral decomposition, (f) precipitation of secondary minerals, (g) electrolysis, (h) hydrolysis, (i) oxidation, (j) physical and chemical absorption, and (k) fabric changes. The combined effect of all the processes is included in the complex phenomenon of electrokinetic stabilization.

Among metals, aluminum anodes have been the most effective in providing an irreversible stabilization of clay soils (Zaslavsky and Revina, 1965). A simplified explanation of electrokinetic stabilization with aluminum anodes is that the clay is altered to an Al^{+3} form either by movement of aluminum ions from the clay lattice or by the picking up of aluminum ions from the electrode. Unfortunately, the irreversible hardening or stabilizing effect occurs only in the immediate vicinity of the anode. Results of laboratory experiments have shown that the affected zone may be only about one-tenth of the distance between the electrodes (Murayama and Mise, 1953). To increase the volume of soil affected, certain chemicals have often been released at the anode to be dispersed through the soil by electroosmosis. When chemicals are added, electrokinetic stabilization is commonly termed electrochemical stabilization. Theoretical considerations, experience, testing, and techniques of applying electroosmosis and electrokinetic (or electrochemical) stabilization are discussed by Casagrande (1952a, 1952b, 1953); Kozan and Fenwick (1961); Zaslavsky and Ravina (1965);

Mitchell (1970); O'Bannon 1971; and Robnett et al. (1971).

Numerous chemicals have been evaluated in the laboratory for application as electrochemical stabilizers. A comprehensive laboratory evaluation was conducted by Esrig (1964) which determined the effect of various chemicals on stabilizing an illitic clay. In these laboratory experiments, the chemical solution was introduced at the anode under a voltage gradient of 1 v/cm for various time durations. The anode was a carbon rod, while the cathode was either brass or steel wire mesh. Shear strength measurements to determine the stability effect were made by vane shear tests. Increase in shear strength due to consolidation and electrochemical effects is listed separately in Table 16. Results showed that sodium silicate (Na_4SiO_4) and calcium chloride ($CaCl_2$) were most effective although all the chemicals tested resulted in a gain in shear strength.

Adamson et al. (1966a) conducted laboratory electrochemical stabilization studies on a bentonitic, disaggregated tuff sampled from the failure plane of a landslide. Saturated solutions of $CaCl_2$ and $Al_2(SO_4)_3$ were introduced to the soil through three cylindrical iron anodes (3/4 in. in diameter). An aluminum screen served as the cathode. Direct shear test results showed an increase in shear strength; cohesion increased from 225 to 425 psf and ϕ increased from 26 to 28 deg. Adamson et al. (1966b) also tested sand containing only small amounts of clay minerals (1.5 to 3.5 percent). Calcium chloride and aluminum sulfates were used. There was little change in the angle of internal friction; however, cohesion increased from values as low as 0 to 20 psf to values of 120 to 200 psf. Results indicated that only small amounts of clay need to be present for electrochemical treatment to be effective.

Field experience has shown that electrokinetic (or electrochemical) stabilization can be effective in stabilizing fine-grained soils although difficulties may be encountered in affecting a large volume. A few cases are summarized in Table 17. An increase in strength was noted by Guseva (1966), Talme (1968), and Zhinkin (1952). Holtz (1959) applied the technique in an attempt to stabilize expansive soils. Favorable stabilization was limited to a zone within 4 ft from the anode. The

Table 16. Results of electrochemical stabilization tests conducted by Esrig (1964).

Chemical Solution*	Concentration	Time hr	Shear Strength (Vane Shear Test) Initial psf	Final psf	Strength Increase Due To Consolidation psf	Strength Increase Due To Electrochemical Treatment psf
KOH	0.001 N	66	25	160	120	40
	0.01 N	120	50	225	140	85
	0.01 N	94	30	450	130	320
	1.0 N	26	20	335	40	295
Distilled water	-	140	17	155	130	25
$CaCl_2$	55.5 g/ℓ	94	4	405	8	397
	55.5 g/ℓ	98	125	425	160	265
	73.5 g/ℓ	42	3	255	19	236
NH_4Cl	62.5 g/ℓ	96	4	110	8	102
KH_2PO_4	44.0 g/ℓ	71	4	210	7	203
$FeSO_4$	70.0 g/ℓ	70	4	35	4	31
Na_2SO_4	71.0 g/ℓ	96	5	265	6	259
$Al_2(SO_4)_3$	111.0 g/ℓ	66	4	75	4	71
	111.0 g/ℓ	43	3	65	4	61
Na_4SiO_4	50.0 ml/ℓ	73	4	375	8	367
		96	4	335	9	326
		90	4	680	60	620
		144	4	600	30	570
		71	3	277	13	264
		92	3	200	6	194
	282.0 ml/ℓ	95	3	60	9	51
		46	3	190	88	102
		96	4	635	27	608
		72	11	525	33	492
		8	12	50	20	30
		24	10	300	20	280
		48	10	280	30	250
		48	12	280	43	237
AM-9	10% by weight	23	4	95	8	87
AM-9**	10% by weight	2.5	20	60	50	10
Arquad 2HT-75	1% by weight	96	4	48	9	39
Aliquat H226	1% by weight	99	4	50	7	43
Aliquat H226 †	1% by weight	44	3	235	14	221
Chempact	2.0 ml/ℓ	19	4	40	6	34
	2.0 ml/ℓ	48	4	45	6	39

Note: N = normal solution.

* Chemical solutions introduced at the anode under a voltage gradient of 1 v/cm.

** Voltage gradient 5 v/cm soil initially treated with 1NKOH.

† Aluminum anode used.

Table 17. Field applications of electrochemical stabilization.

Reference	Soil Conditions	Chemical	Electrode Type	Energy Input or Voltage Gradient*	Results
Guseva (1966)**	Damp soil beneath building	Ca & Mg Salts 10 percent solution		Voltage gradient of 10 v/cm	Load bearing capacity increased from 1.5 Kg/cm^2 to 5-10 Kg/cm^2
Talme (1968)**	Fine grained	35 percent solution $CaCl_2$	Steel	0.5 v/cm	28 percent increase in shear strength with maximum of 1.3 psi observed for undisturbed post-treatment samples and a 375 percent increase in remolded post treatment strengths with a maximum strength of 1 psi
Zhinkin (1952)	Railroad embankment fill (21-35% water content)	4 percent $CaCl_2$	Iron	46 $kwhr/m^3$	Cohesion of soil increased from 0.04 kg/cm^2 to 0.12-0.15 Kg/cm^2
Holtz (1959)	Expansive clay beneath canal lining	7 percent KCl+3 percent $AlCl_3$	Aluminium		Favorable stabilization about 4 ft from anodes
Louisiana Highway Department (1964)	Medium-heavy clay	10 percent $Ca(OH)_2$ lime slurry	Steel	0.75-2.0 v/cm	Water movement was satisfactory, but no appreciable amount of lime migrated and strength improvements were not obtained
Bally and Antonescu (1963)**	Loess soil	Na_4SiO_4 and $CaCl_2$		10-15 $kwhr/m^3$	Electrical treatment allowed a decrease by 50 percent in the amount of sodium silicate solution required to treat a given volume. Time required for treatment was reduced by 80 percent
Dearstyne and Newman (1963)**	Saturated clayey subgrade under airport runway	2 percent solution Aliquat H226	Aluminium		Pavement deflections under 45,000 lb load decreased from 0.2 inches to 0.010 inches. Effectiveness of Aliquat H226 on results was unknown
O'Bannon (1971)	Montmonillonitic chinle clay	4-5 percent KCl+C-16 wetting agent	Steel-rebar	0.6-1.0 v/in.	Favorable decrease in heaving

* v = volt; kwhr = kilowatt-hour.
** As reported by Robnett et al. (1971).

Louisiana Highway Department (1964) was unsuccessful in migrating lime slurry by electroosmosis. Bally and Antonescu (1963) pressure injected a sodium silicate and calcium chloride solution in combination with the application of an electrical potential. Apparently the electrical treatment allowed a decrease by 50 percent in the amount of sodium silicate solution required to treat a given volume. Time required for treatment was reduced by 80 percent. Other successful applications were reported by Dearstyne and Newman (1963) and O'Bannon (1971).

Electrokinetic methods are most appropriate for treatment of fine-grained soils; consequently, such treatment may effectively drain and strengthen zones of degraded shales (in particular, the clay and silt matrix material). The technique may prove advantageous in cases where gravity drainage and/or grouting has been unsuccessful due to the low permeability of the degraded materials or where overburden is inadequate to allow sufficient grout pressures, such as near the toe of the embankment. Laboratory and field tests will be necessary to determine the effectiveness and economic feasibility of electrokinetic treatment of shale embankments. Tests of degraded and intact shales will be required. Possibly, the treatment of intact shales will decrease their susceptibility to degradation. Initial testing should be conducted with the chemicals that have been most effective, i.e., solutions of sodium silicate, calcium chloride, and aluminum salts.

Ion exchange. Ion exchange, a patented technique held by Ion Tech, Inc., of Daly City, California, has been successfully used for treating landslides (Mearns et al., 1973; Arora and Scott, 1974). The technique consists of treating the clay minerals present in the soil mass with a concentrated chemical solution.* The type of solution added depends upon the clay minerals present and the chemistry of the groundwater. After selection of the appropriate chemicals, the solution is applied to the soil through cracks and/or drill holes. Alteration of clay properties occurs as ions from the chemical solution migrate through the soil

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* Chemicals not identified in reference. Ion Tech, Inc., selects chemicals based on laboratory tests of the materials to be treated.

and replace the original cations of the clay. According to Mearns et al. (1973), the conditions necessary for successful ion exchange treatment of landslides are:

- a. A clearly defined surface or zone of failure containing clay minerals.
- b. Saturation of the clay.*
- c. Cracks and/or borings for the introduction of chemicals.

The California Division of Highways reported the apparently successful use of ion exchange to correct a landslide involving a mass approximately 900 ft long and 400 ft wide and having a maximum thickness of 100 ft (Mearns et al., 1973). The slide appeared to be an old landslide reactivated by construction of a highway cut. Initial corrective action consisting of slope flattening and installation of horizontal drains proved ineffective in controlling the slide. The slide mass involved a mixture of several soil types containing significant percentages of clay minerals. Mineralogical analyses showed the clay minerals to be halloysite, a vermiculite-chlorite complex, and kaolinite. Movement appeared to be occurring on an interface between the slide mass and the underlying bedrock. Treatment consisted of introducing a mixture composed of 70 lb of dry chemicals, 15 gal of a strong acid, and 40 gal of water in quantities of 15 to 16 gal per injection point. Subsequently, the mixture was altered to 50 lb of dry chemicals, 10 gal of acid, and 45 gal of water. A comparison of slope inclinometer and posttreatment ground survey data obtained immediately before and 1 yr after treatment showed very little movement of the slide mass. Frequent subaudible rock-noise monitoring conducted during the year following treatment also indicated that the slide had stabilized. However, since rainfall amounted to only 65 percent of the normal rate for the year following treatment, the stabilizing effect of the chemical treatment could not be determined.

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* While saturation is a necessary condition, seepage during the wet season may be detrimental. Mr. E. D. Graf, President of Ion Tech, Inc. (personal communication, 1974), has indicated that the ion exchange treatment should be applied at the beginning of the dry season for best results.

Arora and Scott (1974), members of Ion Tech, Inc., reported several successful landslide corrections using ion exchange. In one case a split-level hillside residence founded on concrete piles and spread footings was suffering extensive settlement due to downhill creep. Corrective action consisted of applying chemicals through 1-1/2-in.-diam holes 4 ft deep and setting 5-ft centers around the periphery of the house. Triaxial compression tests conducted 14 months after treatment indicated approximately a 20 percent increase in the angle of internal friction and a 90 percent increase in cohesion.

In another case a landslide involving a water tank founded on a hillside cut and fill section overlying plastic foundation materials was stabilized by applying chemicals through 1-1/2-in.-diam holes placed on 2-ft centers around the water tank. A comparison of pretreatment and posttreatment laboratory tests showed a reduction in plasticity index from 35 to 17 and an increase in ϕ from 9 to 22 deg.

The few documented case histories summarized above indicate that the technique may be beneficial in treating deteriorated shale fills; however, experimentation is necessary to better define its capabilities. The ion exchange method is conceptually similar to the unpatented drill-hole lime technique discussed previously.

Reconstruction

Reconstruction involving the removal and replacement of large portions of the fill or foundation materials has often been applied in remedial treatment of shale embankments. The technique is costly, but may be necessary where extensive movements and/or shale degradation has weakened the fill or underlying soils. The technique has generally been successful, but new failures have developed, primarily in sidehill fills. For the technique to be effective, material should be removed to a depth well below the known or estimated shear zone of failure surface. In addition, excavation of existing materials can allow optimal location of drainage blankets to intercept subsurface seepage into sidehill fills. For example, in repair of SLIDE-3 on I-74, Indiana, unstable fill material and residual soils were removed, the embankment was reconstructed to a flatter slope with drainage blankets and collector pipes placed on the foundation bench slopes as shown in Figure 62.

Where feasible, excavated material should be replaced with a free-draining granular fill. Degradable shale materials taken from previous or new borrow sources should be broken down and compacted in replacement of excavated fill. Clay soils used in replacement should also be adequately compacted when placed.

Reuse of removed materials is desirable, especially where waste areas and/or new borrow sources are not readily available. Excavated shale fill material can be used in reconstruction; however, in most cases such material will require reworking and/or drying before replacement to insure adequate compaction. Engineering properties of poor quality fine-grained fill removed from the embankment may be improved by mixing the fill with lime prior to replacement. Abrams and Wright (1972) reported that lime treatment by excavating, mixing (typically 3 to 4 percent lime by weight), and replacing had been used successfully by the Texas Highway Department in stabilizing plastic clays, but that in many cases failure reoccurred in the previously stabilized slopes. The inadequacy of the lime treatment was attributed to one or more of the following items:

a. Failure to remove materials to a depth below the failure zone.
b. Poor mixing and/or compaction during replacement.
c. Blockage of internal drainage by the low-permeability lime-soil mixture.

The U. S. Army Engineer District, Memphis, has recently treated unstable main-line levees by removing the fat clays (liquid limit = 80 to 100 percent), mixing them with lime, and replacing. Lime was mixed with the clays in the proportion of 5 percent by volume based on results of laboratory tests. The project is being conducted on an experimental basis (Pitchford, 1975).*

* L. Pitchford (1975), Personal communication, U. S. Army Engineer District, Memphis.

A frequent problem in removing materials is maintaining stability of the remaining embankment. Drainage measures, such as horizontal and vertical drains or pumped vertical wells, can be used to increase stability prior to and during excavation. Vertical pumped wells (100 ft in depth) were installed prior to removal and replacement of the failed embankment section at sta 840+00, I-75, Tennessee (Figures 3-6). If the distressed condition is largely a function of seasonal rainfall, removal of materials can probably be conducted during a dry season without supplementary drainage measures. The large embankment shear trench shown in Figure 84 was to be constructed during a dry season. The shear trench will be constructed of coarse, nondegradable rock fill to provide increased shear resistance and facilitate drainage of the sidehill fill. A drainage blanket (minimum thickness of 3 ft) should also be constructed along the trench bottom and backslope to prevent future clogging of the backfill and/or collector pipe (see discussion, page 126).

Other measures supplementary to removal and replacement of materials may be needed to ensure permanent stability. For example, it is important to provide subsurface drainage measures in remedial treatment of sidehill fill foundations. Excavation should extend well below the shear surface and, where feasible, residual soils should be removed and benches cut into the foundation rock. A drainage blanket should be placed before reconstruction of the fill. Repair of three fills along I-75, Tennessee, involved removal and replacement of failed materials with the addition of horizontal drains and rock buttresses as shown in Figures 48-50. Only after conducting stability analyses and considering local experience can the need for removal and replacement of materials and any supplementary measures be established. Hopefully, other less costly and less time-consuming techniques can be effective, especially if embankment problem conditions are recognized and evaluated before large settlements and/or complete slope failure.

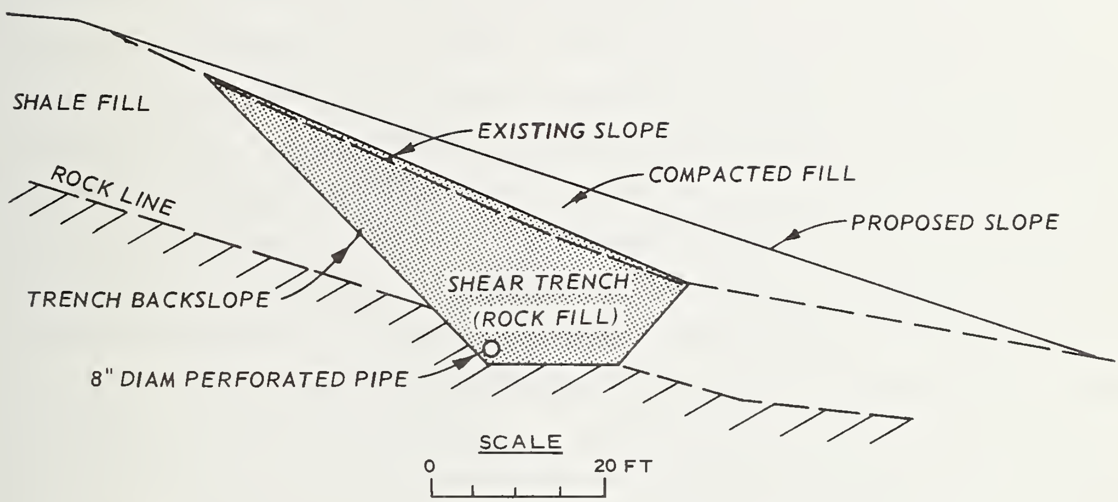

Figure 84. Alternate correction scheme, Blue Grass Parkway, milepost 21, Kentucky (courtesy of the Kentucky Department of Transportation).

V. CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Embankment problems. Embankment problems can generally be divided into two categories: (a) those involving settlement and possibly lateral movement and (b) those involving slope instability. Both problems arise in part from the fact that embankments are commonly constructed as rock fills with material placed in 3- to 4-ft lifts and compacted by hauling and spreading equipment. This practice can result in large voids within the fill. Even where attempts are made to place material in thin lifts and use soil compaction procedures, large voids sometimes occur because of the presence of larger more durable sandstone or limestone particles. Surface runoff and subsurface seepage can introduce large quantities of water into the embankment, especially in sidehill fills. Water is believed to accelerate deterioration of shale particles and aid in the development of zones of weak, impermeable materials which tend to prevent free drainage of the fill. Settlement and spreading of the embankment, accompanied by pavement cracking and dips in the roadway, are usually the first signs of distress and are often the prelude to the more costly problem of slope failure. Thorough evaluation is needed to determine the causes of distress at any particular site, to predict future behavior, and to select appropriate remedial measures.

Evaluation procedures. The success of evaluation procedures in predicting the future behavior of an embankment and indicating appropriate remedial measures is largely dependent upon the early detection and monitoring of embankment distress. Applicable evaluation procedures for a compacted shale embankment are determined by the character of the embankment. The instrumentation, sampling and testing apparatus, and methods of analysis presently available are adequate for use in evaluating compacted shale embankments. Conclusions regarding specific instrumentation, sampling and testing methods, and analysis procedures are presented in the following paragraphs.

Permeability tests on compacted samples of crushed shale and shale-rock mixtures for CE projects indicate the Casagrande open-standpipe porous tube piezometer is the piezometer best suited for use in compacted shale embankments. However, falling head tests should be conducted after installation to verify that the response time is adequate for the rates of change in groundwater likely to occur and also to provide an estimate of the permeability of the embankment. Sufficient numbers of piezometers should be installed to adequately define the groundwater levels (including perched water) within the embankment, adjacent sidehill or cut section, and foundation.

Surface settlement and movement measurements are best made using conventional surveying methods for monitoring movements of markers, pins, or stakes placed on the embankment surface. Simple logitudinal and cross-sectional profiling, without the installation of markers, is believed to be the most useful means of determining settlement and bulging. Measuring the changes in crack openings with time is also a convenient means of defining movement rates. The use of extensometers for monitoring surface movements may have specific applications; however, for most investigations simple taped measurements are sufficient. While devices installed during construction are capable of monitoring both lateral and vertical movements, these devices are applicable only where evaluation is planned during the design stage. Extensometers are capable of measuring settlement at different depths in a vertical hole and lateral movement at different locations in a horizontal hole; however, the need for multiple determinations of settlement and lateral movement does not justify wide use of entensometers. The most frequently used means of monitoring lateral movements within embankments is with inclinometers. Inclinometers provide a convenient method of obtaining the depth, magnitude, and rate of lateral movement at a particular location on an embankment. Settlement measurements at different depths can also be obtained from cased inclinometer holes. The location of a failure plane or zone of large movement can be easily delineated from a series of inclinometer observations, provided the bottom of the inclinometer casing is below the depth of movement.

Undisturbed sampling may be possible in compacted shale embankments using thin-walled, fixed-piston samplers for soillike embankments and double-tube core barrel samplers for rocklike embankments. When undisturbed samples are obtained, laboratory tests can include unit weight determination, permeability, consolidation, triaxial compression, unconfined compression, and direct shear. Disturbed samples can be obtained from boreholes, test pits, or from the surface. Disturbed samples can be used to determine material type and mineralogy, water contents, Atterberg limits, specific gravity, gradation, and compaction characteristics.

In situ investigations can provide a means of determining strength properties directly through field testing or merely permit a detailed examination of the fill. Borehole instruments such as the Menard pressuremeter and the Iowa shear test device are devices which show the most promise for use in compacted shale embankments. The Menard pressuremeter has been used for testing a variety of materials including cohesive and cohesionless soils, intact rock, and shale fills. However, the pressuremeter does not provide an immediate determination of strength properties; correlations of pressuremeter strength with laboratory strengths or strengths from other in situ devices must be made to determine the true strengths of the embankment materials. The Iowa shear test device has been used in many different soil types and in one shale fill. However, in hard soils or rock the shear plates do not penetrate sufficiently under the normal load capacity of the device and this device cannot be used successfully in these materials. The Iowa shear test device shows the most promise for use in cohesive soil fills. The Texas device requires redesign and proof testing before it can be considered for in situ testing of shale fills. Large-scale testing such as direct shear, triaxial testing, and torsional shear are expensive, time-consuming, and may involve many tests to determine the strength properties of the embankment. Plate loading tests performed at different depths will provide modulus values which would indicate the presence of weak zones of materials. When performed periodically these tests would also indicate a change in modulus with time if deterioration was occurring. However, the plate loading test performed at different depths would be too expensive for general use. Visual examinations of the walls of a borehole using cameras would be useful in determining the character of a fill. Geophysical methods such as moisture-density probes, seismic refraction or reflection, electrical resistivity, and continuous vibration have generally not been perfected to the point where they could be successfully used in evaluation of a compacted shale embankment.

Conventional slope stability analyses are adequate for estimating factors of safety against sliding and are adaptable to back analyses (when a slip surface is defined) in determining shear strengths for remedial measures. Predictions of continued settlement can best be made by extrapolation of a plot of settlement observations versus time.

Remedial treatment. Several techniques can be applied in remedial treatment of shale embankments. Each technique offers certain advantages and disadvantages which can make it attractive or undesirable in a given situation. Excessive settlements and slope failures have been most prominent in sidehill fills, largely because of their susceptibility to infiltration of water from the adjacent natural ground. Consequently, greater emphasis should be placed on the application of subsurface drainage measures to stabilize sidehill fills.

Horizontal drains used alone or in combination with vertical drains appear to be the most practicable method of controlling seepage through sidehill shale fills. Horizontal and vertical drains are advantageous because installation does not require excavations which can cause further instability. In addition, vertical drains may be ideal for intercepting seepage through sidehill fill foundations where thin water-bearing strata or flow channels such as open fractures and bedding planes are separated by strata of impervious material.

Other types of subsurface drainage measures which have been used successfully are pumped wells, trench drains, drainage blankets, bench drains, and toe drains. Pumped wells are most useful as a temporary measure to maintain stability while applying other remedial measures. Deep trench drains placed behind sidehill fills or along the downhill slopes (i.e., shear trenches) are an excellent means of intercepting subsurface seepage. However, these drains have the disadvantage of requiring large amounts of excavation and reconstruction making their installation difficult and costly. Expensive excavation and reconstruction are required for installation of drainage blankets and bench drains which are most effective when placed directly against aquifer layers beneath the fill. Toe drains are relatively inexpensive to install and can aid in preventing loss of toe support which can lead to progressive slope failure.

Infiltration of surface runoff may contribute to shale degradation and cause an increase in shear stresses. Surface drainage can be improved by sealing cracks in the highway pavement and embankment slopes and filling (or grading) low areas of the fill slopes to prevent ponding of water. Slope surfaces can be chemically treated to improve runoff. Significant infiltration may also occur where drainage ditches are left unpaved or where paved ditches are heavily cracked.

Flattening of embankment slopes, commonly with the addition of a berm (or buttress), has been an effective method of stabilizing shale fills; however, it should be recognized that the usual practice of flattening the slopes by adding fill can add both driving and resisting forces. Consequently, the addition of a berm (or buttress) on the lower portion of the embankment may be more advantageous than overall slope flattening. The main disadvantage of flattening slopes and adding berms is the need to extend right-of-way and acquire large quantities of borrow materials. This problem can be somewhat alleviated by using retaining walls.

Retaining walls have received little use or consideration in repair of shale fills. Costs are usually justified only in cases where right-of-way is severely limited. Difficulties in providing an adequate foundation and lengthy construction time inhibit the application of retaining walls as a general remedial technique. Crib walls, gabion walls, and reinforced earth walls are advantageous because they can withstand large differential settlements and are constructed using relatively free-draining materials.

Piles can be placed in one or more rows along an embankment slope to form a type of retaining wall. Small diameter piles can be driven in place to provide temporary support of failing slopes. A primary weakness of pile walls is the potential for shear beneath or between the piles. Consequently, the piles must be closely spaced and anchored in firm material. For relatively large fills, closely spaced large-diameter concrete cylinder piles may be advantageous; however, additional field experience with the technique is needed to determine its applicability.

The stabilizing effect of retaining walls can be increased by means of anchored ties placed in holes drilled beyond the zone of failure. Anchored retaining walls may be advantageous for support of short buttress fills adjacent to sidehill embankments. Additional field experience is needed to determine the capabilities of anchored retaining walls for remedial treatment of shale embankments.

Among the lime, cement, and chemical treatment methods reviewed, cement grouting appears to be the most practical technique. Cement grouts harden rapidly and have successfully arrested settlements of shale embankments by filling large voids within the fill. Where slope shear failures have developed, cement grouts may effectively stabilize shale fills by penetrating along shear surfaces. This phenomenon has been observed in numerous cases where cement grouts have stabilized shear failure in clay soils, including saturated clays. Cement grouting may be sufficient to stabilize fills containing a high percentage of interconnected voids (porosity of 20 to 40 percent), provided the fills are not subject to subsurface seepage. When applied to sidehill fills, it is likely that supplementary drainage measures will be necessary to counteract the sealing effect of the cement.

Chemical grouting, lime stabilization via drill-hole lime or lime-slurry injection, electrokinetic stabilization, and ion exchange may be beneficial in cases where cement grouts have been unsatisfactory because of the presence of fine-grained, clayey soils of low permeability. However, expert guidance is normally required when applying these techniques and the effectiveness of the methods is difficult to predict. Since these techniques have not been used in remedial treatment of shale fills, field trials or experimentation will be needed to determine the practicability and application of these methods.

Reconstruction involving the removal and replacement of materials may be necessary where extensive movement and/or shale degradation has occurred. Reconstruction has been used often and has generally been successful; however, reconstruction of large embankment sections can be expensive. In many cases large-scale reconstruction can probably be avoided by applying surface and subsurface drainage techniques soon after defining the problem.

Recommendations

Evaluation procedures. The methodology outlined in Figure 85 (see inside back cover) is recommended for use in the evaluation of compacted shale embankments. A discussion of the methodology and specific recommendations regarding some of the steps is presented in the following paragraphs.

In many instances States have overlaid problem pavement sections with asphalt as a solution to settlement or have repeatedly repaired sloughing of embankment side slopes. These types of solutions are sometimes temporary and continued maintenance does not correct the problem. It is therefore recommended that procedures be established whereby records are kept and examined periodically to locate sections of roadway where frequent maintenance has been performed. These sites should then be inspected by geotechnical personnel in an effort to identify the cause of the distress. At times the exact cause, such as inadequate surface drainage, can be identified and appropriate remedial action initiated; however, the cause of excessive settlements and/or lateral movement of the embankment or slope instability will require further investigation.

When probable causes are determined to lie within the embankment or foundation (i.e., not restricted to pavement or base course distress), it is recommended that steps be taken which will develop the information needed to define the problem. These steps consist of reviewing the history of design and construction, along with all available geologic and soils data, and establishing a monitoring program for the embankment. The monitoring program should include, as a minimum, settlement surveys (longitudinal and sectional profiles) and the installation and observation of piezometers and inclinometers. Disturbed samples should be obtained and undisturbed sampling should be attempted at different depth intervals during boring operations for the installation of piezometers and inclinometers. The character of the embankment materials should be established at that time. From analyses of monitoring data (prepared in summary plots), and a review of all information, especially settlement profiles and inclinometer observations, it should be possible to determine if the problem lies within the embankment or is related to foundation distress. For problems related to the foundation, a study of the strength and compressibility of the foundation materials should be undertaken.

If the origin of the problem is within the embankment, the question can be asked, "Are deformations large enough to mobilize the maximum shear resistance?" The answer to this question lies in the evaluation of instrumentation data and in the results of shear strength tests on any undisturbed samples obtained. If settlement and lateral movement are increasing at a slow rate and inclinometer data indicates progressive movement over a considerable depth range at a slow rate but with no definite failure zone, then the answer is probably "no." In this case, it is recommended that monitoring be continued with repeated checks made on the ultimate settlement and/or lateral deformation as predicted by plots of settlement and lateral movement versus time. Should the rate of movement increase and a failure plane develop at anytime during the monitoring, the answer to the question would then be "yes" and lateral movement and settlement would be related to slope instability. If lateral movement and settlement is related to slope instability, it is recommended that strength properties be determined in order that analyses for arriving at the factor of safety against sliding of the embankment can be performed.

Of the various means of determining strength properties, it is recommended that undisturbed sampling and laboratory testing be accomplished where possible. The feasibility of collecting undisturbed samples will have been determined during the installation of piezometers and inclinometers. Borehole devices are recommended where undisturbed samples cannot be collected or to supplement undisturbed sampling. The Menard pressuremeter is recommended for use in soil and rock fills, but may be of limited value in rock fill due to the presence of sharp pieces or large, hard particles. The Iowa shear device is recommended for use in soillike fills. Large-scale in situ shear tests are recommended only where other tests are not believed to give representative results or where other tests cannot be conducted.

Test results must be reviewed to determine if they are representative of the strength of the embankment material. If test results are not considered to be a reliable representation of the strength properties, either additional testing should be conducted or strength parameters should be selected based on previous experience and engineering judgment. After selection of strengths, the stability analyses can be made.

In analyzing the problem, the first important factor is whether a failure zone has been defined. If a failure zone has not been defined, it is recommended that strengths be selected from laboratory tests and/or in situ tests and the stability analysis be performed to determine the critical failure surface (the surface giving the lowest factor of safety). The pore water pressures existing within the fill, as defined by piezometers, should be used in the analysis. Additional analyses should be made for any changes in the water table that are likely to occur (especially during the wet season). After the analyses have been performed, an evaluation of the adequacy of the embankment stability should be made. If the embankment is judged inadequate, remedial measures should be initiated. If the embankment is judged adequate, monitoring should be continued with periodic evaluation of the data.

Back analyses are recommended where a failure plane has been defined by surface measurements and/or inclinometer observations. Results of these analyses should be compared with laboratory testing and/or in situ testing and strengths selected for design of remedial measures.

Remedial treatment. The primary consideration in remedial treatment of shale fills should be surface and subsurface drainage measures. When other remedial techniques are applied, drainage measures are usually a necessary supplement. Design of drainage installations should be conducted by experienced engineers and it is recommended that designs be flexible to allow alterations based on field performance. The efficiency of subsurface drainage installations should be checked by monitoring drainage discharge rates and changes in groundwater table elevations and/or piezometric head.

As a part of any remedial treatment scheme, efforts should be made to minimize infiltration of surface runoff into shale fills. Cracks in the highway pavement and embankment slopes should be sealed and low areas of the embankment slopes should be filled (or graded) to prevent ponding of water. Median and side ditches should be paved and existing paved ditches inspected and repaired as necessary. Where significant settlements have occurred, buried pipes connected to surface drains should be checked for breakage, blockage, or separation. Damaged pipe sections should be replaced or new sections installed to discharge the collected water away from the embankments by a more direct route.

Control of subsurface seepage is likely to be the key factor in treatment of sidehill fills. Horizontal drains should be installed in an attempt to arrest subsidence and/or lateral movements of sidehill fills before the development of an extensive slope failure. The majority of the horizontal drains should be initiated at the base of the downhill slope and should penetrate deep enough within the natural ground behind the fill to intercept subsurface seepage before it reaches the fill. If the natural groundwater table is observed high within the fill or natural slope behind the fill, horizontal drains should also be installed through the fill or in the hillside above the fill. Additional horizontal drains located entirely within the fill may be required to drain saturated fills of low permeability. Horizontal drains should be installed using established procedures whereby steel casing is drilled in place and removed after the drain pipe is installed. These procedures eliminate the potential problem of borehole caving which can occur where drill pipe is removed prior to installation of the drain pipe. Nonperforated pipe should be used to a distance of 10 to 20 ft from the drain exit to minimize blockage caused by root growth through the perforations. Where soil fines may be carried through the perforations, the pipes should be wrapped with filter cloth before installation. Water intercepted by horizontal drains should be discharged into paved ditches or collector pipes and carried away from the distressed fill. Horizontal drain installations should be periodically inspected and cleared as necessary to maintain their free draining capability.

Where thin water bearing layers are separated by impervious materials, a row of closely spaced interconnected vertical drains (diameter of 2 to 3 ft) should be installed behind the fill to intercept seepage. These drains are installed by drilling vertical boreholes and interconnecting them by belling out the bottom of each borehole. If necessary, well screens or filter cloth should be installed prior to backfilling to prevent carriage of fines into the drain. Also, a pipe should be placed in some of the boreholes to allow later monitoring of water levels. Finally, the interconnected boreholes should be backfilled with a permeable nondegradable material. The drains should be completed a few at a time so that boreholes are left open only for a short period to prevent borehole caving. Intercepted water should be discharged through a system of horizontal drains intersecting the belled out sections or individual vertical drains.

Where distress in the toe region persists after installation of horizontal (and possibly vertical) drains, it is recommended that buried toe drains be installed. Excavation for toe drains should be conducted during a dry season.

Where extensive movements and/or shale deterioration have caused large reductions in shear strength, drainage measures alone may not stabilize the fill. In these instances it is recommended that one or a combination of the following remedial methods be used in addition to drainage measures:

a. Reconstruction involving removal and replacement of weak materials.

b. Slope flattening and/or berms (or buttresses).

c. Retaining walls.

d. Cement grouting.

Important factors to consider in making the final selection are:

a. Strength of foundation and fill.

b. Right-of-way limitations.

c. Availability of materials.

d. Construction feasibility, costs, and time limitations.

e. Local experience.

f. Advantages and limitations of each remedial method.

Recommendations concerning the important aspects of reconstruction, slope flattening and berms, retaining walls, and cement grout are given in subsequent paragraphs.

When weak unstable materials are to be replaced, excavation should be carried to a depth well below the shear zone. In treating sidehill fills, a drainage blanket (minimum thickness of 3 ft) should be placed prior to reconstruction. Where feasible, residual soils should be removed and the drainage blanket placed on benches cut into the foundation rock. Where complete removal of weak materials is not economically feasible, deep shear trenches (also serving as interceptor trench drains) should be incorporated into the downhill slope or beneath a berm. Where shallow failed sections are removed, it is recommended that the drainage blanket be supplemented with horizontal drains (and possibly vertical drains behind the fill) to prevent new failures involving material below the reconstructed zone. When feasible, excavated material should be replaced with a free-draining nondegradable fill. Clay soils and/or degradable shales should be compacted in thin lifts when used in replacement of excavated fill. Benching, moisture control, and special efforts to break down shale pieces may be required to achieve proper compaction. Reuse of removed materials may be necessary where borrow sources and/or waste areas are scarce; however, in most cases such material will require reworking, drying, or chemical treatment (for example, lime treatment) before replacement to ensure adequate compaction and shear strength.

When material is added to flatten slopes or construct berms (or buttresses) proper compaction and adequate drainage are essential. Where feasible, berm (or buttress) fills should be constructed of nondegradable rock. Clay soils and/or degradable shale material should be compacted in thin lifts when placed as berm fills. Benching, moisture control, and special efforts to break down shale pieces may be required to achieve proper compaction. In repair of sidehill fills, drainage blankets should be placed beneath the berm to intercept subsurface seepage. The need for supplemental drainage measures such as horizontal drains, a combination of horizontal and vertical drains, or deep trench drains should also be considered. Embankment or berm slopes adjacent to or extending into rivers and streams must be protected from erosion due to currents and wave action. These slopes may also be subjected to submergence and subsequent drawdown. These conditions should be considered in stability analyses. If necessary, those portions of embankment slopes which will be submerged should be constructed of a nondegradable free draining rock fill.

Retaining walls recommended for use in permanent support are reinforced earth, crib, and gabion walls. These retaining walls have similar advantages since they are constructed using relatively free draining materials and can withstand considerable differential settlement. The selection of a particular wall depends on cost and local experience. In construction of a reinforced earth wall it is important that the backfill consist of a granular material such as sand and gravel with less than 15 percent fines. Similarly, a granular backfill should be placed behind crib and gabion walls. A filter zone (or filter cloth) is also recommended to prevent piping of backfill or natural materials into or through crib and gabion walls.

Anchored retaining walls and large diameter cylinder piles may also be useful for permanent support. These methods should be used only on a trial basis at selected sites where a substantial savings over conventional methods could be realized.

Retaining walls consisting of one or more rows of closely spaced, small diameter piles should be used primarily to provide temporary support prior to installation of more positive remedial measures. The piles can be rapidly installed by driving them in place; however, a more secure anchorage can be obtained by placing steel piles in predrilled holes that penetrate stable rock strata and backfilling the holes with concrete. In treatment of sidehill fills, these piles should not be placed at a location, depth, or spacing which could prevent later installation of horizontal drains.

Where large deformations have been attributed to the presence of a high percentage of interconnected voids (porosity of 20 to 40 percent), cement grouting should be conducted in an attempt to fill these voids. Engineers experienced in grouting should be consulted to assist in the design of the grout mix and grouting program. Following grouting of sidehill fills, subsurface drainage measures such as horizontal drains should be installed to counteract the sealing effect of the cement grout.

The following techniques may be applicable where weak fine-grained clayey soils and/or a well-developed shear surface is present:

a. Cement grouting to penetrate along the shear surface.

b. Chemical grouting.

c. Lime stabilization (drill-hole lime or lime-slurry injection).

d. Electrokinetic stabilization.

e. Ion exchange.

These methods are not recommended for routine application. These methods should be used only on a trial basis with the aid of experts at selected sites where a substantial savings over more conventional methods could be realized.

A. APPROACH FOR PHASES I AND III

Phase I: Identification of Factors Responsible for the Deterioration of Compacted Shales

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

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

Information from the literature and field visits was used to establish a basis for identifying physical and/or chemical parameters affecting the deterioration of shales. Parameters considered to be important were mineral associations, particle degradation, clay content and sensitivity, pore water composition, and chemical alterations (i.e., change of pyrites to iron oxides and combination of sulfur into sulfates with volume change, swelling, and weakening of the shale).

Examination of selected unweathered samples of problem and nonproblem shales, collected during field trips and by courtesy of several States, was done as part of the variability study to aid in the development of meaningful parameters and to evaluate selected laboratory tests for determining deterioration susceptibility. Examinations include natural water content, X-ray diffraction (determination of gross mineral constituents), hardness, and slaking durability.

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

Data on natural moisture content, degree of weathering, effects of hydrologic cycles, and climate zones as well as construction practices were considered in attempting to identify extrinsic factors contributing to shale embankment problems.

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

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

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

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

Task C, laboratory tests. Information from the literature, results of field data and observations, and past experiences with laboratory testing of compacted shale mixtures to duplicate prototype conditions will be used to prepare a testing program for needed compaction and shear strength tests. Samples of shale mixtures used in embankments exhibiting both stable and unstable behavior obtained while conducting studies under Phases II and III, Task A, will be used. Use will be made of the WES Soils Research Laboratory compaction and triaxial shear testing equipment that accommodates samples up to 18 and 15 in. in diameter. Preliminary tests will be made to develop methods for adding desired water and determine curing conditions that are representative of prototype conditions. Tests will then be performed to determine the effects of scaled gradations with 6-in.-diam specimens on consolidation and strength properties when compared to companion tests on specimens containing near prototype gradations and maximum grain size. Specimen preparation by kneeding compaction and impact compaction will be compared to determine degradation and the most suitable procedure which duplicates field compaction methods.

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

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

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

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

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

B. ORGANIZATIONS CONTACTED

Sixteen States were designated as contacts by the FHWA. All of the highway departments in these States were questioned regarding compacted shale embankment problems. FHWA regional offices were also asked about these problems and, as a result, New York was also contacted. Several USGS offices and State geological survey offices were contacted for pertinent geologic information. Purdue University was questioned regarding research on compacted shale embankments being conducted for the Indiana State Highway Commission. CE offices were contacted for their experience with compacted shales in embankment dams. Other Federal agencies, educational institutions, and private enterprises were also contacted. A complete listing of the agencies is given below:

State Highway Departments

California	Oklahoma
Colorado	Oregon
Indiana	Pennsylvania
Kentucky	South Dakota
Missouri	Tennessee
Montana	Utah
New York	Virginia
North Carolina	West Virginia
Ohio	

Federal Agencies

U. S. Army Corps of Engineers

Division - South Pacific
District - Sacramento

Division - North Pacific
District - Portland

Division - Ohio River
District - Huntington
Louisville
Pittsburgh

Division - Southwestern
District - Fort Walton
Little Rock
Tulsa

Division - South Atlantic
District - Mobile

Division - North Atlantic
District - Baltimore

Division - Missouri River

Department of Interior

U. S. Geological Survey - Denver, Colorado
Menlo Park, California
Reston, Virginia

Bureau of Reclamation - Denver, Colorado

Universities

Purdue University - West Lafayette, Indiana

University of California - Berkeley, California

University of Illinois - Urbana, Illinois

University of Missouri - Rolla, Missouri

University of Oklahoma - Norman, Oklahoma

Other Organizations

Tennessee Valley Authority - Knoxville, Tennessee

Law Engineering Testing Company - Birmingham, Alabama

C. SAFETY FACTOR EVALUATION USING SLOPE INCLINOMETER DATA

A method of computing a safety factor where horizontal movements beneath fills are large has been formulated by Johnson (1975).* The approach compares shear strains computed from field measurements with laboratory stress-strain data from simple shear tests to estimate the mobilized shear resistance relative to the maximum shear strength. A summary of the method for application to a highway fill is as follows:

a. Compute shear strains from inclinometer data at various depths and locations including, if available, the effects of differential vertical settlements.

b. Determine the point along laboratory stress-strain curves to which field observations correspond.

c. Compute a point safety factor by dividing the maximum shear strength from laboratory tests by the mobilized shear resistance. Figure C-1 is a graphical summary of the method.

Caution must be exercised in using this method when movements have occurred before the installation of the inclinometer as the shear strain computed from inclinometer data will obviously be less than that which has occurred within the embankment giving a higher local safety factor. The method is better adapted to embankments where continuous records of movement are available.

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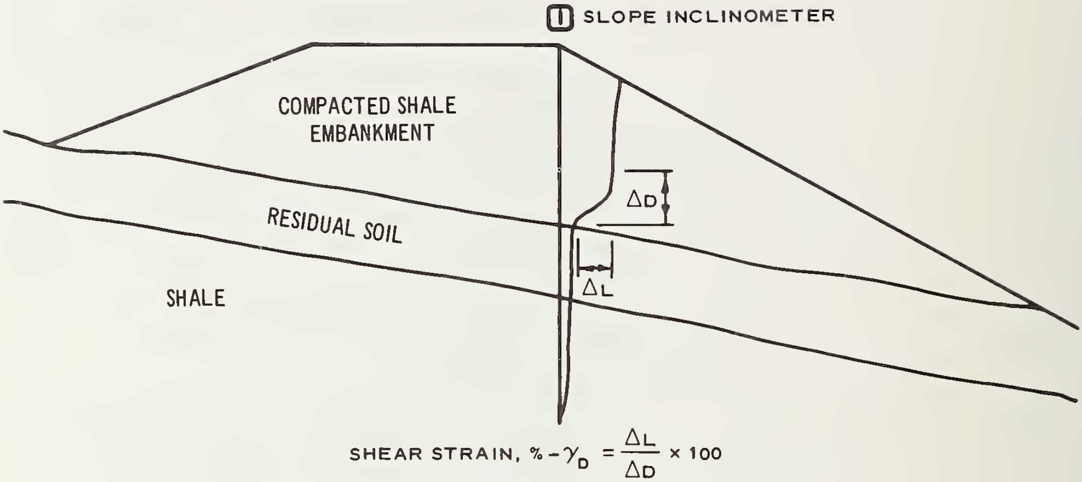
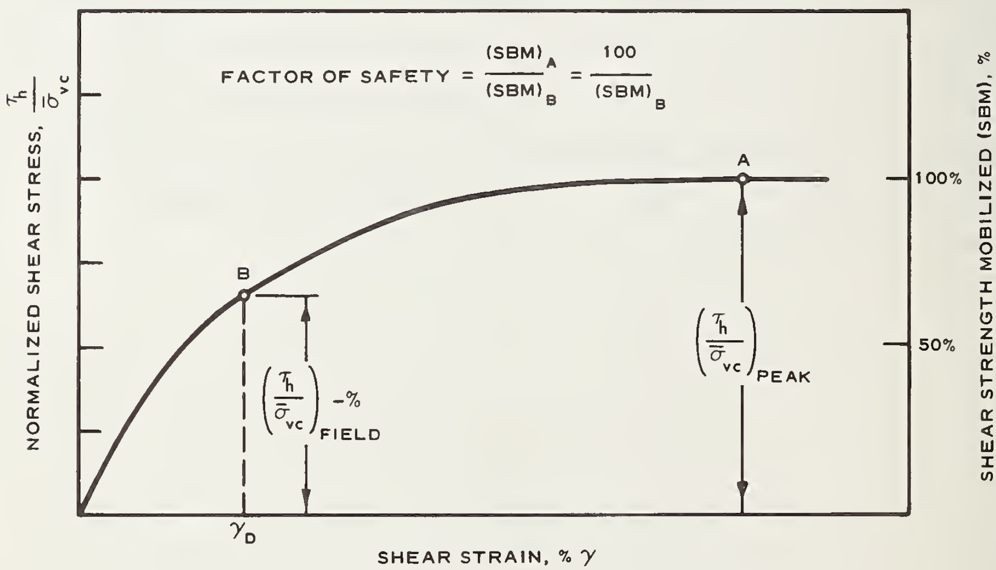
* S. J. Johnson (1975), Personal communication, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.



a. FIELD DATA



b. NORMALIZED LABORATORY STRESS STRAIN CURVE

Figure C-1. Safety factor evaluation using slope inclinometer data (Johnson, 1975).

D. REINFORCED EARTH - THEORETICAL CONSIDERATIONS

Basic Concept

The exact state of stress in a reinforced earth mass is not known; however, it is possible to set a range within which soil behavior can be characterized. Let A be a soil element within a cohesionless semi-infinite homogeneous mass (Figure D-1a) acted upon by vertical stress σ_v and horizontal stress σ_h. If the vertical stress is increased without changing the lateral strain ε_h then the horizontal stress may be increased proportionately to the vertical stress.

$$\sigma_h = k_o \sigma_v \tag{D-1}$$

where k_o is the coefficient of earth pressure at rest, and the state of stress within the element A corresponds to the at-rest condition.

Theoretical studies by Jaky (1948) which were verified experimentally have shown that k_o can be expressed as

$$k_o = 1 - \sin \phi \tag{D-2}$$

where ϕ is the effective angle of internal friction.

If the vertical stress is increased such that the soil element A begins to compress in the vertical direction and expand in the horizontal direction, then the value of σ_h will begin to decrease while shear stress will increase as shown in Figure D-1b. However, there is a limit beyond which any increment in the vertical stress may cause the soil to fail in shear, and the relationship between σ_h and σ_v under such limiting conditions may be expressed as

$$\sigma_h = k_a \sigma_v \tag{D-3}$$

where k_a is the coefficient of active earth pressure and the state of stress at point A is termed the active condition. For cohesionless

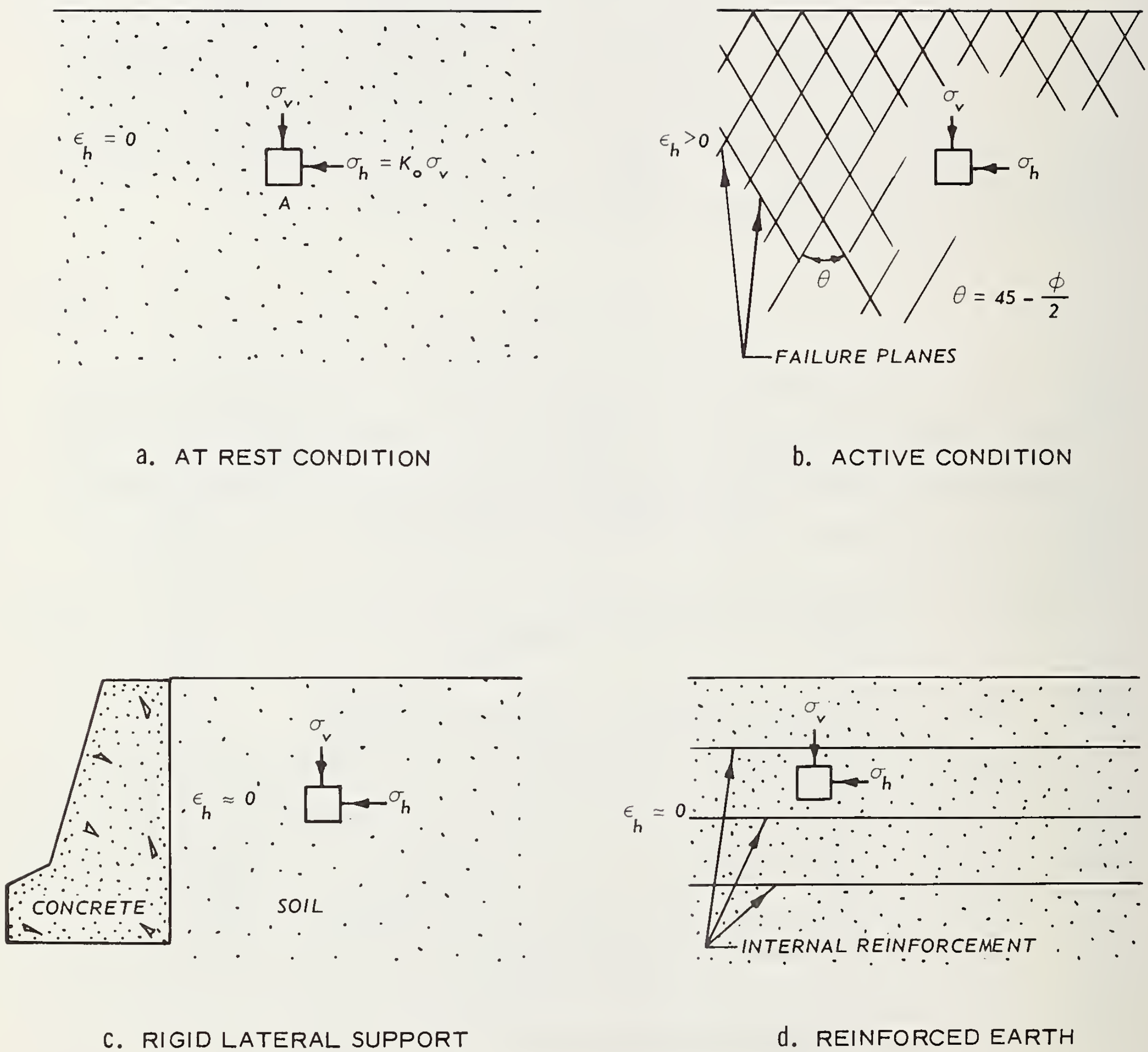


Figure D-1. Schematic of soil element under different conditions.

soils, k_a may be expressed as

$$k_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \tan^2 \left(45^\circ - \frac{\phi}{2}\right) \qquad \text{(D-4)}$$

By examining equations D-2 and D-4, it can be seen that the value of k_o is higher than k_a for any realistic value of ϕ. This implies that in order to prevent failure, the lateral strain ε_h which induces shear stress must be prevented (Figure D-1c) or reduced to a minimum value. This can be achieved by increasing the lateral stress σ_h such that the coefficient of earth pressure k may satisfy the following:

$$k_a < k < k_o \qquad \text{(D-5)}$$

where k is the ratio σ_h to σ_v for any state of stress between the active and the at-rest condition. Excessive lateral deformation that might cause active failure can be restrained internally by reinforcing the soil with frictional and high tensile material as shown in Figure D-1d. It must be noted, however, that the problem of determining the resulting stresses is highly indeterminate and depends on the physical properties of both soil and reinforcing material, geometry, and loading conditions.

Design Assumptions for Reinforced Earth Walls

To arrive at an approximate determination of the horizontal thrust acting on the reinforced earth wall (Figure D-2), under static conditions, it is necessary to make reasonable assumptions such as:

a. The wall will move laterally during construction enough to create a limiting equilibrium condition within the backfill material behind the wall. This assumption enables the applicability of the classical earth pressure theories such as those of Rankine and Coulomb.

b. The lateral movement of the wall, regardless of how small, may generate enough confining pressure through friction of the reinforcing elements such that net shear stress within the

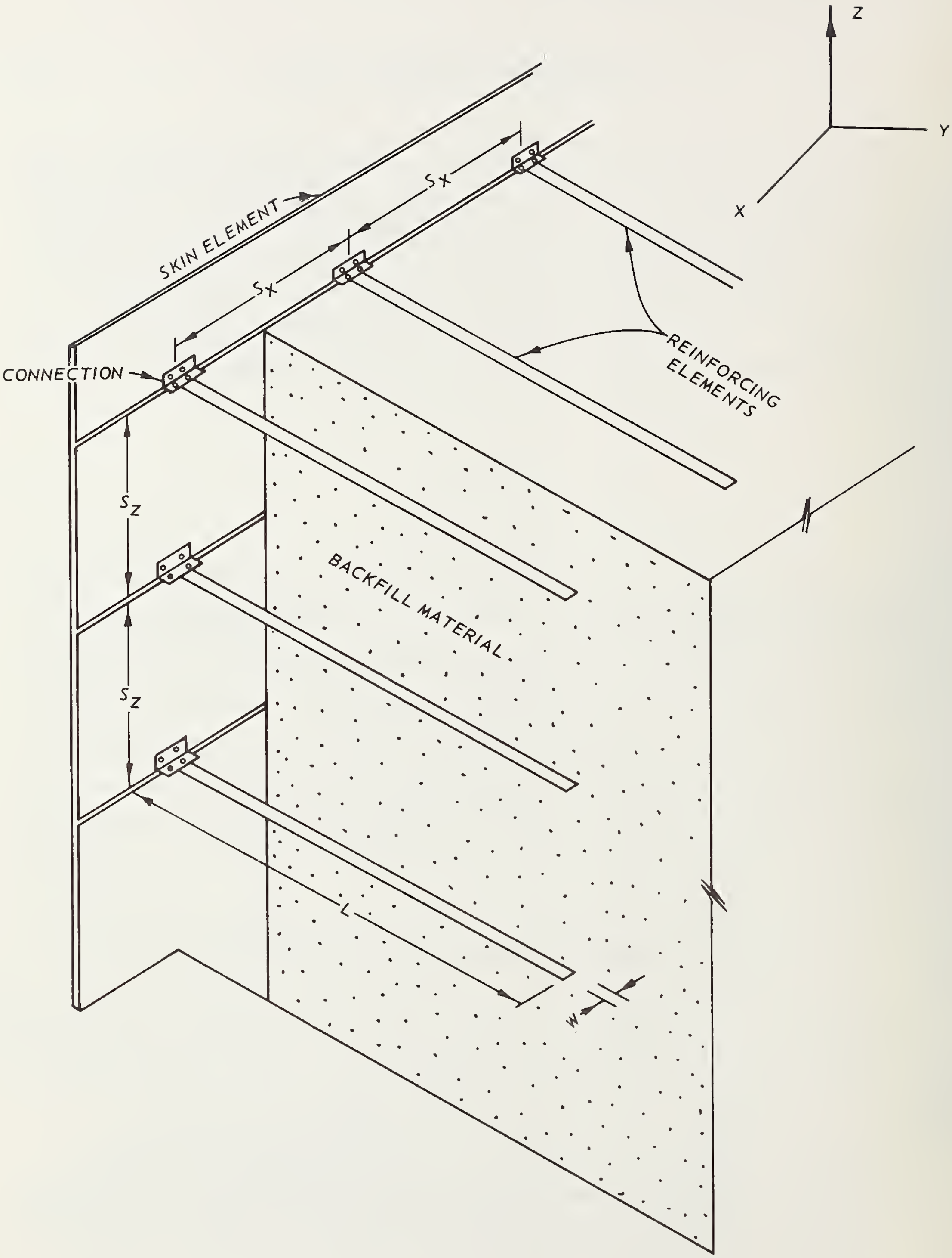


Figure D-2. Schematic of the major elements of a reinforced earth wall.

backfill is less than its maximum strength.

c. The wall is frictionless so that there is no significant transfer of vertical stresses from the backfill material to the wall. Under such conditions, the vertical and horizontal stresses are equivalent to the major principal stress σ_1 and minor principal stress σ_3, respectively.

d. The shear stress generated from the frictional forces are fully mobilized along the effective length of the reinforcing elements.

e. The backfill material is uniformly inclined, and any surcharge load exerts a uniformly distributed pressure.

Stress Analysis Using Rankine Theory

Let Figure D-3 represent a smooth frictionless wall inclined an angle α with the vertical. Assume also that the surface of the backfill is inclined an angle i with the horizontal and carries a uniform surcharge of intensity q. The thrust acting on the wall F is the resultant of two forces: the gravity force W representing the weight of the wedge ACD and the surcharge, and a force P representing the total pressure acting on the vertical plane AC.

$$P = \int_0^H p \, dH \tag{D-6}$$

where the pressure p at any point B is a conjugate to the vertical pressure σ_v and acts parallel to the surface of the backfill

$$\sigma_v = (\gamma d + q) \tag{D-7}$$

where d is the vertical depth of the point under consideration, and γ is the unit weight of the soil. According to Rankine theory, p at point B can be expressed as

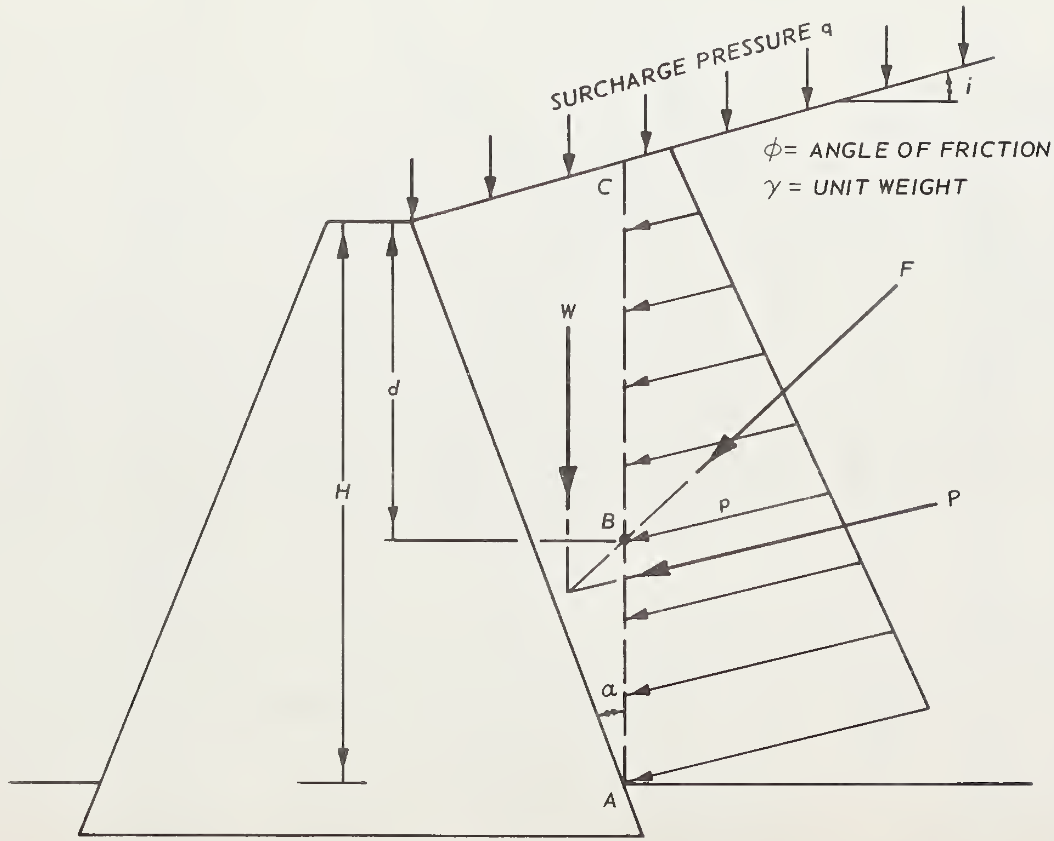


Figure D-3. Pressure distribution according to the Rankine theory.

$$p = (\gamma d + q)\ \frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}} \tag{D-8}$$

Where ϕ is the angle of internal friction and p acts parallel to the slope of the backfill soil. The horizontal component of p may be expressed as

$$\sigma_h = p \cos i = (\gamma d + q) \cos i\ \frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}} \tag{D-9}$$

Earth Pressure Theory - Reinforced Earth Wall

Consider a simple retaining wall with horizontal backfill and vertical frictionless wall as shown in Figure D-4. The horizontal stress at any point, after substituting for cos i by unity, is reduced to

$$\sigma_h = (\gamma d + q)\ \frac{1 - \sin \phi}{1 + \sin \phi} = k_a(\gamma d + q) \tag{D-10}$$

Assume the reinforcing ties are placed in a regular pattern, as shown in Figure D-2, with horizontal and vertical spacing S_x and S_z, respectively. The total horizontal thrust F_h acting on an area bounded by S_xS_z at d distance below the surface may be expressed as

$$F_h = \int_0^{S_x} \int_0^{S_z} \sigma_h \ dzdx = k_a(\gamma d + q)S_xS_z \tag{D-11}$$

In order to satisfy equilibrium conditions along the y axis, the horizontal thrust of the backfill material should be equal to the tension force in the ties. Consequently, the tension force carried by a single tie at depth d is

$$T_d = F_h = k_a(\gamma d + q)S_xS_z \tag{D-12}$$

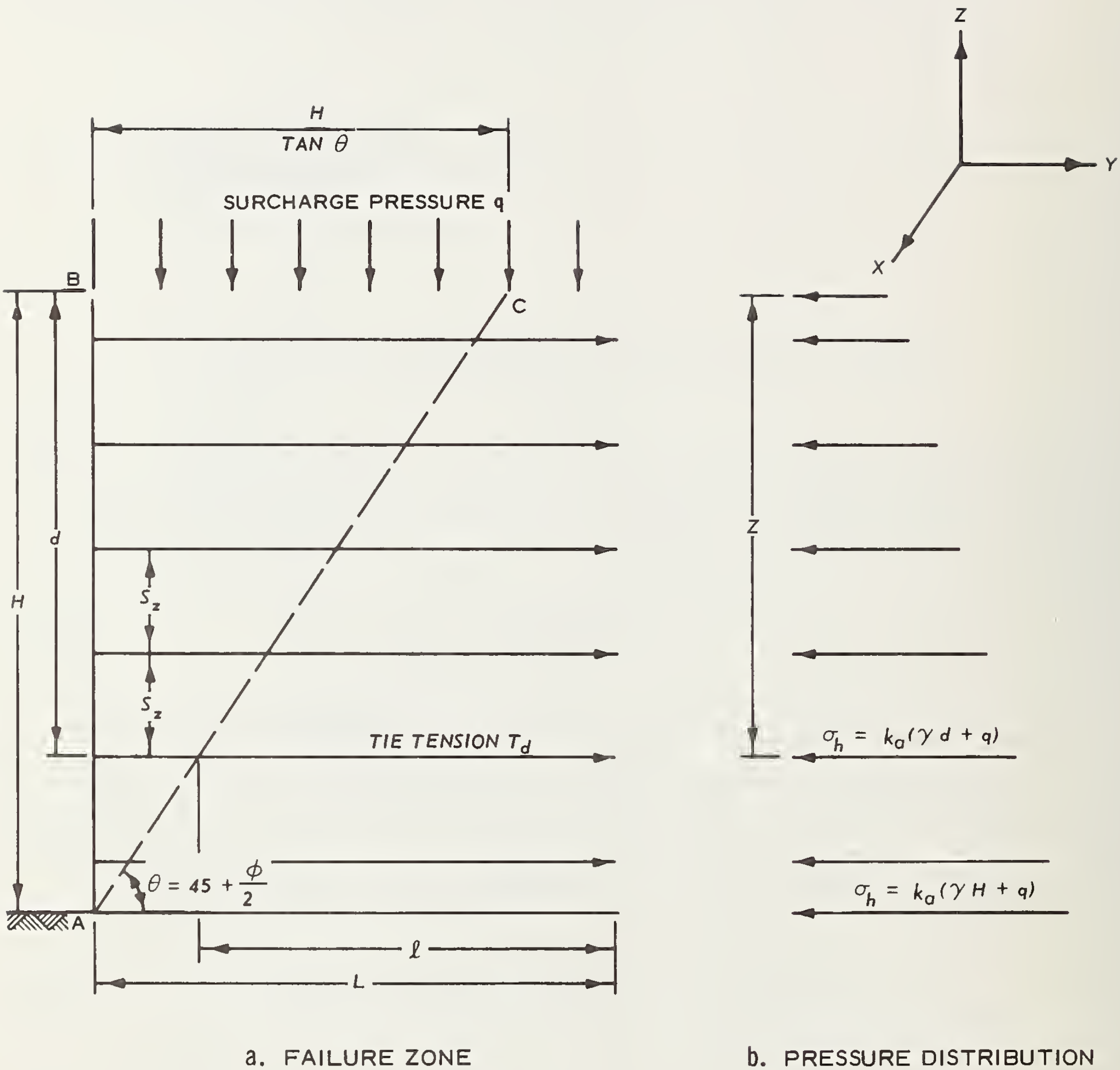


Figure D-4. Pressure distribution within reinforced earth according to the Rankine theory.

where T_d is the tensile force in the reinforcing tie.

It is clear from Equation D-12 that T_d takes a maximum value when the depth d assumes the fill height of the wall H, thus

$$T_{max} = k_a(\gamma H + q)S_x S_z \tag{D-13}$$

The knowledge of the tensile force and the tensile strength of the reinforcing ties enables the determination of the size of the reinforcing elements. Referring to Figure D-4 it can be seen that the total force carried by each tie must be resisted by frictional drag in the surrounding soil backfill. The maximum frictional force F_ϕ that may develop in a tie can be determined from the total normal force applied to the tie and the coefficient of friction between the tie and the backfill as

$$F_\phi = 2(\gamma d + q)w\ell \tan \delta \tag{D-14}$$

where w is the width of the tie, ℓ is the length of the tie within the stable zone, and δ is the friction angle between the soil and the tie.

For equilibrium conditions, F_ϕ must be equal to F_h defined in Equation D-11 if the factor of safety (FS) is equal to unity. However, if the frictional force on the tie is not fully mobilized then the factor of safety may be defined as

$$FS = \frac{F_\phi}{F_h} = \frac{2w\ell \tan \delta}{k_a S_x S_z} \tag{D-15}$$

Note that the factor of safety against slippage is independent of wall height, unit weight of soil, and the surcharge.

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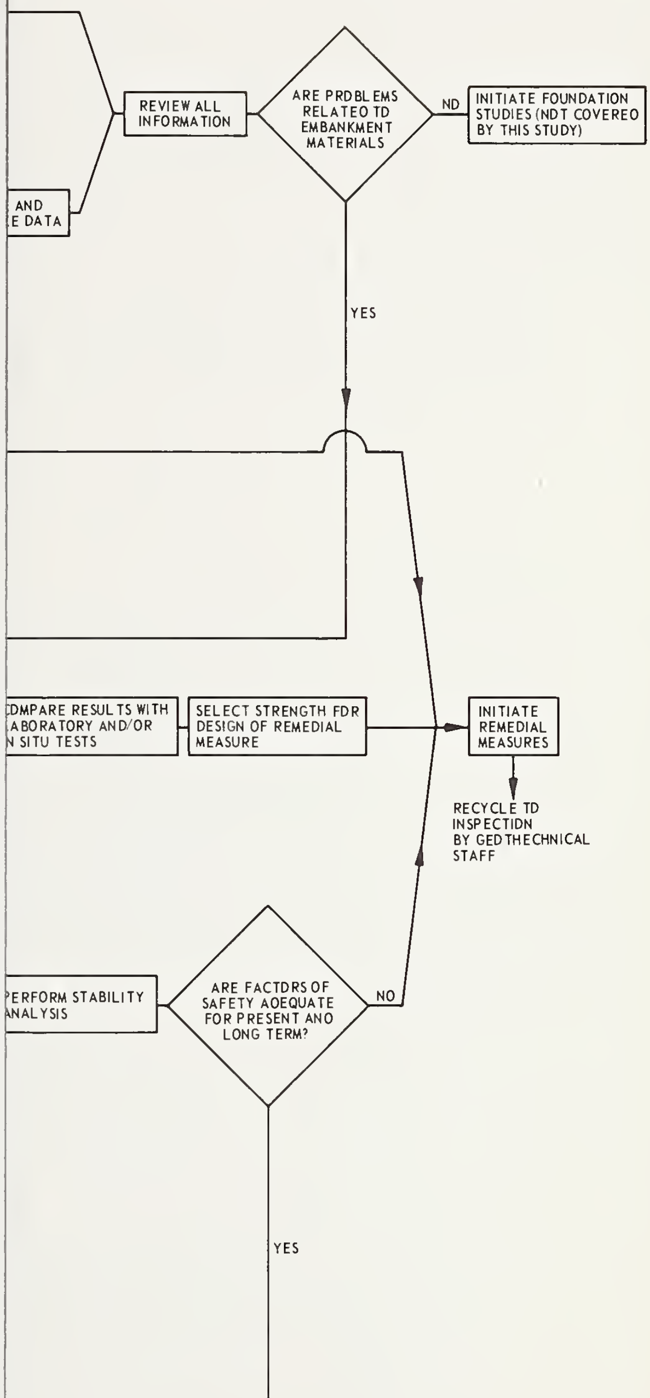

chart of recommended
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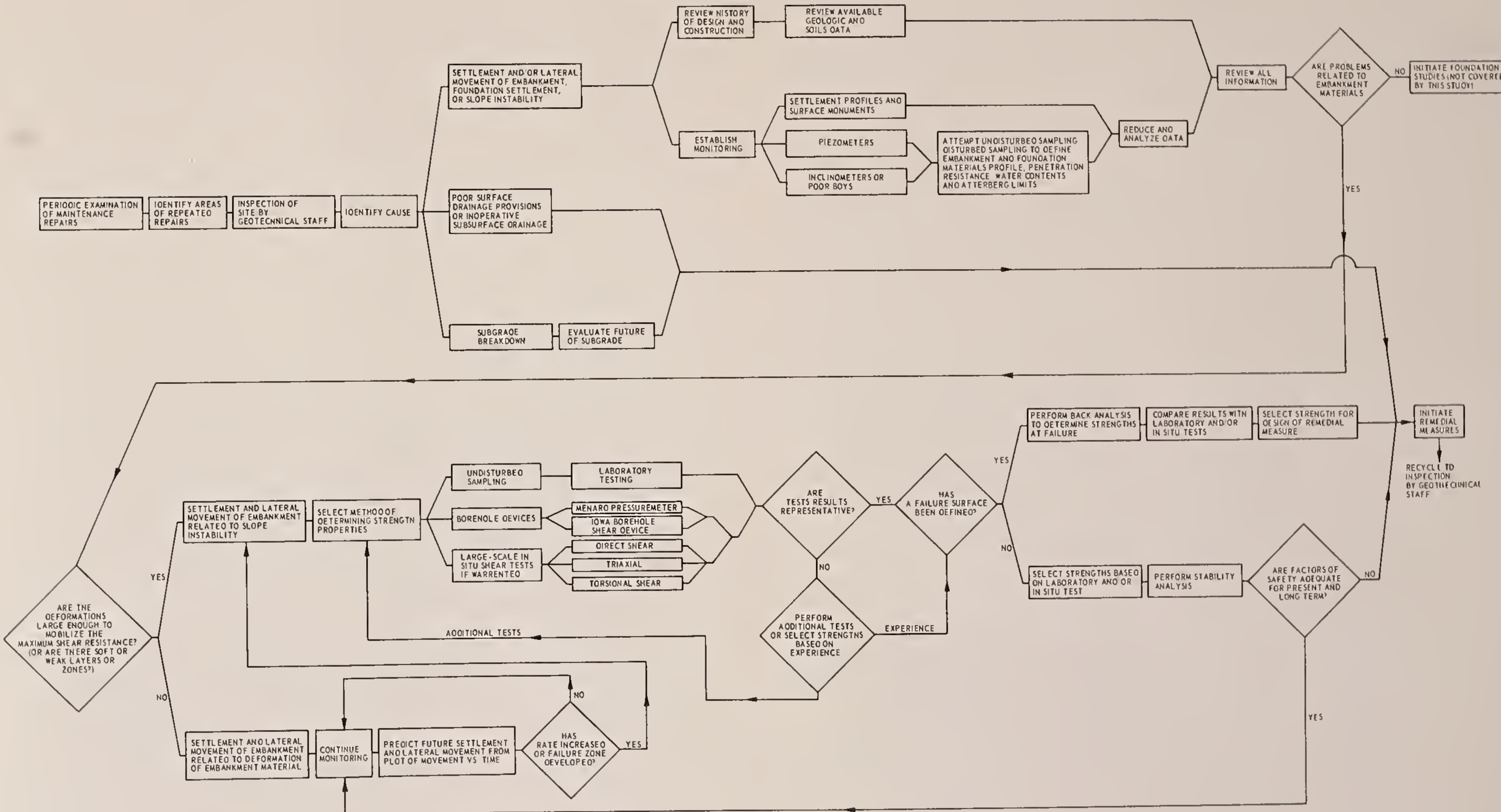

Figure 85. Flow chart of recommended methodology for evaluation of compacted shale embankments.

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