NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 8

CONSTRUCTION OF EMBANKMENTS

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

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CONSTRUCTION OF EMBANKMENTS

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
OF STATE HIGHWAY OFFICIALS IN COOPERATION
WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
HIGHWAY DESIGN
CONSTRUCTION
EXPLORATION-CLASSIFICATION (SOILS)
FOUNDATIONS (SOILS)
MECHANICS (EARTH MASS)

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1971

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectvity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NCHRP Synthesis 8

Project 20-5 FY'70
ISBN 0-309-02000-X
L. C. Card No. 71-173644

Price: $2.40

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the Federal Highway Administration. Individual fiscal agreements are executed annually by the Academy-Research Council, the Federal Highway Administration, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract. It has been accepted by the Highway Research Board and published in the interest of effective dissemination of findings and their application in the formulation of policies, procedures, and practices in the subject problem area.

The opinions and conclusions expressed or implied in these reports are those of the research agencies that performed the research. They are not necessarily those of the Highway Research Board, the National Academy of Sciences, the Federal Highway Administration, the American Association of State Highway Officials, nor of the individual states participating in the Program.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Highway Research Board
National Academy of Sciences
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PREFACE

There exists a vast storehouse of information relating to nearly every subject of concern to highway administrators and engineers. Much of it resulted from research and much from successful application of the engineering ideas of men faced with problems in their day-to-day work. Because there has been a lack of systematic means for bring such useful information together and making it available to the entire highway fraternity, the American Association of State Highway Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Highway Research Board to undertake a continuing project to search out and synthesize the useful knowledge from all possible sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series attempts to report on the various practices without in fact making specific recommendations as would be found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available concerning those measures found to be the most successful in resolving specific problems. The extent to which they are utilized in this fashion will quite logically be tempered by the breadth of the user's knowledge in the particular problem area.

FOREWORD

By Staff

Highway Research Board

This report should be of special interest to soils, foundations, design, materials, and construction engineers responsible for embankments. The report offers information on subsurface investigation practices, foundation design and construction procedures, design criteria, specifications, and construction practices and quality control procedures for embankments.

Administrators, engineers and researchers are faced continually with many highway problems on which much information already exists either in documented form in in terms of undocumented experience and practice. Unfortunately, this information is often fragmented, scattered, and unevaluated. As a consequence, full information on what has been learned about a problem is frequently not assembled in seeking a solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem. In an effort to resolve this situation, a continuing NCHRP project, carried out by the Highway Research Board as the research agency, has the objective of synthesizing and reporting on common highway problems—a synthesis being defined as a composition or combination of separate parts or elements so as to form a whole greater than the sum of the separate parts. Reports from this endeavor constitute a new NCHRP Report series that collects and assembles the various forms of information into single concise documents pertaining to specific highway problems or sets of closely related problems. This is the eighth report in the series.

Embankments have long been an important element in highway construction, particularly in rough terrain and in areas of low elevation subjected to periodic flooding. Modern geometric design criteria with demands for grade separation and gentle grades and curvatures have resulted in embankments assuming an even more important role. In addition, the development of massive and efficient earthmoving equipment has made it desirable to use embankments where previously bridges were more economical. Thus, at this point in time, embankments have assumed a major role in highway construction and it behooves those responsible for design and construction to be aware of the most recent information available on practices that can produce well-engineered embankments.

This report of the Highway Research Board provides information on materials, design, and construction considerations that must be coordinated and interplayed from initiation of the site investigation through the completion of embankment construction. More specifically, the report provides information on practices for subsurface investigation of embankment areas, treatment of foundations, design criteria, specifications, construction, and quality control for embankments.

To develop this synthesis in a comprehensive manner and to insure inclusion of significant knowledge, the Board analyzed available information (e.g., current practices, manuals, and research recommendations) assembled from many highway departments and agencies responsible for highway planning, design, construction, and maintenance. A topic advisory panel of experts in the subject area was established to guide the researchers in organizing and evaluating the collected data and for reviewing the final synthesis report.

As a follow up, the Board will attempt to evaluate the effectiveness of this synthesis after it has been in the hands of its users for a period of time. Meanwhile, the search for better methods is a continuing activity and should not be diminished. An updating of this document is ultimately intended so as to reflect improvements that may be discovered through research or practice.

CONTENTS

ACKNOWLEDGMENTS

This synthesis was completed by the Highway Research Board under the supervision of Paul E. Irick, Assistant Director for Special Projects. The Principal Investigators responsible for conduct of the synthesis were Thomas L. Copas and Herbert A. Pennock, Special Projects Engineers.

Special appreciation is expressed to Harvey E. Wahls, Professor of Civil Engineering, North Carolina State University, who, as special consultant to the Advisory Panel, was responsible for the collection of data and the preparation of the report.

Valuable assistance in the preparation of this synthesis was provided by the Topic Advisory Panel, consisting of Bud A. Brakey, Assistant Staff Materials Engineer, Colorado Department of Highways; Joseph D'Angelo, Supervising Civil Engineer, Massachusetts Department of Public Works; David S. Gedney, Chief, Soil and Rock Mechanics Branch, Construction and Maintenance Division, Office of Engineering and Operations, Federal Highway Administration; John R. Sallberg, Highway Research Engineer, Structures and Applied Mechanics Division, Office of Research and Development, Federal Highway Administration; and Charles H. Shepard, Soils Engineer, Ohio Department of Highways.

R. E. Bollen, Engineer of Materials and Construction; J. W. Guinnee, Engineer of Soils, Geology and Foundations; and L. F. Spaine, Engineer of Design, all of the Highway Research Board, assisted the Special Projects staff and the Advisory Panel.

Information on current practice and ongoing research was provided by many highway agencies. Their cooperation and assistance were most helpful.

CONSTRUCTION OF EMBANKMENTS

SUMMARY

Highway embankment failures may result from a variety of causes. The satisfactory performance of an embankment is dependent on its own stability and that of the underlying foundation. Currently, more embankment problems are the result of poor foundations than of faulty placement of the embankment itself. This points out the need for an adequate subsurface investigation of the foundation conditions at embankment sites.

The route location phase should include provisions for considering sites where subsurface conditions could cause embankment problems. Examples of these sites include areas of potential landslides, sidehill cuts and fills, abandoned mining operations, soft foundations, buried stream channels, and sanitary landfills. Soil surveys provide detailed information of specific conditions along the selected route. In many agencies, there is a need to increase the depth of exploration in embankment areas to sample the materials affected significantly by embankment loads.

Embankment construction begins with the preparation of the foundation. Any unusual condition likely to cause problems during construction or during the life of the embankment should have been identified and corrective measures prescribed prior to start of construction. Consolidation of soft foundation material is a widespread problem. The difficulties associated with sidehill fills and cut-to-fill transitions are also classified as critical by many agencies. Benching and adequate drainage provisions are the most widely accepted solutions for correcting both sidehill fills and cut-to-fill transitions.

In the design of embankments, geometric design criteria and safety standards usually have precedence over other considerations such as soft foundation and poor materials. The design of high embankments should consider the quality of the fill materials because the weight of the embankment is critical.

Standard specifications of highway agencies continue to require fill material to be placed in relatively thin lifts and compacted by rolling with suitable equipment. There is, however, a trend toward minimizing procedural specifications and placing greater reliance on density requirements. Moisture content is a continuing problem, particularly with silty soils and swelling clays. Current procedures for building rock embankments are generally satisfactory. A move to thicker lifts may be justified with vibratory compactors.

Sand-cone and balloon methods are the two most prevalent test methods used to determine in-place density. Both are time consuming. The use of nuclear equipment to perform nondestructive density and moisture measurements is increasing. In-place densities are most commonly evaluated in relation to the AASHO T-99 maximum density. The most common procedures for the field evaluation of maximum dry density involve the use of a one-point compaction test with a family of moisture-density curves.

Density requirements, except specifications based on statistical quality control, are considered to be minimum standards that must be exceeded by all field test results. Statistical concepts for density requirements help to evaluate the significance of an occasional bad test.

Problems with expansive clays are more common in cut sections than in

embankments. It is usually possible to use these soils in the fill so that they do not represent a major problem to the completed construction. However, special methods and controls may be required.

Frozen soils cannot be compacted satisfactorily. Unless the frozen layer of material can be removed from both the embankment and the cut or borrow area, operations should be suspended.

Although construction practices directed toward protecting the environment have always been encouraged, improved practices are being introduced that will have a significant impact on highway construction. Waste disposal and erosion control are areas that are receiving much attention.

For successful earthwork construction, it is important to integrate design, construction, and soils considerations, beginning with the initial route location studies and continuing through the completion of construction.

CHAPTER ONE

INTRODUCTION

Geometric design and other route location considerations often require modern highways to be constructed at elevations above existing ground. Although such conditions have long been faced at water crossings and in mountainous or hilly terrain, the limited access and more stringent geometric design requirements of the Interstate Highway System have introduced the problem of supporting pavement systems above ground in areas of flat topography. When a pavement is to be supported above existing ground, a bridge structure or an earth embankment is required. At water crossings, narrow ravines, and some sites of extremely poor foundation conditions, bridging usually will be the most satisfactory solution. In some other cases, strong consideration should be given to adjustment of the center line or the grade so as to place the pavement system at or below natural ground. However, in the vast majority of situations, an earth embankment will provide the most practical and economical road support system. As a consequence, embankment construction is a major component of modern highway construction.

EMBANKMENT FAILURES

The function of a highway embankment is to provide support for a pavement system above natural ground. An embankment has failed when it causes roughness or damage to the roadway. The failure may be spectacular and catastrophic, as in the case of slides resulting from instability of the embankment or the underlying foundation materials. A total loss of the pavement section and portions of the embankment itself may result (Fig. 1). However, the failure is usually more subtle. Creep and/or consolidation of the embankment or the underlying foundation materials may produce failure by the gradual development of excessive differential settlements of the pavement surface, causing rutting, dips, or cracks (Fig. 2). Thus, embankment performance is associated with the stability and the deformation of both the embankment and the underlying foundation materials.

There are many reasons for unstable embankments. Failures within the foundation may be the result of inadequate site investigations, insufficient consideration of foundation conditions in design, or improper implementation of the design solutions during construction. Failures originating within the embankment itself may be caused by poor materials, unsatisfactory construction methods, or ineffective quality control procedures. Currently, more embankment problems are the result of poor foundation conditions than of faulty placement of the embankment itself. These foundation problems are caused primarily by inadequate consideration of soft foundation soils, sidehill locations, cut-fill transitions, and groundwater conditions. Relatively few problems result from poor placement of fills, because generally good construction practices and quality control procedures for placement of fills have been developed throughout the past 30 years.

EMBANKMENT LOAD

Highway embankments are major structures that produce significant loads on the underlying foundation soils. The weight of each foot of fill is roughly equivalent to the weight of one story of a conventional office or apartment building. In other words, a 20-ft-high earth embankment

Figure 1. Massive slide failure.

will weigh as much as a 20-story office building occupying an equivalent contact area. Furthermore, the rate at which the contact pressure is dissipated with depth beneath the natural ground surface depends on the least dimension of the loaded area. Because of the great width of many embankments, the stress beneath the center of the embankment usually decreases very slowly with depth. The combination of these factors means that large stress increases can be produced at significant depths beneath an earth embankment.

The preceding principles are illustrated by the example shown in Figure 3, in which the stresses produced by a 20-ft-high earth embankment are compared to those produced by a bridge pier foundation. Although the contact pressures are much higher for the bridge foundation, the stresses decrease very rapidly with depth. At depths of more than 10 ft, the stresses caused by the embankment loading exceed those of the bridge foundation. At a depth of 80 ft, the stresses caused by the embankment loading are still more than 80 percent of the surface contact pres-

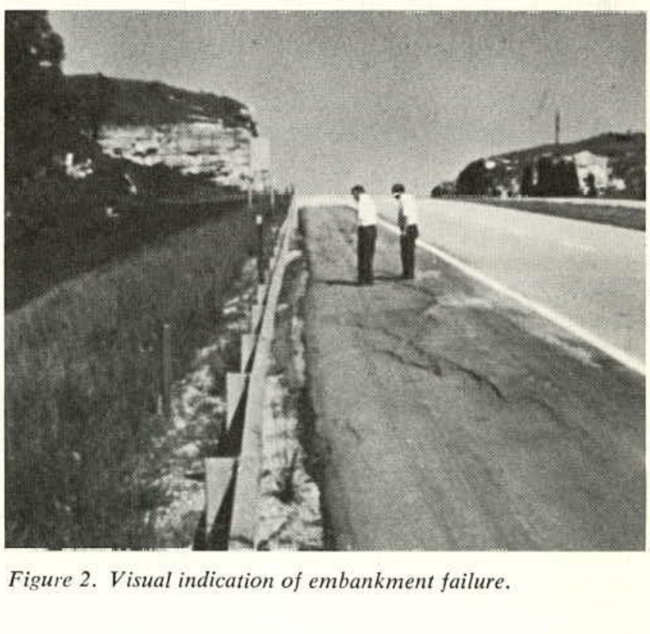


Figure 2. Visual indication of embankment failure.

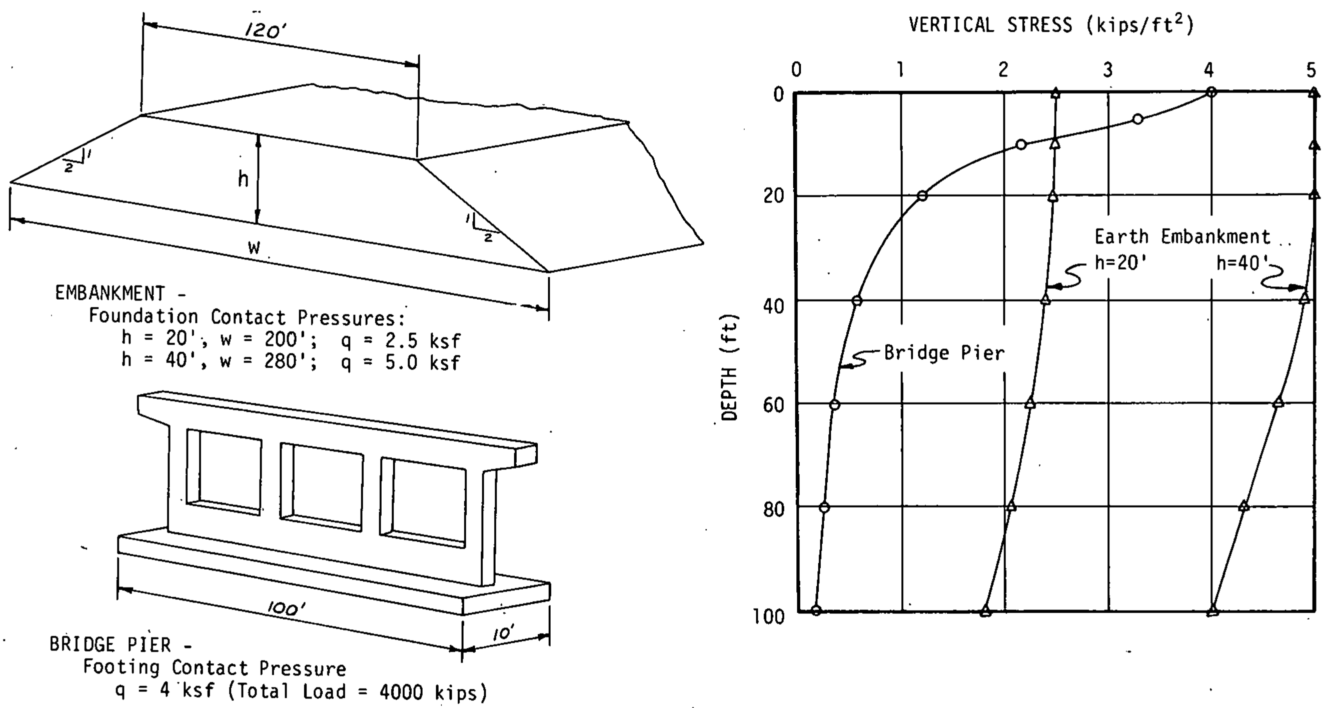


Figure 3. Comparison of vertical stresses beneath center lines of bridge pier and earth embankments.

sure and are more than eight times greater than those under the bridge foundation. The differences are even more dramatic for the stresses beneath a 40-ft-high embankment, also plotted in Figure 3. Hence, an adequate subsurface investigation of the foundation conditions beneath embankment sites becomes an essential prerequisite for satisfactory embankment design and construction.

SCOPE OF SYNTHESIS

For earthwork construction to be successful, materials, design and construction considerations must be intermeshed from the initiation of the site investigation through the completion of construction. Consequently, construction practices cannot properly be separated from design and materials considerations. For this reason, the synthesis includes:

1. Subsurface investigational practices for embankment areas.

2. Design and construction procedures for the treatment of embankment foundations.

3. Design criteria, specifications, and construction practices for embankments.

4. Quality control procedures for placement of embankment materials.

Both good practices for normal conditions and special practices for handling unusual conditions are presented.

Cut zones are not considered except as they affect embankment construction (e.g., as sources of embankment materials or special procedures at cut-fill transitions). Special treatments or procedures applied to the upper surface of an embankment are regarded as part of the pavement construction and have been excluded. Furthermore, the special problems associated with the design and construction of bridge approaches have been presented in *NCHRP Synthesis 2,* so are not discussed in detail herein.

CHAPTER TWO

SUBSURFACE INVESTIGATIONS

Adequate knowledge of subsurface conditions often is the key to successful earthwork construction. Most agencies have long recognized the need for comprehensive subsurface investigations at bridge sites. Currently, there is a growing awareness of the potential effects of the extremely heavy loads imposed on the subsoils by embankments and, as a consequence, the investigational programs for embankment foundations are being upgraded.

Subsurface information can be used at several stages in the development of a highway project. First, subsurface conditions can be an important and sometimes decisive factor in route location. Second, after specific lines and grades are established, detailed subsurface investigations are required for the design of embankment sections.

ROUTE LOCATION

Subsurface conditions can significantly affect the cost of highway construction and hence should be one of the considerations in highway location studies. Although other factors often will outweigh the influence of the subsurface conditions, many examples can be cited of locations where the subsurface conditions were or should have been the decisive factor. These examples include locations in areas of potential landslides, sidehill cuts and fills, abandoned mining operations, limestone cavities, and various soft foundation conditions, such as peat bogs, marshes, buried stream channels, and sanitary landfills.

The expertise of the geologists and soil engineers who are responsible for the detailed soil surveys, conducted after the location has been established, should be used in the preliminary investigations. When the preliminary investigation is conducted by special reconnaissance personnel, trained to consider a variety of route location factors, it is essential that these personnel also have sufficient training and experience to recognize potential geotechnical problem areas for which the advice of the specialists should be sought.

Several agencies have successfully instituted a policy of preparing preliminary geotechnical reports for major route location studies. These reports are prepared by geotechnical specialists in advance of public hearings so that subsurface conditions can be considered fully together with other route location factors. Preliminary geotechnical reports generally are prepared from existing information and from limited field reconnaissance. Sources of information include published and unpublished geological surveys and maps, groundwater and hydrologic surveys, topographic maps, soil conservation reports, agricultural soil maps, air photos, and existing detailed soil surveys from previous highway projects in the vicinity of the proposed route. Field reconnaissance commonly is limited to recollections of past personal experience or to walking the line of each proposed route. Occasionally, geophysical exploration techniques, such as seismic and resistivity surveys, are used; and in a very few instances, when extremely critical conditions are identified, probe borings and/or samples may be justified. The preliminary geotechnical report includes general descriptions of the topography, the geology, and the soil conditions along each proposed alignment. Typical profiles or geologic cross sections are desirable. This information is used to identify potential problem areas, including consideration of the potential for landslides, stability of cut and fill slopes, extent of rock excavation, groundwater conditions, potential foundation conditions for bridge structures and embankments, and availability and quality of construction materials. The potential need for special foundation treatments may be noted.

The potential of air photos for the preparation of preliminary geotechnical reports deserves special consideration. Almost all highway agencies currently are obtaining air photos of proposed route locations for photogrammetric purposes; i.e., preparation of topographic maps, right-of-way acquisition, and earthwork calculations. These same air photos also can provide qualitative information on the soil and rock conditions.

SOIL SURVEYS

After the line and the grade for a proposed route have been established, a detailed soil survey is made. It includes a field exploration program, laboratory testing, and, usually, some soil mechanics analyses. The report of the soil survey generally includes detailed descriptions of the soil and rock materials encountered along the right-of-way, a plot of the soil profile, and a summary of all laboratory test results. Problem areas are identified and design solutions proposed. Also, the quality and quantity of materials sources for embankments, subgrades, and pavement components are reported. Several agencies (e.g., Kentucky) have prepared extensive specifications for the preparation of a soil survey. These manuals are excellent sources of detailed information on the requirements of a satisfactory soil survey.

The findings and recommendations of the soil survey must be adequately reported to the engineers responsible for both the design and the construction of the embankment. It is highly desirable for the boring logs and soil profiles to be plotted on the construction plans. Note

Field Exploration Procedures

Typically, both geophysical and soil boring techniques are employed for the soil survey. The most common geophysical methods are seismic refraction and electrical resistivity. These techniques are most useful in determining the position of the groundwater table, the depth to rock, and the delineation of various rock strata. They are less effective in differentiating among soil types and do not provide for visual identification of the subsurface materials. Thus, when these procedures are used, they are almost always supplemented by some sort of boring program.

Current good practice for embankment foundation investigations includes the use of disturbed borings and samples supplemented with undisturbed sampling in critical areas. The use of a hand or power auger to obtain disturbed samples is by far the most common soil exploration technique. In addition, split-barrel samplers (standard penetration tests), which provide undisturbed samples, or static cone penetrometers sometimes also are used. When soft deposits of cohesive soils are encountered in embankment areas, undisturbed samples commonly are obtained with hydraulically operated thin-walled samplers. Most borings for embankment foundations are terminated when rock is encountered. In some instances, rotary or diamond core borings are used to investigate the soundness of bedrock or to obtain samples of soft weathered rock. For economic reasons, undisturbed sampling techniques generally are limited to critical problem areas that previously have been identified from disturbed borings.

The spacing between borings will depend on the specific geologic conditions along the right-of-way and the need for exact data by the designer for specific locations on the highway. Because the critical areas commonly represent only a small percentage of the total investigation, heavy reliance must be placed on the judgment and experience of field personnel to obtain adequate borings in all potential problem areas. At least one boring should be made beneath each fill. For long embankments the maximum interval between borings can range from 250 to 1,000 ft, with spacings of 300 to 500 ft most common. In problem areas the spacing between borings will be significantly reduced and may approach 50 ft when defining the extent of soft deposits. Typically, borings are located along the center line of the pavement, except when defining the limits of soft deposits or investigating sidehill locations.

Many agencies continue to specify inadequate minimum depths for borings in embankment areas. The required depth of borings should be related to the width and height of the proposed embankment. On the basis of the depth to which significant stresses are produced by embankment loads, borings ought to extend to depths equal to the half-width (i.e., the distance from the center line to the toe of the slope) or to twice the height of the embankment. The width criterion is fundamentally more correct, but the height criterion is easier to use in practice. These criteria may be tempered by experience with local geologic conditions. Also, borings may be terminated at shallower depths when firm bedrock is encountered. Conversely, borings never should be terminated in a soft deposit, but should continue until firm material is located.

Laboratory Investigations

Routine laboratory investigations include mechanical analyses and Atterberg Limits tests for classification of all soils encountered along the right-of-way. Depending on the soil classification system used within a specific agency, additional tests, such as volume change, swell potential, sand equivalent, resilience, or compaction tests, may be required of all samples. In addition, field densities and moisture contents are determined. Procedures for conducting all tests are detailed in each agency's test manual or by reference to AASHO or ASTM standards.

When the strength or the compressibility of the foundation soil is to be determined, undisturbed thin-wall samples are essential. Relatively conventional consolidation tests are required when soft compressible foundation materials are identified and settlement analyses are to be made. If the stability of a soft foundation or a sidehill is of concern, the shear strengths of the subsoils are determined from laboratory unconfined compression, direct shear, or triaxial compression tests. The type of test may be related to the method of stability analysis and the factors of safety that will be employed. Carefully performed simple tests can be more meaningful than poorly conducted sophisticated tests.

EMBANKMENT SOILS

The field investigation of soils to be used in embankments generally is limited to auger borings. The laboratory testing of these soils includes the gradation, plasticity, and miscellaneous tests required for classification. Some type of compaction test is performed to determine a standard maximum density and the optimum moisture content at which it is obtained. Frequently, a stiffness test, such as a CBR or stabilometer test, is conducted on the compacted material for use as a subgrade property in pavement design. Shrink-swell factors may also be estimated for use in earthwork computations.

Currently, there is only an occasional need for strength and consolidation tests of embankment soils. Laboratory and field data indicate that the compression of an embankment that has been compacted to current density requirements is usually negligible. Similarly, for the current standard design slopes, the strength of an embankment generally is not critical. Exceptions may occur in embankments exceeding 50 ft in height or when extremely poor quality soils must be used. In these instances, strength and consolidation tests should be conducted on the compacted embankment soils. Although there is minimal need for these tests today, it is likely that their use will become more common in the future as higher embankments are constructed and lower quality soils must be used more frequently.

CHAPTER THREE

FOUNDATION PREPARATION

The construction of an embankment begins with the preparation of the foundation. When good foundation conditions are encountered, only simple straightforward procedures are required. However, when poor foundation conditions are involved, complex and expensive treatments may become necessary to prevent poor performance or embankment failure. Soft foundation soils, sidehill locations, cut-fill transitions, and groundwater problems are examples of conditions that generally require special treatment. The occurrence of these four conditions is sufficiently widespread that their treatments are discussed in detail.

Because special foundation treatments can be very expensive, it is extremely important that the need for special treatments be identified in the soil survey and adequately considered in the preparation of design drawings and specifications. When unanticipated problem areas are encountered during construction, corrective measures not only delay construction but also inevitably cost more than if anticipated in the design. Thus, planning for treatment of poor foundation conditions becomes an important aspect of embankment design. Recognition and analysis of poor foundation conditions and recommendations for corrective treatments generally are the responsibility of the geologists, and soils or materials engineers who prepare the soil survey.

MINIMAL PREPARATION

Clearing and grubbing are the first steps in the preparation of an embankment site. The topsoil usually is removed. In many instances, all stumps also are removed from the right-of-way. However, because of increasing costs of removal and problems of disposal, it is becoming increasingly common to allow stumps to remain in place when they are a sufficient distance below the subgrade. Typically, trees may be cut within 3 or 4 in. of natural ground and the stumps left in place when the embankment height is greater than 5 or 6 ft.

For very low embankments, the foundation material may be undercut to improve the uniformity within the subgrade zone and to eliminate some cut-fill transitions. Generally, the depth of undercutting is specified so as to produce a minimum embankment height of approximately 3 ft. At least one agency is undercutting most foundation materials to the ditch line and constructing low embankments, even in cut zones. More commonly, the natural ground below shallow fills is proof rolled or compacted in place. If soft spots are encountered, undercutting may be required. In some instances the excavated material can be dried and then recompacted; in others the excavated material must be wasted and replaced by acceptable material.

If satisfactory foundation conditions are encountered, the preceding procedures are all that are normally required and the construction of the embankment itself can be started. If unanticipated foundation problems are exposed during these preliminary operations, geologists or soil specialists should be consulted for recommendations of additional treatments. If the poor conditions were anticipated in the design, additional corrective treatments already will have been prescribed in the plans or specifications.

SOFT FOUNDATIONS

Soft foundation soils include a variety of peats, marls, and organic and inorganic silts and clays. These deposits may be localized, as in a peat bog or a river crossing, or they may encompass vast areas, such as tidal marshes or glacial lake beds. The compressible material may occur at the surface (e.g., marshes and some peat bogs), or it may be buried beneath a mantle of satisfactory soil. Examples of the latter case include abandoned stream channels, lake beds, and peat bogs, which may be covered by a dessicated crust or a more recent soil deposit.

When soft soils are discovered, both the strength and the compressibility testing, and analysis, will depend on the magnitude of the project and also the types of treatment being considered. For example, if it is planned to excavate the soft material, extensive knowledge of its properties usually is not required. On the other hand, if a sand drain installation is contemplated, a very extensive investigation is necessary.

Soft foundations create several types of embankment problems. The foundation soil may displace laterally under the weight of the fill, causing major movements and disruptions of the embankment, as shown in Figures 4a and 4b. If the lateral displacement is prevented, the foundation soil still will compress and cause settlement of the embankment and the pavement, as shown in Figure 4c. Some settlement is to be anticipated in almost all embankment foundations; it is detrimental only if it produces fracture or excessive roughness of the pavement.

The total or differential settlement that can be tolerated by a pavement rarely is specified except in the case of bridge approaches, for which the tolerable settlement commonly is specified as ½ to 1 in. For roadway embankments, the allowable settlement after paving depends on the length of the fill and the rate at which settlements develop. Experience has indicated that 6 to 12 in. of settlement can be tolerated in long embankments, if any variations in the settlement are uniformly distributed along the length of the embankment. At least one agency permits a differential settlement of 2 in. per 100 ft. In some instances, predicted settlements of several feet can be accepted if the

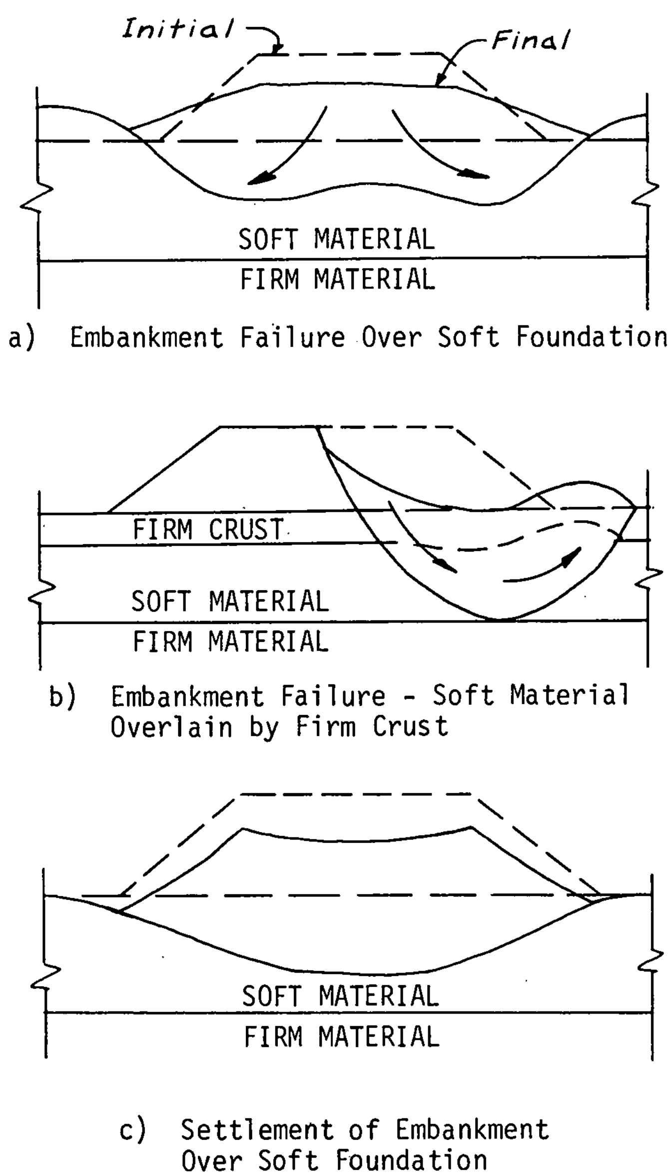


Figure 4. Typical embankment problems over soft foundations.

settlement develops very slowly over long periods of time (e.g., 25 to 50 years). In general, however, the amount of settlement that can be tolerated by a pavement system is not well defined and requires further study.

When long-term settlement is anticipated, flexible pavements often are used to facilitate future maintenance. Also, interim or stage construction of the pavement sometimes is employed. These procedures may be more economical than attempting to improve the soft foundation conditions prior to construction of the pavements. In some critical situations, these procedures may be necessary in addition to some initial partial treatment of poor foundation conditions.

Several treatments commonly are used to improve soft foundations. They include (a) removal by excavation or displacement and (b) consolidation by preloading with a waiting period or surcharging with or without sand drains and berms. Another procedure is the use of lightweight embankment materials, perhaps in combination with one of the preceding treatments.

Removal

Removal by excavation is perhaps the most common method for treating soft foundation conditions. Excavation usually is the most economical and satisfactory solution when the unsuitable material does not extend to depths of more than 15 or 20 ft. Beyond this depth, alternate solutions may be more effective than excavation. Of course, the depth beyond which excavation ceases to be the most economical treatment for a specific site will depend on the soil type and groundwater conditions at the site. Examples can be cited in which as much as 40 ft of soft material have been successfully excavated.

When excavation is employed, the depth of excavation should extend to firm soil. However, the required lateral extent of excavation is less clearly defined. Four typical methods for defining the zone of excavation are shown in Figure 5. In examples (a) and (b) the width of the zone of excavation depends only on the height and slope of the embankment, whereas in examples (c) and (d) it also depends on thickness of the unsuitable material. The use of the section in Figure 5 is restricted to deposits that are less than 5 ft thick. Wide zones of excavation reduce the danger of lateral creep of the embankment.

The chief advantage of complete removal by excavation is the certainty that the potential foundation problems will be eliminated. The method is most effective for relatively shallow surface deposits of limited extent (for example, peat bogs and marshes). It becomes costly and impractical for extensive deposits of compressible clays or deposits that are buried beneath a mantle of suitable material. Also, every cubic yard of excavated material must be replaced by an equivalent quantity of good fill. Thus, the availability of sufficient quantities of suitable fill material becomes one criterion for the effective use of the excavation method. Furthermore, if unsuitable material is excavated below the groundwater table, special placement procedures may be required to dewater the excavation or to place the fill under water.

Removal by displacement is an old technique that has been used effectively on peat and marsh deposits. Today, however, displacement methods are used infrequently and with much caution because of the uncertainty of complete removal. Displacement is accomplished by controlling the position of fill placement so as to displace the unsuitable material laterally and ahead of the embankment construction. Usually the displacement force is produced by the weight of fill, but occasionally it is supplemented by blasting, jetting, or surcharge. The filling operation must be controlled very carefully to prevent entrapment of pockets of unsuitable material. Because of these uncertainties, detailed specifications and careful field inspection are necessary when displacement procedures are employed. Core samples of the completed embankment are desirable to verify that no pockets of unsuitable material remain.

Consolidation

Other treatments involve the consolidation of the soft foundation materials prior to finished grading and construction of the pavement section. Because consolidation is a time-dependent process, these techniques generally require a delay or waiting period between completion of the embankment and construction of the pavement section.

When the consolidation of soft materials is considered, it is necessary to estimate both the magnitude and the time required for development of the ultimate consolidation of the soft foundation under the loads imposed by the embankment and pavement system. Also, the stability should be investigated to ensure that the stresses produced by the embankment loads do not exceed the strength of the subsoil and produce massive slides and uncontrolled lateral displacement of the foundation soils. These analyses, together with the related consolidation and strength tests of the foundation soils, must be included in the soil survey whenever consolidation procedures are recommended.

If the predicted settlement is more than can be tolerated by the roadway, first consideration is given to constructing the embankment to subgrade elevation and waiting for completion of the consolidation due to the embankment loading prior to final grading and construction of the pavement section. In this case, the predicted magnitude of settlement is used only to estimate the additional quantities of fill required and thus is of secondary importance. However, when settlements of several feet are produced, the additional fill required to compensate for the settlement can become a significant percentage of the total earthwork. The primary consideration when delaying pavement construction is the time required for completion of the consolidation process. One year usually is the maximum acceptable delay period. Occasionally, when soft foundation problems are recognized far enough in advance, scheduling can be arranged to allow as long as two years for consolidation. More commonly, however, when the predicted delay periods are longer than a year alternate methods are considered to accelerate the rate of settlement. Because the primary consolidation time is related to the square of the drainage distance within the compressible soil, the necessity of additional treatments becomes increasingly probable with increasing thickness of the soft layer.

The most common procedure for accelerating the rate of settlement is application of a surcharge load, which is produced by constructing the embankment to a height in excess of the design height. Settlements will occur more rapidly under the combination of the design and the surcharge loads than under the design load only; hence, the waiting period will be decreased. Sufficient fill must be available to develop the surcharge. Also, the addition of a surcharge load increases the prospects for stability problems and may require additional adjustments in the embankment design (for example, flattening of slopes or construction of berms) to compensate for this.

The rate of settlement also can be accelerated by improving the drainage of the compressible layer with sand blankets or vertical sand drains. Placement of a thick granular layer over the compressible stratum prior to constructing the embankment insures free drainage of the upper surface of the compressible layer. It does not, however, appreciably shorten the drainage path in the compressible layer; hence, its effectiveness is limited to relatively thin surface deposits of compressible material. On the other hand, installation of vertical sand columns through the compressible stratum can appreciably reduce the drainage path by developing radial flow to the pervious

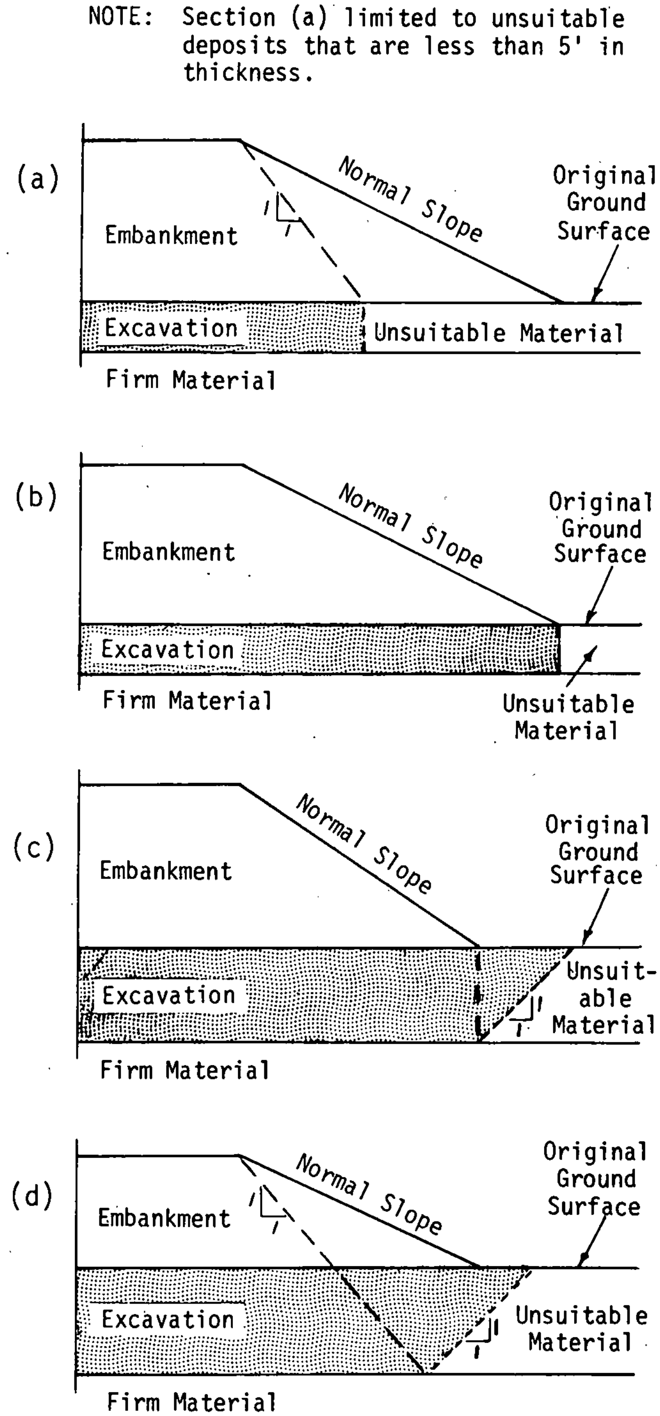


Figure 5. Examples of excavation of unsuitable material.

sand columns. The radial drainage distance, hence the time required for consolidation, are controlled by the spacing between drain wells. Thus, the primary consolidation time can be reduced from many years to several months by the installation of a properly designed vertical sand drain system. Surcharges commonly are employed in conjunction with sand drains and the benefits of both techniques are combined. Surcharging also will decrease the amount of post-construction secondary compression.

Theories and design procedures for sand drain systems now are readily available in the soil mechanics literature. Moreover, because the initial installation by an agency often has been treated as a research project, an increasing number of well-documented case studies of instrumented sand drain installations are becoming available. Although many good case studies remain unpublished, excellent reviews of the design and installation of sand drains are found in Moore and Grosert (1968), Johnson (1970), and Moran, Proctor, Mueser and Rutledge (1958).

As effective as the sand drain method may appear, it is used infrequently in highway construction. The foremost reason is that sand drain installations are relatively expensive and should be considered only when other methods are not adequate. Also, sand drains may not be suitable for certain very soft deposits. For example, in very soft marsh deposits, lateral movements under the embankment may tend to disrupt the vertical sand column and eliminate its effectiveness.

Considerable controversy remains in regard to various methods for installing sand drains. The driven closed-end mandrel procedure is most common because it is rapid, efficient, and the least expensive installation method. However, it is also the most criticized method because large volumes of soil are displaced and the resulting soil disturbance may reduce stability and increase compressibility.

Nondisplacement installation methods, such as jetted mandrel or hollow-stem auger methods, are believed to reduce soil disturbance effects but are more expensive than driven mandrel installations. Some agencies now ban the use of the driven closed mandrel and require more expensive nondisplacement installations; others believe that the advantages of nondisplacement methods have not been sufficiently documented to justify their higher cost. Few field studies of the relative effects of various installation methods are available. Two excellent unpublished field studies in New Hampshire and Maine have led to the recommendation of jetted nondisplacement methods for sand drain installations in sensitive soils in these states. The installation method was less significant in relatively insensitive soils.

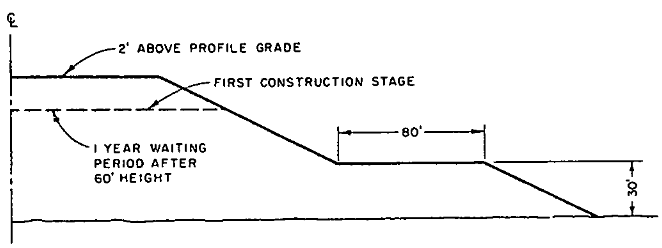


Figure 6. Typical embankment with berms and stage construction.

In all preloading methods, with or without surcharges and/or sand drains, the stability of the embankment and foundation system must be considered. The danger of lateral instability generally is increased by the use of a surcharge and sometimes by the installation of sand drains. A factor of safety against sliding of at least 1.5 usually is required. If lower factors of safety are calculated, the embankment section may be redesigned by either flattening the slopes or adding berms. The choice between these two methods is based on consideration of right-of-way limitations and ease of construction and maintenance When berms are used, it is important that they be sufficiently wide to be effective. This must be evaluated by additional stability analyses of the revised embankment section. Frequently, the berm must extend laterally to intersect the original critical stability circle. A typical embankment section with berm is shown in Figure 6.

In a relatively few instances, stage construction of the embankment is used to overcome stability problems. Because the foundation soils become stronger as they consolidate with time, the factor of safety against sliding is most critical initially. The rate of construction is controlled so as to take advantage of the increases in strength with consolidation. Construction rates may be controlled by specifying an incremental construction height to be followed by a waiting period with no construction prior to starting the next incremental height, or by specifying a continual maximum rate of construction (e.g., 5 ft per week). Stage construction should be avoided whenever possible, because of the close construction supervision that is required for it to be successful.

In all instances where soft foundation materials are being consolidated, field instrumentation and close construction supervision are imperative. Because of the many uncertainties involved in predicting field settlement rates from laboratory tests, field measurements commonly are used as guides to control the rate of construction or the duration of waiting periods. The extent of the field instrumentation will depend on the complexity of the project. On simple projects, only measurement of the settlement of the embankment surface may be necessary. However, often it is also desirable to measure the settlement at points within or at the base of the embankment. As the projects become more complex and stability becomes a major consideration, the pore pressures within the soft material and the lateral movements of the embankment and the foundation also may be measured. These measurements are needed particularly in projects involving sand drains, berms, or stage construction. A discussion of the types of instrumentation is presented in Chapter Seven.

Stability and settlement problems also can be reduced through the use of lightweight embankment materials. However, applications of this technique are extremely rare, primarily because of the limited availability or the prohibitive cost of lightweight aggregates. Also, experiments with various methods for deep lime stabilization of soft soils have been ineffective or prohibitively expensive.

SIDEHILL FILLS

Sidehill fills are cited by many agencies as their most critical embankment problem. Instability and sliding generally are associated with two geologic factors common to sidehill locations. First, in many locations bedding planes or boundaries between weathered and unweathered material tend to slope downhill. The addition of a sidehill fill often produces a tendency for sliding along these natural planes of weakness. Second, the construction of a sidehill fill disrupts the natural movements of surface water and groundwater. The accumulation of water within the embankment zone increases the tendency for sliding by increasing the weight of the sliding mass and at the same time decreasing the soil's resistance to sliding. Even when water does not appear on the surface, the accumulation of groundwater along the sloping bedding planes within the hill can produce a slippery plane along which sliding is likely to develop. Thus, groundwater control becomes a major consideration in the construction of satisfactory sidehill fills. However, before the groundwater can be controlled, its presence must be identified. This is often difficult during field surveys because the groundwater conditions are likely to fluctuate throughout the year, and the most critical conditions may not exist when the field survey is made.

Because of the difficulties in identifying and correcting sidehill problems, strong consideration should be given to adjusting alignments so as to avoid hills. However, adjustment often will not be possible and the sidehill site must be used. In these cases, special construction practices will consist of some type of benching to key the embankment to a firm foundation and special drainage provisions to prevent the accumulation of surface water and groundwater.

Benching consists of excavating into the sidehill to establish a horizontal platform upon which to construct the embankment. Benching is usually required when the natural slope exceeds 4:1 or 6:1. The specific dimensions of the benches may be given on the design drawings or in special provisions for each project. Sometimes the actual sizes of the benches on specific jobs are established in the field during construction. Often the minimum width of a bench is established on the basis of the width of construction equipment. There is a tendency to make benches too narrow. To be effective, they must be wide enough to allow the embankment to be anchored in firm material. This means, for example, that benches should intercept the transition zone between weathered soil and the unweathered parent material. Typical longitudinal benches are shown in Figure 7.

A variety of drainage control procedures can be used for different groundwater conditions. Many of these, not particularly limited to sidehill locations, are discussed in the section on "Drainage Provisions."

Stabilization trenches are used on some sidehill locations. Although the trenches are constructed primarily to control deep groundwater, they also key the embankment into stable underlying foundation soils. Stabilization trenches are excavated with a bottom width of at least 12 ft and side slopes as steep as possible. The depth of the trench is determined by the position of the groundwater to be intercepted. Excavations have ranged as deep as 35 ft below center line. The bottom, high side, and ends of the trenches are blanketed with a 3-ft layer of permeable material and perforated pipe is laid in the bottom of the trench. Short transverse trenches with perforated pipe are constructed at intervals along the main trench to provide a drainage outlet. The embankment is then started by backfilling the stabilization trench. A schematic diagram of a stabilization trench is shown in Figure 8.

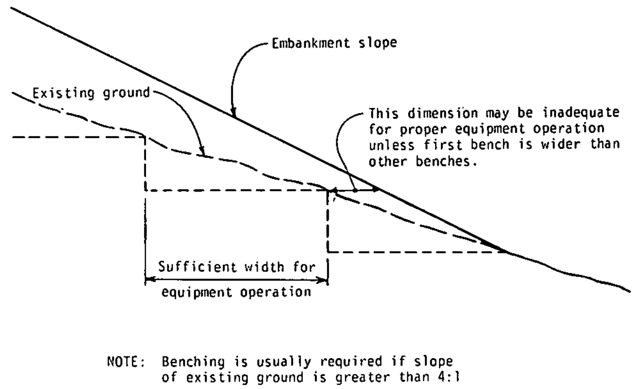


Figure 7. Cross section of typical longitudinal benches.

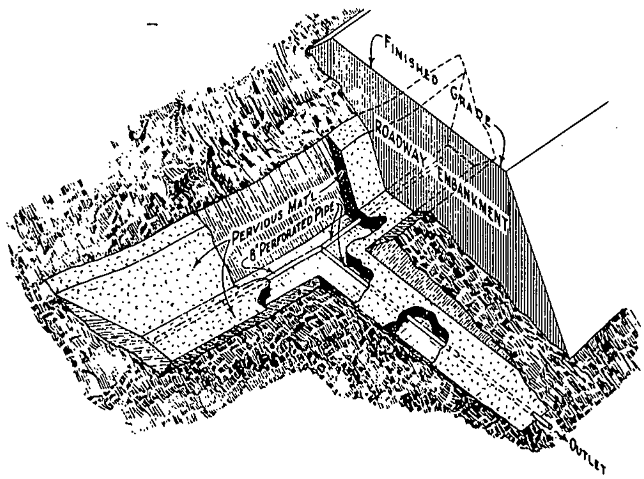
CUT-FILL TRANSITIONS

Cut-fill transitions basically are transverse sidehill locations and thus involve the same considerations discussed in the preceding section. Benches and drainage systems may be required. However, in order to maintain a uniform subgrade, some details of the benches at cut-fill transitions differ from the bench sections in sidehills. The bench must extend sufficiently into the cut zone to ensure that all unstable soil is removed from the subgrade zone. In addition, the uppermost bench is tapered to provide a gradual transition from the embankment to the natural ground. An example of typical benching practice at a cut-fill transition is shown in Figure 9.

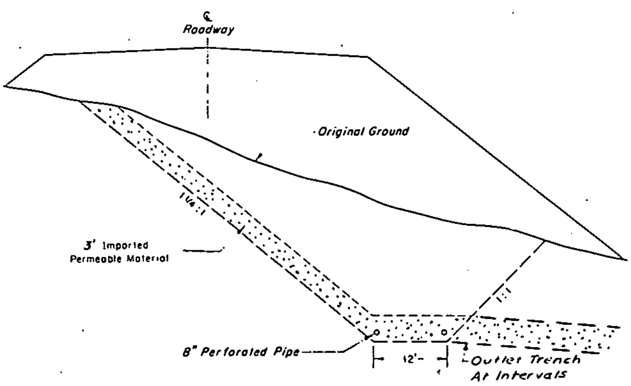
Drainage is particularly critical at transitions because the cut slopes provide a potential source of groundwater. As a result, it is good practice to provide for drainage at all cut-fill transitions. A simple pipe installation is shown in the benched section in Figure 9. If necessary, transverse stabilization trenches or other more complex systems can be constructed.

DRAINAGE PROVISIONS

The control of groundwater is an important aspect of embankment construction except in arid regions. A variety of subsurface drainage systems are used in current practice. Except in a few special cases, they consist of pervious



a. Diagrammetric sketch



b. Cross section

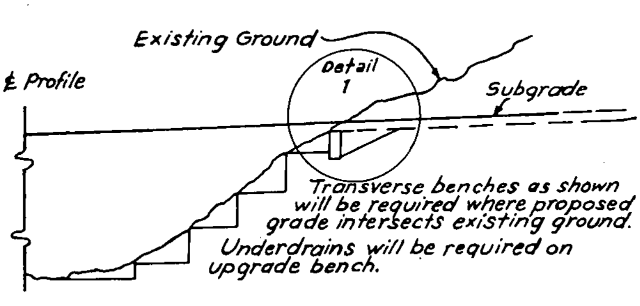
Figure 8. Longitudinal stabilization trench.

blankets and/or some type of drain pipe system. Most of these subdrain systems are very effective when they are designed with sufficient capacity and placed in the proper location.

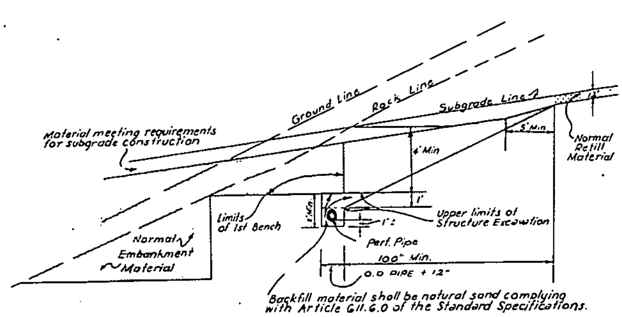
Drainage provisions for groundwater control can be very expensive. For example, one agency has indicated that the cost of subsurface drainage facilities represented approximately 10 percent of one $5 million project and almost 14 percent of another $11 million project. Thus, the economics of groundwater control facilities becomes an important consideration. It may be extremely uneconomical to provide subdrain facilities in locations for which their need is not indicated by the soil survey. On the other hand, the consequences of omitting drainage provisions where they actually are needed can be even more expensive. To quote one soils engineer: "To ignore the subsurface water in the construction of cuts or fills . . . in the hopes that construction or natural conditions will improve the subsurface drainage, can be a disastrous and costly process. Almost without exception it is more economical and more practical to correct adverse subsurface drainage conditions before construction rather than to attempt to handle this situation as a maintenance operation" (Smith, 1964). Some drainage facilities, such as pervious blankets or stabilization trenches, cannot be installed after the embankment has been constructed. Even when the unanticipated groundwater is discovered at the outset of construction, extra costs are likely to develop because of the need for extra materials and resulting construction delays. These economic considerations emphasize the importance of a thorough study of existing and potential groundwater conditions during the preliminary field investigations for the soil survey report.

In relatively flat terrain with high groundwater tables, many problems can be eliminated by ensuring that the grade line is several feet above natural ground. Such grade lines frequently require the abandonment of the concept of balanced earthwork. However, the advantages in relation to drainage and maintenance outweigh the advantages of balanced design. In addition, for low embankments in some northern regions, pervious blankets are used to cut off capillary rise into the embankment and thus to reduce the potential for frost heave.

As the terrain becomes more hilly or mountainous, groundwater problems are likely to become more severe. In addition, the interruption of natural surface runoff must



DETAIL FOR TRANSVERSE UNDERDRAIN
CUT TO FILL CONDITION



DETAIL 1

Figure 9. Typical section at cut-fill transitions.

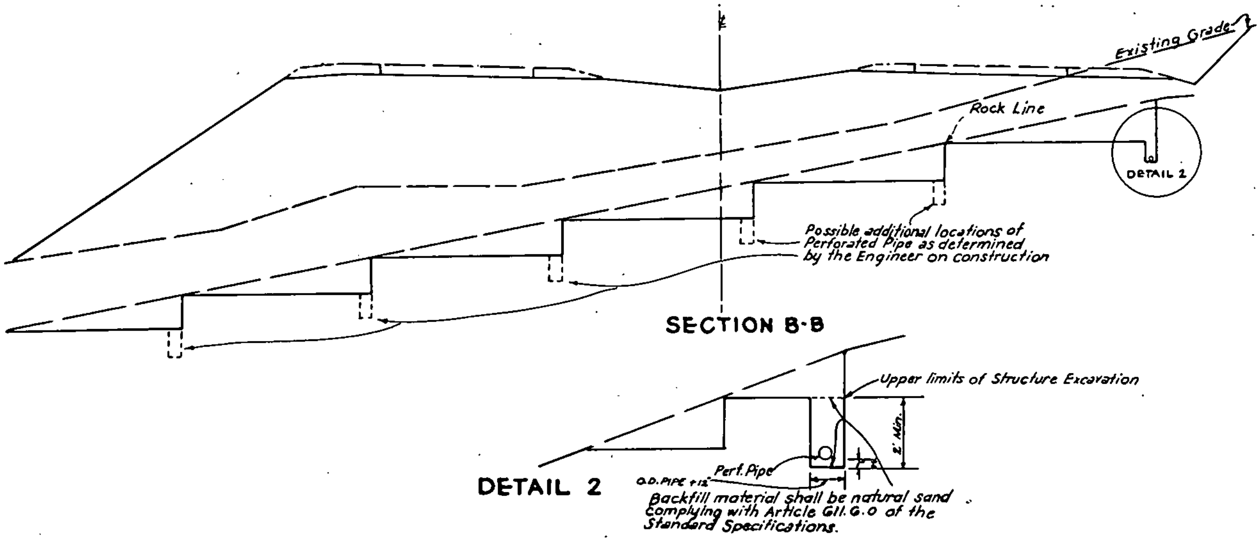


Figure 10. Typical detail for longitudinal underdrain.

be considered. Furthermore, in these locations major subsurface drainage problems can occur in relatively impervious fine-grained soils with fissures, joints, or bedding planes. These conditions do not lend themselves to the use of rigorous analytical solutions for calculating quantities and paths of seepage.

The most commonly used subdrain systems consist of pervious blankets and/or perforated pipe subdrains. These subsurface drainage facilities may be more commonly required in cuts than in fill areas. With respect to embankments, the most common need for subdrain systems will be at cut-fill transitions and in sidehill locations. The specific design of underdrain systems and pervious blankets may vary considerably. Both longitudinal and transverse subdrain systems are used, depending on the topography. Typical subdrain provisions used by one agency at cut-fill transitions are included in Figure 9. Similar installations in sidehill fills are shown in Figure 10. Sometimes longitudinal drains are installed in trenches in cut or natural slopes above fills to intercept groundwater moving down the slope. Whenever pipe drains are installed in relatively impervious materials, it is essential to provide a bed of permeable material for the pipe and to backfill the excavation with the same material.

When large quantities of water are involved, a continuous pervious blanket may be more economical than a pipe drain system. Pervious blankets usually are at least 1 ft thick, and careful consideration must be given to the gradation of the pervious material. The gradation specifications must ensure that the material is coarse enough to be free draining, yet not so coarse as to permit migration of the fine-grained soil into or through the permeable material. Much has been written on the design of filter systems; and although much of this literature is directed toward filters for earth dams, the concepts generally are applicable to highway drainage problems. These design procedures generally relate the grain size characteristics of the filter material to those of the adjacent soil. One commonly used requirement, shown in Figure 11, is that the 15 percent particle size of the filter should be at least five times larger than the 15 percent size and no more than five times larger than the 85 percent size of the adjacent soil layer.

In hilly or mountainous terrain, the method of subsurface drainage will depend on the topography and the depth at which water-bearing strata are encountered. This may be illustrated by considering the variety of drainage methods employed in California. When the water-bearing material is relatively shallow (at depths of less than 10 or 20 ft) and underlain by firm material, the soft wet material may be excavated and replaced with a pervious blanket. When the subsurface water is encountered at depths of 10 to 30 or 40 ft, stabilization trenches, described in the preceding section, may be employed. Occasionally, horizontal drains are used to remove subsurface water at greater depths than is economically practical by stripping or trenches. These drains are installed by drilling laterally from a point at or below the toe of the proposed fill slope. Such drains, which may be installed to lengths of 150 to 300 ft, can reach well beyond the toe of the uphill slope. Sometimes additional drains are installed from the toe of the uphill slope to intercept subsurface water before it reaches the embankment. Usually collection systems are required to remove the outflow from the horizontal drains.

Groundwater at great depth can be intercepted by installing vertical relief wells in conjunction with horizontal drain systems. The primary purpose of the vertical wells is to relieve hydrostatic pressures rather than to provide for complete drainage. Relief wells have been installed to depths of 40 ft. Six-inch-diameter perforated pipe is placed in the center of a 24-in. hole, which is then backfilled with

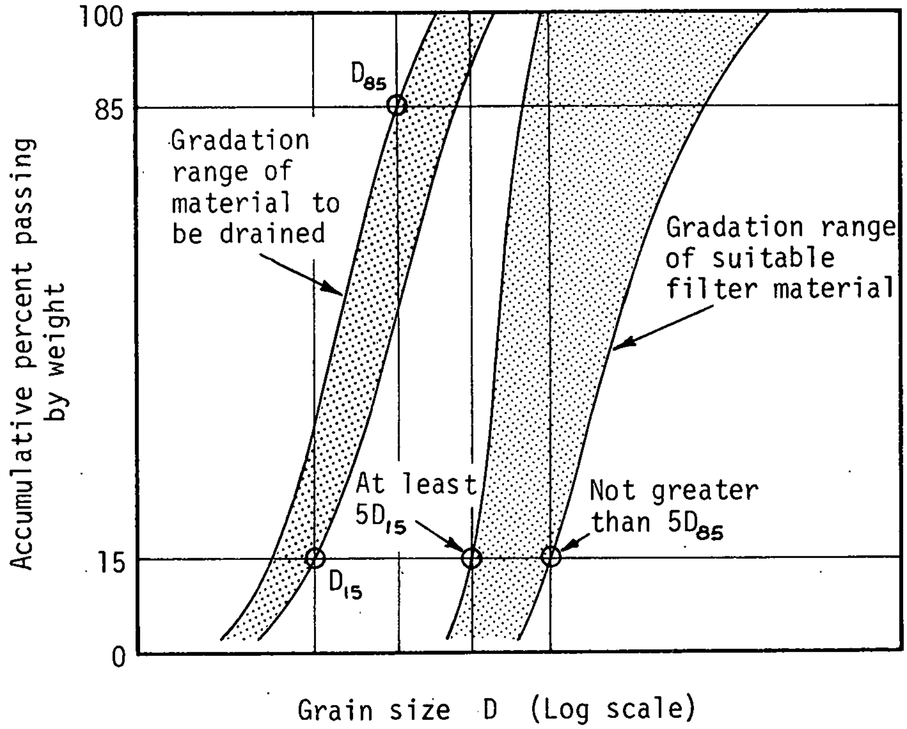


Figure 11. Typical gradation requirements for filters and pervious blankets.

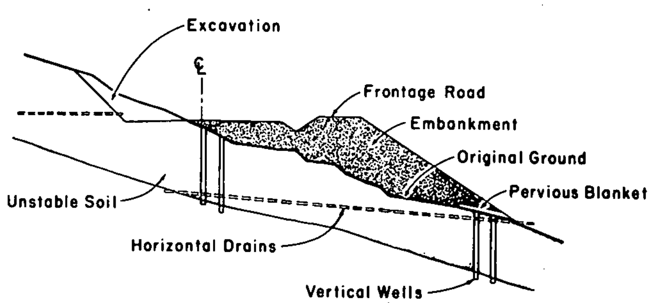


Figure 12. Cross section of typical vertical wells and horizontal drains.

permeable material. Horizontal drains are installed to intercept the relief wells. A cross section of a typical relief well and horizontal drain system is shown in Figure 12.

Subsurface drainage methods, which vary widely in cost and complexity, are available for the control of almost any groundwater condition. The major problem is the identification of the severity of the groundwater conditions at specific sites so that the most economical and effective subdrainage system can be employed. The importance of an adequate site investigation of the groundwater conditions cannot be overemphasized.

CHAPTER FOUR

DESIGN OF FILLS

DESIGN CONSIDERATIONS

In practice, the design of highway fills generally consists of establishing the height and the side slopes of the embankment and specifying criteria for placement of the fill. The placement criteria generally are included in each agency's general specifications and are discussed in Chapter Five. The factors used to establish the size and shape of the embankment are discussed briefly in the following paragraphs.

The height of an embankment depends on the proposed highway grade line, which is established primarily on the basis of geometric design criteria and secondarily on the basis of balanced earthwork. Only rarely does the strength or compressibility of the fill or the underlying foundation material influence the design height. Standard side slopes are established to satisfy safety standards and to facilitate maintenance. Typical standard slopes established on the basis of such criteria are shown in Figure 13. Current trends indicate that the use of 2:1 as a standard design slope is common. Design engineers ordinarily will assume that the standard design slopes will be stable unless stability problems and alternate slope recommendations have been indicated by the soils engineers in the soil survey for the project. When such recommendations are included in the soil survey, they usually are incorporated into the design. Thus, except in cases of poor foundation conditions, described in Chapter Three, the engineering properties of the fill and underlying foundation soils have relatively little influence on the design geometry of the fill.

Balanced earthwork is practiced when it can be accomplished without violating geometric design criteria. However, strict adherence to balanced earthwork design can lead to serious construction and maintenance problems by encouraging the use of poor quality soils from cut sections and the use of locations with poor foundation conditions. To accomplish a balanced design, it is necessary to estimate the shrink and swell factors for the excavated materials and to estimate the quantities of poor quality materials that should be wasted. In areas of relatively flat topography, balanced design appears particularly undesirable because the grade line would be established very close to natural ground level, with frequent transitions between small cuts and low fills. Under such circumstances, construction of a uniform subgrade is made more difficult. Furthermore, surface and subsurface drainage problems may be introduced, and winter maintenance may be made more difficult in northern regions. For these reasons, many engineers now prefer to ignore balanced earthwork concepts and to construct continuous low embankments over relatively flat terrain.

The design load used to evaluate the stability and deformation of an embankment is the weight of the overlying embankment and pavement materials. Except for the upper few feet, embankments are not seriously affected by traffic loads. In current practice, the density rather than the strength or compressibility is specified for fill materials. When fill is compacted in accordance with current density requirements, its compressibility will be negligible. Hence, except for very high fills, the compressibility of compacted materials is not generally evaluated. Similarly, the strength of compacted materials generally is assumed to be adequate to maintain the standard design slopes. As discussed in Chapter Two, stability and settlement considerations primarily concern the behavior of the foundation material beneath the fill. Current field experience tends to justify the assumption that for current standard design slopes and compaction requirements, the strength and compressibility of the compacted fill usually is not a problem.

Exceptions are noted in the case of high embankments, for which the strength and compressibility of the fill materials must be given special consideration. Most agencies define high embankments as those exceeding 40 or 50 ft. In the construction of high embankments, sometimes low-quality materials must be wasted, and in other instances zoning of material must be practiced. When zoning is necessary, the higher-quality materials are placed in the bottom of the embankment where the loads are the largest. Lower-quality materials are used in the upper parts of the embankment and capped with a layer of select material, which will serve as the subgrade subjected to traffic loads. Problem soils sometimes are placed in the center or lower portions of an embankment. For example, expansive soils will perform more satisfactorily deep within an embankment where they are better protected from drastic moisture

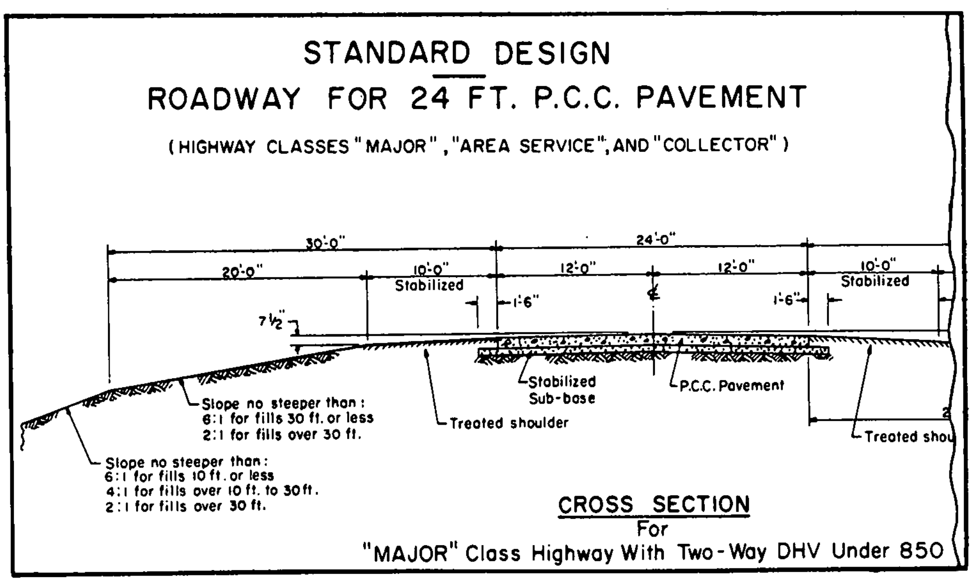


Figure 13. Typical standard design slopes.

changes and their tendency to swell is inhibited by the weight of the overburden. Similarly, micaceous soils may be placed where they will not be subjected to traffic loads.

The strength and the compressibility of the fill must be considered when low-quality materials must be used or soils must be placed under adverse conditions. This situation most commonly arises when extremely wet fine-grained soils must be used. Recognition of such potential situations in the soil survey and consideration of them in the embankment design will minimize problems and delays during construction. For example, if plan notes indicate that certain soils will be troublesome when wet, the construction division and the contractor may be able to schedule construction so as not to expose these soils during rainy seasons. In other instances, plan notes or special provisions may indicate special zones in which wet soils may be placed or special construction procedures (e.g., sandwiching with good materials).

The design of earth embankments is more significantly influenced by geometric design criteria, safety standards, and maintenance requirements than by the strength and the compressibility of the embankment and its foundation. When stability and settlement are the dominant factors, the embankment design more commonly is influenced by the properties of the foundation than by those of the fill material. In other words, current procedures and specifications for placement of fill are producing compacted materials with adequate engineering properties for current standard designs, except for the special situations outlined in the preceding paragraphs.

PREPARATION OF PLANS AND SPECIFICATIONS

Construction practices for the improvement of foundations for embankments are reviewed in Chapter Three. It is emphasized that the need for these construction procedures must be identified during the soil investigations and incorporated into the project design. In the preparation of design drawings and specifications, special treatments for embankment foundations should be indicated in sufficient detail by means of special provisions and plan notes. Complete subsurface information, including boring logs and soils profiles, should be presented on the construction drawings. Similarly, the need for special embankment placement procedures, such as zoning of materials or anticipated placement under adverse conditions, should be specified clearly on the plans. Failure to adequately inform the contractor of subsurface conditions, special problems, and procedures when contracts are let inevitably leads to additional costs and delays when the unanticipated conditions develop during construction.

We need to do more of this

CHAPTER FIVE

PROCEDURES AND REQUIREMENTS FOR PLACEMENT OF FILL

ROLLED EARTH FILLS

The standard specifications of all state highway agencies require rolled or compacted fills, in which the fill is spread in relatively thin layers and compacted by rolling, usually with special compaction equipment. Some specifications indicate the detailed procedure by which the compaction is to be accomplished, including moisture control, lift thickness, type and size of compaction equipment, and the number of coverages of the equipment. Alternately, the required density of the compacted soil may be specified. In this case, the lift thickness and moisture content commonly are controlled, and minimum equipment standards sometimes are indicated. The combination of a more detailed procedural specification and a minimum density requirement can lead to potential legal problems if the contractor adheres closely to the detailed procedure and yet is unable to achieve the required density. However, a minimum compactive effort can be required in conjunction with a density standard in order to improve the uniformity of compaction.

The current trend in compaction specifications for embankments is to minimize the procedural requirements and to place greater reliance on density requirements. The contractor is being given more freedom and responsibility for the selection of equipment and procedures that will produce satisfactory results. Sometimes it is more economical for the contractor to use compaction equipment already on the project with increased effort, if necessary, rather than to secure different equipment that may be more efficient for a particular material condition. Similarly, additional compaction effort at existing moisture contents sometimes will be more economical than adjusting the moisture to optimum. Lower costs for the contractor generally result in lower bids for the work.

Currently, all highway agencies have density requirements in their standard specifications for embankments, but approximately 25 percent of the agencies use alternate specifications for compaction without density control for certain types of construction. In these cases, the specification may require a minimum number of passes of a specific piece of equipment or the use of compaction equipment to the visual satisfaction of the inspector. In practice, the latter type of specification generally requires the walkout of a sheepsfoot roller or the use of a pneumatic roller until there is no further observable densification of the soil. Also, density requirements frequently are eliminated for rock fills and hydraulic fills, which are discussed separately in subsequent sections of this chapter.

Lift Thickness

The maximum lift thickness is most conveniently specified in terms of the loose or uncompacted thickness. For earth fills, satisfactory uncompacted lift thicknesses range from 6 to 12 in., with 8 in. being most common. Thicker lifts can be allowed on individual projects if the contractor can demonstrate that he can achieve satisfactory compaction. Lifts of 12 in. or greater may be satisfactory for granular soils, particularly for vibratory compaction.

Lift thickness requirements sometimes must be relaxed for placement of the initial lifts over soft foundation materials. An 18- to 24-in. mat of uncontrolled fill may be allowed to provide the contractor a working base for his equipment. However, sometimes the contractor must be required to start construction with light equipment instead of constructing an excessively thick construction mat.

Equipment

Equipment specifically designed for compaction is required for most embankment construction. Relatively few equipment requirements are specified, but usually the equipment must be approved by the resident engineer. In practice, minimum equipment standards appear to be of little practical concern to most highway engineers. Most contractors are using adequate equipment with regard to both size and type of compaction. Sheepsfoot and pneumatic-tired rollers are most commonly used for cohesive soils. In addition, various soils have been compacted satisfactorily with smooth-wheeled and segmented rollers, vibratory compactors, and specialized equipment that combines compactive actions (e.g., the vibratory sheepsfoot roller). Typical examples of several common types of compactors are shown in Figure 14.

For clean well-graded granular soils, vibratory compactors clearly are more effective than other compaction equipment. However, all of the factors that influence vibratory compaction are not fully understood. Thicker lifts may be permissible, and even desirable. It is likely that vibratory compactors will be more widely used in the future as their capabilities become more clearly defined.

For other soils, no one type of compactor is markedly superior to all others. Specific comparisons among different types of compactors are complicated by the fact that the performance of each type is significantly influenced by many characteristics, summarized in Table 1. Many full-scale investigations have been devoted to the study of factors that influence field compaction, perhaps the most extensive being those conducted since the late 1940's at the Waterways Experiment Station of the U.S. Army Corps of Engineers. More recently, the Illinois Institute of Technology Research Institute (IITRI) conducted a major study of field compaction equipment for the U.S. Bureau of Public Roads (Federal Highway Administration). The results of these studies have been published by these agencies. Many of the factors influencing field compaction also have been summarized in several publications of the Highway Research Board, particularly *HRB Bull. 272* and *Hwy. Res. Record No. 177.*

In practice, a considerable amount of embankment compaction is attempted with hauling equipment. Although hauling operations can produce significant densification of earth fills, compaction solely by hauling operations is considered inadequate. Uniform coverage and, as a consequence, uniform density generally are not achieved. To improve uniformity, compaction by hauling equipment should be supplemented by rolling with compaction equipment.

The heavy loads imposed by hauling and paving equipment create a major problem in some embankment construction. The wheel loads from this equipment may produce higher stresses in the compacted soils than the stresses to be anticipated from traffic loads after the road is in service. In many states, examples can be cited of heavy

TABLE 1

CHARACTERISTIC VARIABLES PERTAINING TO THE DIFFERENT TYPES OF FIELD COMPACTION EQUIPMENT

TYPE OF FIELD COMPACTOR	CHARACTERISTIC VARIABLES
Tamping-type rollers (sheepsfoot, segmented)	Gross weight, type of foot, number of feet/drum, area of foot, foot contact area (%) of total area of cylinder generated, length of feet, dimensions of drum, contact pressure
Pneumatic-tired rollers	Gross weight, roller dimensions, type of roller, tire inflation pressure, contact pressure, contact area
Smooth-wheel power rollers	Gross weight, diameter of rolls, width of rolls, and compression (#/in. of roller)
Vibratory compactors, base-plate type	Gross weight, weight of each vibrating unit, contact area of base plate, unit contact pressure, frequency, amplitude, dynamic force, speed of travel
Vibratory compactors, roller type	Gross weight, roller weights, dimensions of roller, frequency, amplitude, speed of travel
Track-type tractors	Gross weight, width of track, contact pressures, speed of travel
Tampers	Weight, area of base plate, height of jump

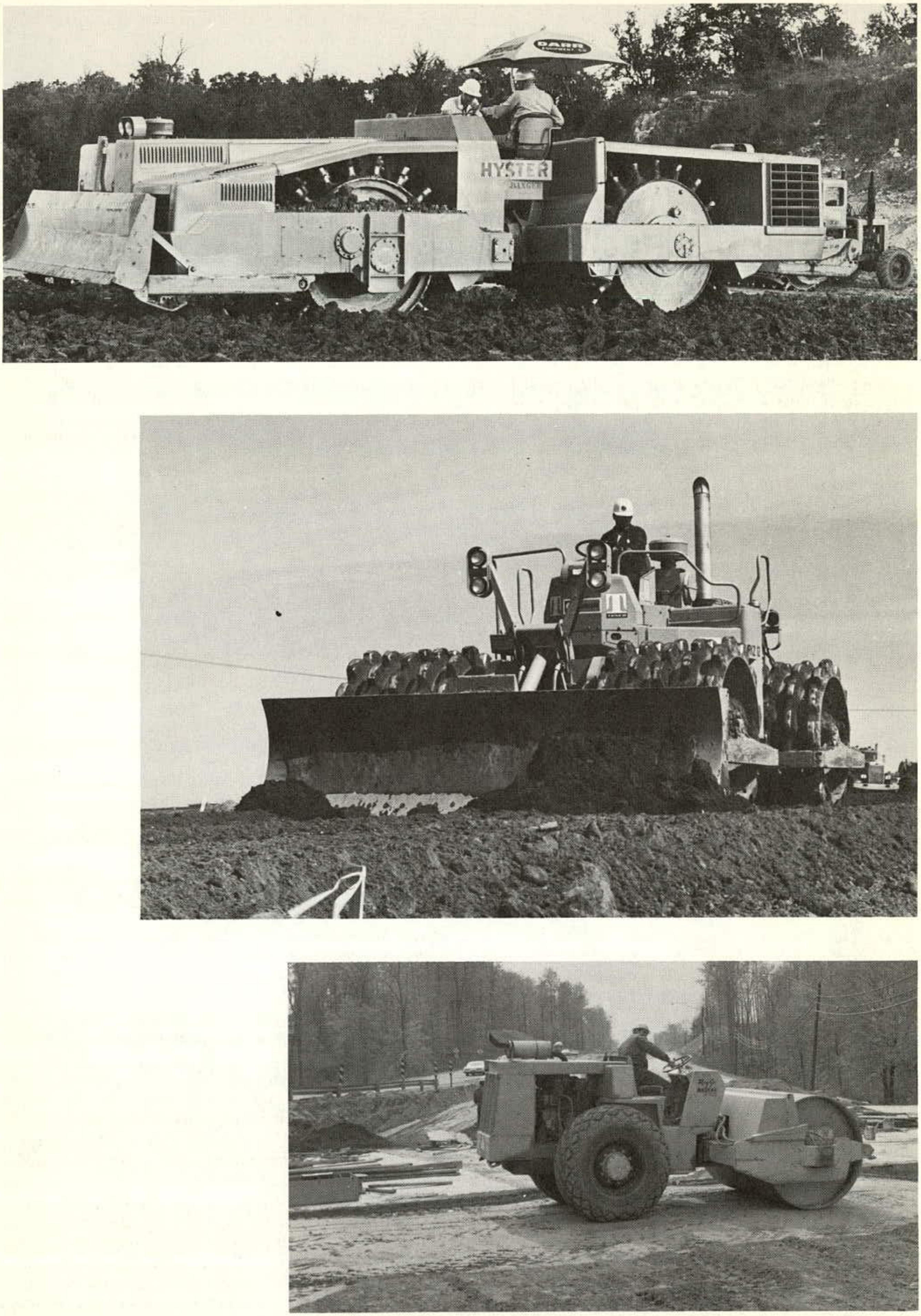
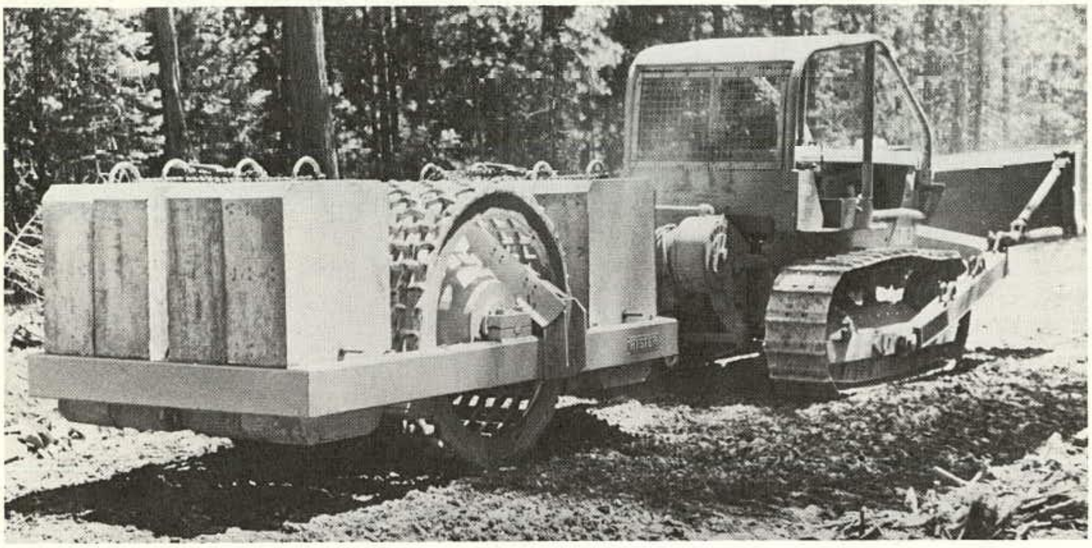


Figure 14. Examples of typical compaction equipment.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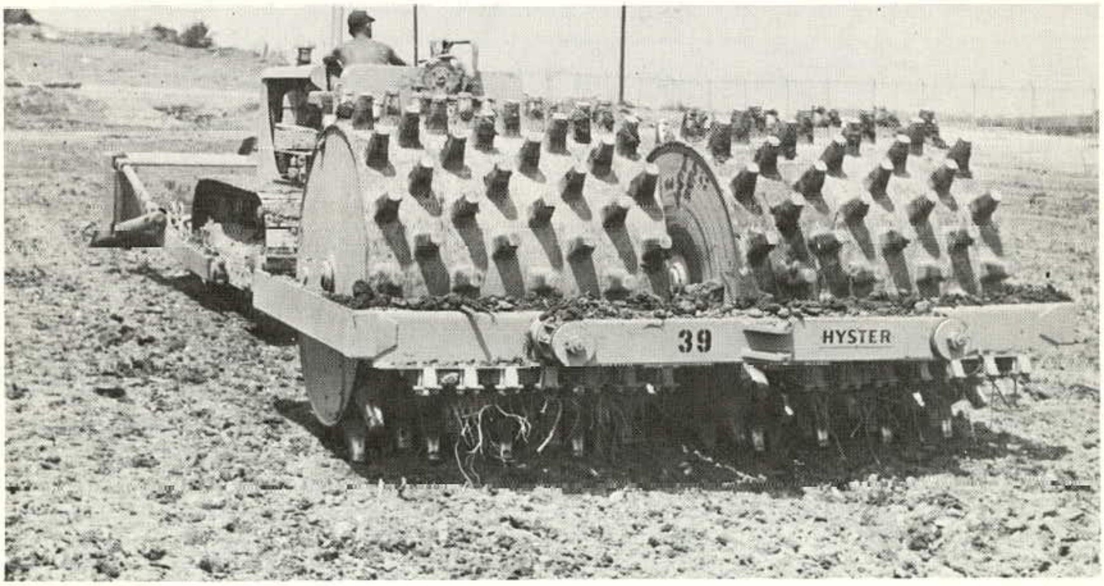
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construction equipment causing stability failures in compacted embankments that had already satisfied compaction specifications. Almost all of the cited problems occur in silty materials that are extremely sensitive to moisture and density conditions. Many engineers anticipate that these problems with heavy equipment will become more common in the future unless corrective measures are introduced.

Moisture Control

Proper moisture conditions are essential for the efficient compaction of earth embankments. Satisfactory moisture conditions almost always are specified in relation to the optimum moisture content for a standard laboratory compaction test. Specific quantitative maximum and/or minimum moisture contents may be specified. Alternately, moisture requirements may be expressed qualitatively, such as "near optimum," "as required by the engineer" or "as required for compaction." In the field, the latter terminology usually is interpreted by the engineer in relation to the laboratory optimum moisture content for the soil. There are practical advantages and disadvantages to both qualitative and quantitative moisture specifications; highway engineers differ regarding the more desirable of the two types.

General quantitative moisture requirements should be applied with caution. Because compaction characteristics differ with soil type, it often is difficult to specify one moisture content range for all soil types. Furthermore, the optimum moisture content and the appropriate moisture range for field compaction depend not only on soil type but also on the type and size of the compaction equipment. For example, because the optimum moisture content decreases with increasing compactive effort, heavier compaction equipment frequently operates more efficiently at moisture contents that are less than the laboratory optimum moisture content. This problem can be minimized by specifying only an upper quantitative moisture limit, which appears satisfactory except for expansive clays.

There is a growing tendency by some highway agencies to minimize specific moisture requirements as long as density requirements are being satisfied and a stable embankment is being produced. In effect, the responsibility for proper moisture control is being shifted to the contractor. In some instances, the contractor will find it mandatory to adjust the moisture conditions to obtain satisfactory compaction. In other cases, particularly on the dry side of optimum, the required density might be obtained more economically by extra passes of the available equipment rather than by adding moisture. However, if the required density is not achieved or excessive weaving or rutting is observed under the compaction equipment, specific quantitative moisture specifications are invoked.

Close moisture control is necessary for silty soils and swelling clays. The behavior of silts is extremely sensitive to moisture content. Slight increases in moisture can lead to instability under compaction equipment, even though adequate densities are being attained. Consequently, stable embankments are very difficult to construct unless the moisture is closely controlled. Field moisture control in silts is directed toward compaction at moisture contents slightly less than the optimum.

Swelling clays should be compacted on the wet side of optimum because the swelling potential is considerably less than for dry-side compaction. This is often true even in arid regions where the moisture contents of swelling soils in their natural state are much lower than optimum. In extreme cases where sufficient water is not available, alternate procedures for reducing swelling potential may be attempted. Special procedures used for swelling soils are discussed further in Chapter Seven.

When additional moisture is required, better moisture control generally is obtained when the water is added at the excavation. When borrow pits are used, the water commonly is added prior to excavation, using spray or sprinkler systems. However, when material from cuts is used, the water generally is added on the fill with water trucks equipped with splash bars or spray attachments. The addition of spraying equipment reduces ponding of water in tire ruts and thus significantly improves the distribution of moisture. Disc harrows and various types of mixers frequently are used to obtain more adequate mixing of water and cohesive soils. Specialized equipment for pulverizing clays and simultaneously adding water also is available but is rarely used. Decisions regarding where and how moisture will be added often are the contractor's responsibility. However, because improper prewetting procedures can create serious construction problems, prewetting procedures and equipment should be subject to the highway agency's approval.

When natural soils are too wet, discing frequently is used to improve the rate of drying through aeration. Discs should be required on jobs where wet cohesive soils are anticipated. In addition, contractors are encouraged to blade a crown on the surface and to roll the surface to seal it. This practice decreases rewetting caused by rainfall and frequently eliminates construction delays. However, these procedures also may increase the potential for erosion of slopes, and the need for special erosion control procedures should be considered.

Severe moisture control problems arise in very wet climates, such as the Pacific Coast of Oregon and Washington, where the natural soils are very wet and the climatic conditions hinder natural drying. Under such circumstances, it is sometimes impossible to dry cohesive soils satisfactorily. If these soils are not wasted, they must be compacted at moisture contents much higher than optimum. For these conditions, density requirements occasionally must be reduced to levels that can realistically be attained at the field moisture content. Embankments should be designed with this in mind. Notes then should be added to the plans to warn of potential instability.

To overcome extremely wet conditions, sandwich construction can be successfully used for embankment construction in regions such as New England, where ample sources of granular materials are readily available. The wet cohesive soil and coarse granular material are placed in alternate layers. However, the procedure is impractical when granular materials are not plentiful.

Density Requirements

The density requirements for embankments are based predominantly on the AASHO T-99 Compaction Test or a similar test with an approximately equivalent compactive effort. The most common requirement, used by approximately one-half of the state highway agencies, is 95 percent of the maximum dry density obtained in the T-99 test. An additional one-third of the agencies use density requirements that range from 90 to 102 percent of the maximum density from the T-99 test or its equivalent. Only six state agencies specify density in terms of the AASHO T-180 test, and one agency uses a special test that produces densities approximately equaling those from the T-180 test. Field densities of 90 to 95 percent of the T-180 maximum density are required by these agencies.

Almost all density requirements are specified in terms of dry density (i.e., weight of solids per unit volume). However, one agency currently specifies its requirements in terms of wet density, and at least one other agency is considering such a specification. The primary advantage of using wet density is the elimination of the need for moisture content measurements. When the field moisture content is less than optimum, the percent of wet density is lower than the percent of dry density. On the wet side of optimum, the reverse is true; however, the differences are relatively small. Nevertheless, a wet density specification does have the disadvantage of encouraging the contractor to add water and to compact at as wet a condition as possible.

Higher densities, which are usually equal to subgrade requirements, may be required in the upper 1 to 6 ft of an embankment, where stresses are produced by wheel loads. However, some highway engineers feel that the density required in the upper parts of the embankment can be economically produced uniformly throughout the entire embankment, because the contractor will furnish equipment that can provide the higher density levels with a reasonable number of coverages. Consequently, except for very high fills there appears to be little interest in variable density requirements, as a function of position within an embankment.

Most highway agencies appear satisfied with their current density requirements and are not considering significant changes. Furthermore, many of the changes in recent years have been directed toward simplifying density requirements without significantly affecting the required density levels. Many variations in density requirements with material type, maximum dry density, or elevation have been eliminated or simplified. In addition, several changes in density requirements have been introduced to accommodate statistical quality control concepts, discussed in Chapter Six.

ROCK FILLS

In many regions, materials containing appreciable quantities of rock are commonly used to construct high-quality embankments. For embankment construction, rock is defined as a material containing more than 25 to 35 percent by weight larger than the ¾-in. size. Materials classified as rock by this definition range from rocky soils, such as many glacial tills, to fragments from rock excavation, which contain relatively few soil-size particles.

Procedural specifications usually are employed for placement of rock fills because the density control procedures for soils become less reliable when rock-size particles are encountered. Typically, the lift thickness and a minimum number of coverages of a smooth-wheeled roller or a vibratory compactor are specified. For example, one agency specifies a minimum of six passes of a 10-ton smooth-wheeled roller, whereas another requires four passes of a tamping roller or two passes of a vibratory roller.

The lift thickness requirements for compacting soil generally are relaxed to accommodate the larger particle sizes in rocky fills. The maximum lift thickness commonly ranges from 2 to 4 ft and also dictates the maximum particle size that is permitted. Two-foot lifts and maximum particle sizes are perhaps the most commonly used specification. However, there is some indication that thicker lifts can be successfully placed with heavy vibratory compaction equipment. One agency obtained a significant cost reduction on a recent project by permitting vibratory compaction of 4-ft lifts with 3-ft maximum particles instead of requiring 2-ft lift thicknesses.

The experience of many highway engineers indicates that adequate density and stability of rock fills frequently is produced by proper spreading procedures. Typically, the rock fill is dumped at some distance (on the order of 25 ft) from the end of the fill and pushed forward by bulldozers or grading equipment to form the allowable lift thickness. The vibrations produced by these procedures often develop adequate density without additional rolling. Sometimes water is sluiced or jetted into the fill to improve the densification of rock fills containing only minimal amounts of fines. The minimum rolling specification is then regarded as a proof test.

Most highway engineers believe that high-quality rock embankments are easily constructed using current placement procedures. However, in the future it may become more economical to use thicker lifts in conjunction with heavy vibratory compactors. Finally, although past experience with rock fill embankments has been very good, there remains a need for a procedure with which to document the quality of rock fills at the time of construction.

HYDRAULIC FILLS

Hydraulic procedures occasionally are used for constructing highway embankments at river crossings. Hydraulic fills usually are 20 to 30 ft high, with the lower 10 to 15 ft under water where rolled earth procedures would require dewatering. These fills are constructed from granular materials and capped with cohesive soil to prevent scour and erosion. Almost all of the hydraulic fills appear to be performing satisfactorily.

Hydraulic procedures rarely are required in design specifications but sometimes are permitted as an alternate under special provisions, which should specify the details of excavation, transportation, and placement procedures. In addition, gradation requirements must be carefully controlled to limit the percent of fines. Density requirements generally are omitted.

Hydraulic procedures appear to produce adequate densities under many circumstances. However, hydraulic fills may remain relatively loose when placed at elevations below the natural ground or surface water level. For such cases, additional densification can be achieved by requiring several passes of a vibratory roller when the surface of the fill first reaches an elevation slightly above the water level.

CHAPTER SIX

QUALITY CONTROL PROCEDURES

Quality control and acceptance procedures include density and moisture measurements, test rolling, and visual inspection. In current practice, most compaction control is accomplished by measuring the field density and comparing it to the maximum density obtainable for this material in a specified laboratory compaction test. Because the control generally is in terms of dry density, the measurement of field moisture content also is involved, even when moisture requirements are not specified quantitatively. Consequently, this chapter is concerned primarily with current practices for obtaining field moisture and density measurements and procedures for converting these measurements to relative densities or percent compaction. Other control techniques are discussed briefly at the end of the chapter.

DENSITY CONTROL PROCEDURES

Types of Field Moisture and Density Tests

The types of unit weight, or density, tests are summarized in Table 2. In general, no single method dominates current use. In some agencies the test methods vary from one district to another, with the local selection governed by the personal preferences of the district personnel. Often, strong local opinions exist regarding the relative reliability of the various test methods. These attitudes seem to be related closely to local experience and details of local testing procedures.

The most common field density tests are destructive tests that involve digging a hole, determining the weight and moisture content of the soil removed, and determining the volume of the hole created. The two most prevalent destructive test methods are the sand-cone method and the balloon method, in which the volume of the hole is determined by refilling it. Sand is used in the first case and water (in a rubber membrane) in the latter. The major disadvantage of these tests is the length of time required to conduct the test, which severely limits the number of tests that can be performed without delays to construction.

The relative accuracy of sand-replacement and water-replacement tests is significantly affected by the care with which each test and the related calibration procedures are performed. For example, the accuracy of the sand-cone method depends on the unit weight of the reference sand, which is influenced by the gradation of the sand and moisture, or humidity, conditions during the test. Hence, it is good practice to calibrate the reference sand in the sand-cone apparatus before each test. This is easily done by determining the weight of sand required to fill the calibrated volume of the sand-cone apparatus. The accuracy of both test methods also can be improved by increasing the volume of the sample. This often is necessary for coarse materials (for example, crushed stone or rocky glacial tills), because the true volume becomes more difficult to assess when large particles are encountered. In addition, a variety of rock correction procedures have been developed for adjusting test results to compensate for the rock content.

Undisturbed drive samples sometimes can be used to supplement sand- or water-replacement tests. This method is relatively rapid and effective in fine-grained soils. Undisturbed block samples can provide even more reliable densities of cohesive soils, but are rarely used because of the time and care required to obtain good samples. Both drive samples and block samples are difficult to obtain in rocky clays and almost impossible to obtain in cohesionless soils.

In recent years there has been an increasing use of nondestructive nuclear tests for both moisture and density measurements. Surface nuclear gauges using backscatter, direct transmission, or air gap concepts are employed for compaction control. The depth of material surveyed is 3 to 4 in. by the backscatter method, and somewhat less by the air gap method. The depth of material surveyed by the direct transmission method may be predetermined through a range of 2 to 12 in. The chief advantage of the nuclear methods is the speed of the operation, which permits many more density measurements to be obtained. The major disadvantage is the high cost of the equipment. Early concern for the reliability of the nuclear density measurements has decreased significantly in recent years. Results of numerous research investigations have shown that measurements by nuclear methods are at least as accurate as those by other commonly used methods.

Although the time required for an individual density measurement is much less for nuclear methods than for destructive tests, calibration and repair time also must be considered when evaluating the over-all time saving provided by nuclear methods. Early nuclear equipment required frequent recalibration for each individual soil. However, recalibration for varying soil types generally is not required when direct transmission and air gap methods are used, because results by these methods are not significantly influenced by the chemical composition of the soil. An exception is iron content in soil, which affects results and must be taken into account by recalibration. The introduction of nuclear methods produces only a small percentage reduction of the total testing time when, as a part of the control procedure, a standard compaction test is conducted in the field in conjunction with each field density test. However, most organizations that have used nuclear methods for control over a period of several years are convinced that the advantages far outweigh the disadvantages. Although a few agencies are decreasing their use of nuclear methods, the general trend is toward more widespread use of such methods. In some instances, the rate of increase in the use of nuclear moisture-density tests is controlled by the cost of purchasing additional equipment.

Seismic methods have been considered for nondestructive density testing of compacted soils. However, the initial studies have been relatively unsuccessful, and this method does not appear to be receiving further serious consideration.

Almost all density control is based on dry density, and thus moisture content determinations usually accompany field density measurements. When destructive tests are used, a sample of the soil removed from the hole is used to perform a moisture content test. The standard laboratory technique of drying overnight in a thermostatically controlled oven generally is too slow for construction control. Consequently, for many years the prevalent field procedure has been to dry the soils over an open flame. This method, which is still commonly used, is relatively satisfactory for coarse materials but somewhat unreliable for fine-grained soils. In recent years, devices that make use of chemical reactions, microwaves, and infrared heat have been developed for rapid moisture content determinations. Experience indicates that these devices are at least as reliable as the open flame method for fine-grained soils. However, many of these devices can accommodate only a relatively small sample and, consequently, are unsuitable for use with coarse materials.

Moisture contents also can be estimated rapidly from a family of moisture-density curves in conjunction with one-point field compaction tests, discussed in the following section.

TABLE 2

UNIT WEIGHT TESTS

GROUP I	GROUP II	GROUP III
Destructive		Nondestructive
Disturbed	Undisturbed	
1. Sand replacment: (a) Glass jar and funnel (b) Glass jar and cone (c) Sand density cylinder 2. Water replacement: (a) Water balloon (b) Dens-O-Meter 3. Oil replacement	1. Block sample 2. Drive cylinder	1. Nuclear 2. Seismic

Field Evaluation of Standard Maximum Compacted Dry Density

In current practice, almost all density requirements are expressed as a percentage of the maximum density attained by a specified compaction test procedure. One of the major problems in the practical interpretation of a density criterion is determination of the proper dry density to which the measured field density should be compared. Typically, laboratory compaction tests are performed on representative samples of primary materials prior to construction. The moisture-density or control curves from these tests generally are available for field control. A common field control procedure is to compare the measured field density to the control curve that the inspector judges is most representative of the compacted material. To aid in relating control curves to field materials, a library of jar samples of materials for which the control curves were attained is sometimes kept at the job site to facilitate visual identification of materials. However, in many instances the primary materials are mixed in the earth-moving operations, so that none of the laboratory curves may be directly applicable to the material being placed on the fill. To overcome this problem, a field evaluation of the maximum dry density often is conducted in conjunction with field density measurements.

The most common procedures for the field evaluation of maximum dry density involve the use of a one-point field compaction test and of a family of moisture-density curves. One common practice is to use the laboratory compaction test procedure to compact the field material at the placement moisture content. The moisture-density point so obtained is plotted with the family of control curves for the job, and the maximum dry density for the material is estimated by constructing a new moisture-density curve through the test point and roughly similar in shape to the available curves. Many agencies now have developed elaborate statewide collections of typical moisture-density curves. The Ohio family of curves, which was among the earliest sets, is shown in Figure 15. In the Ohio system, the penetration resistance as determined by the Proctor needle is used in conjunction with the field compaction test. The statewide family of moisture-density-penetration resistance control curves is then used to estimate the maximum dry density and optimum moisture content for the field material and to determine the moisture content at which the soil is being compacted. Ohio now uses a circular slide rule, which is supplied to all inspectors, to simplify identification of the proper typical curve from the one-point field data. The Ohio moisture-density curves have served as the model for families of curves developed by other

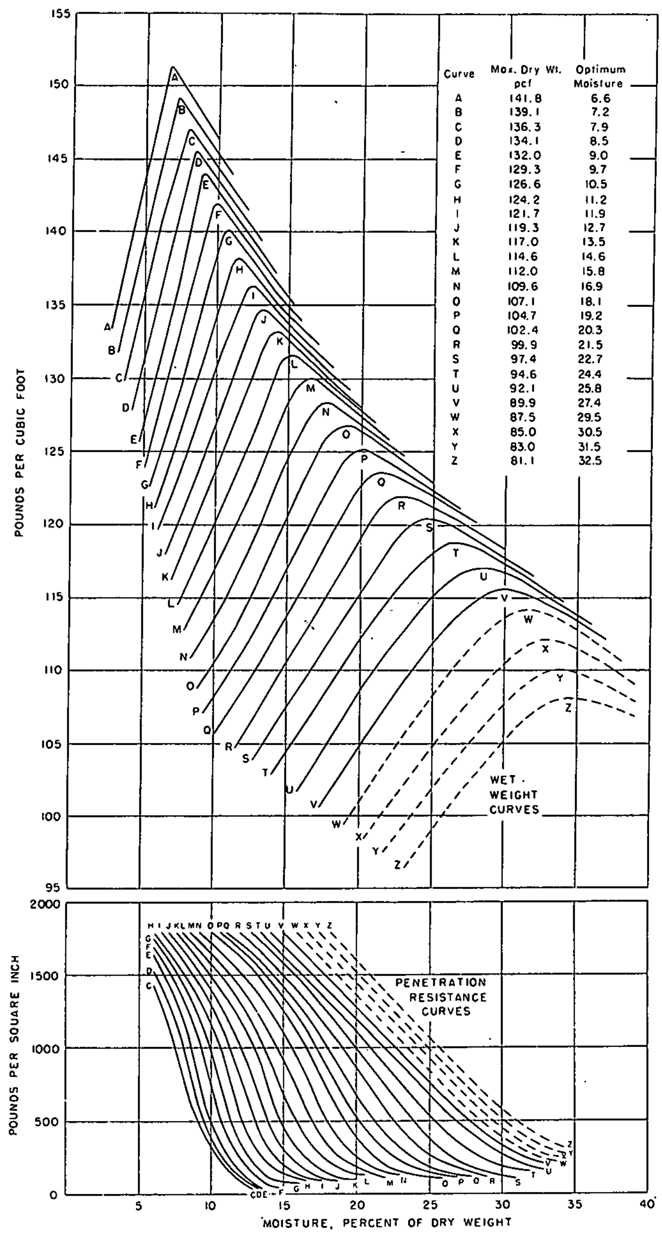

Figure 15. Typical moisture-density curves (Ohio Dept. of Highways).

agencies. Commonly, some of the Ohio curves are adopted and additional curves are added to represent local soils that do not fit the Ohio curves. For example, South Dakota has added a group of curves representative of the Pierre shales, and both South Dakota and Louisiana have added curves for lime-treated soils.

Control practices, which require at least a one-point field compaction test in conjunction with each density field test, are time consuming and can potentially delay construction. However, such control practices are necessary because of the great variations in the materials encountered in highway construction. The area concept, which has been introduced in California in conjunction with nuclear control tests, decreases delays by requiring only one field compaction test for each six field density tests within an area judged to be uniform.

Conventional field density test procedures and the laboratory impact compaction test are unsuitable for soils that contain large quantities of rock. A variety of procedures have been developed to correct for the coarse-aggregate content in compaction tests. These have been reviewed in a number of publications (e.g., *HRB Bull. 319*). However, it is generally agreed that the reliability of these rock correction methods decreases as the percentage of rock increases.

Number and Location of Tests

The frequency of density testing for each project should be determined by the field engineer on the basis of the degree of difficulty of the project. The types and variability of soils, the climatic conditions, and the experience and cooperativeness of the contractor should be considered. Often it is desirable to require more tests when a project is first initiated and then to reduce the rate of testing when the job is progressing smoothly. For the majority of embankments, satisfactory compaction is achieved without difficulty and only a minimum of test results is required. However, on the relatively few jobs for which problems are encountered, much higher frequencies of testing are necessary. In the words of one engineer, "80 percent of our testing is done on 20 percent of our compacted materials."

During recent years minimum testing requirements in terms of tests per unit length of roadway or per cubic yard of material placed have been developed by most highway agencies, largely with the encouragement of the Federal Highway Administration. Typical minimum frequency requirements are one test per 2,000 cu yd of embankment or one test per lift per 1,000 lin ft of embankment. Also, a minimum number of tests per day sometimes is specified for embankments that are being actively worked by the contractor. However, it is important to ensure that the minimum frequency of testing does not become the standard frequency of testing for all jobs and that, as a result, insufficient testing be conducted where problems exist.

The selection of specific locations at which density tests will be conducted has traditionally been left to the judgment of the inspector. Frequently, inspectors are instructed to look for average, or representative, conditions when selecting a testing site. Sometimes he is instructed to look for weak spots, on the premise that the density requirements represent a minimum standard that all materials must meet. Also, it is argued that if the weaker-appearing spots can be shown to be acceptable, the entire fill should be satisfactory and the number of tests required can be substantially reduced.

In recent years, random sampling techniques, which are a requirement of statistical quality control procedures, have been proposed for selecting test locations. Currently, these concepts are rarely being used for embankment control. One exception is the California area concept specification. In this method, the engineer first designates an area that in his judgment is uniform with respect to soil type and placement conditions and procedures. The area should appear uniform by visual inspection (i.e., any obvious soft or wet spots are excluded from the area). The designated area is divided into two or more subareas of approximately equal size. Then specific locations for density tests are selected randomly within each subarea. Thus, the area method combines engineering judgment and random sampling in the selection of test locations.

Compliance with Specifications

It is the goal to achieve compliance with specifications throughout the embankment. However, density tests show conditions only at the location where they are made and specifically represent only a minute portion of the fill. Other areas must be related to the areas tested by engineering judgment and observations of the construction process. Continuous visual inspection is necessary to ensure that proper construction procedures are being consistently followed with respect to such items as lift thickness, maintaining proper moisture conditions, maintaining a systematic pattern of roller coverages, and adequate rolling of all areas. Deficiencies should be identified and corrected on a current basis as the work proceeds. Such inspection also provides a basis for selecting locations for tests that will be meaningful for checking compliance with specifications. For example, areas that visual inspection shows to be questionable should be tested.

With the exception of several recent specifications based on statistical concepts, density requirements are considered to be minimum standards that must be exceeded by all field test results. If an unsatisfactory test result is obtained, the material is rerolled with or without moisture adjustments or removed and replaced, depending on the nature and severity of the deficiency. Usually the material is retested for compliance with specifications, or, if the deficiency was minor, it may be accepted by visual inspection after rerolling. It is the engineer's responsibility to give the inspector specific instructions on his duties to secure compliance with the specifications, and to supervise his work to ensure that proper practices are being followed. The engineer must also establish with the contractor the understanding that the inspector is the engineer's representative on the work, and that the inspector's instructions must be followed.

Statistical concepts for density requirements help to evaluate the significance of an occasional bad test. For any statistical distribution of test results, the probability of an unfavorable test result can be reduced only by raising the mean value or reducing the standard deviation of the test results. Even for closely controlled field experiments the standard deviation can be expected to exceed 2 pcf. Thus, for normal construction conditions an occasional unfavorable test can be anticipated even when the average density of the fill is 5 percent greater than the minimum requirements.

Statistical specifications usually indicate both a required average density and a minimum density that all tests must exceed. Several agencies currently specify density requirements in this manner. For example, in California six density tests are performed in each uniform area, and the results of each test are expressed as a percentage of the maximum density obtained from one standard California compaction test of the soil from the six test locations. The average of the six results must exceed the required percent compaction and no more than one-third of the individual results may be lower than the required compaction level. The following is an excerpt from California Test Method No. 321-D:

E. Number and Location of Nuclear Tests

1. The area concept will be used with this test. That is, the engineer will determine from a series of density tests whether to accept or reject a designated area. The engineer shall determine the area by inspection, based on uniformity of factors affecting compaction. Insofar as possible, the area designated shall be homogeneous for both character of material and conditions of production and compaction. Perform six or more nuclear density tests in each area.
2. In order to assure that the material throughout the area is effectively represented by the testing, disperse the six or more individual test sites as impartially as possible over the area. Accomplish this by dividing the area into two or more sub-areas of approximately equal size and by selecting two or more test sites within each sub-area. Each sub-area must contain an equal number of tests. The exact location of each in-place density test site shall be of a random nature. Determine the density and moisture of the material by the nuclear tests as described in part D above.
3. If the designated test area, described in E-1 above, is of limited size (e.g., structure backfill, etc.), then division of the area into sub-areas is not required and three or more test sites shall be selected at random and tested for density and moisture.
4. For all treated and untreated soils and aggregates, except Classes A and B Cement Treated Bases, obtain equal representative portions of material from each nuclear test site, within the area, and thoroughly mix together to form a composite sample for the laboratory maximum density test representing the area. Determine the test maximum density in accordance with Test Method No. Calif. 216, Part II, Method A or Method B.
5. In many cases, after a substantial number of laboratory density tests have been performed on a particular material, it is permissible to establish a "common" maximum density value in lieu of E-4 above. A check of the maximum density value, by performance of the laboratory tests, must be made, for at least every 5 days of testing, on the various soil types used and each time there is a materials change.
6. Care must be taken that the same soil type exists over the given area. This is so that the one maximum density test is consistent with the in-place tests.
7. Using the maximum density test value, calculate the percent relative compaction for each in-place test. The average of all of these relative compaction values must be at or above the specified minimum compaction value for acceptance of the compaction in the area. In addition, at least two-thirds of the relative compaction values from the individual in-place tests must be at or above the specified minimum compaction value for acceptance of the area.
8. When in the opinion of the resident engineer a situation exists relating to the availability of nuclear testing equipment, which would delay the determination of in-place densities, then Test Method No. Calif. 216 may be used.
9. For Classes A and B Cement Treated Bases, representative samples for test maximum density shall be obtained and tested according to Test Method No. Calif. 312.

This specification currently is being revised to include statistical treatment of the compaction test results from several areas within an embankment.

OTHER CONTROL PROCEDURES

Some of the agencies that are relaxing or eliminating moisture requirements are supplementing density requirements with a requirement that the embankment remain stable under construction equipment. Control of this requirement currently is qualitative and commonly relies heavily on the judgment of the inspector. It would be desirable to develop quantitative control procedures for evaluating the stability under equipment. One agency has suggested that as a guide, "As much as six (6) inches of rutting could be tolerated under the modern, large capacity, heavily loaded equipment." Several engineers have suggested that some sort of towed "deflectometer" would be useful, but currently such equipment is not being used.

Proof rolling and test sections are rarely used in conjunction with embankment construction. Proof rolling more commonly is used on subgrades or in cuts. A few engineers are considering embankment compaction control by means of proof rolling after every 4 or 5 ft of increase in embankment height. However, it would be necessary to quantify the permissible deflection under the roller. Proof rolling is most appropriate for soils compacted wet of optimum. For dry soils, stability under the roller is no assurance of adequate density or stability under wet climatic conditions.

In some types of construction, test sections sometimes are used to establish satisfactory construction procedures, which are then used for quality control. Such factors as moisture, lift thickness, and number of coverages are established for a specific compactor, and density control is not used. This type of control is most effective when relatively large quantities of uniform material are being placed. Thus, it is more commonly used in control of base course construction than for highway embankments. However, the initial lifts of most embankments can be considered as trial sections through which the contractor establishes appropriate procedures. In these instances, density requirements are maintained as the basis for quality control.

As noted in Chapter Five, some agencies have alternate specifications that provide for compaction without density control. Frequently, this is called "ordinary compaction"; it commonly is controlled visually by the inspector. In many agencies, compaction without density control is used for secondary and other nonfederal-aid projects. There is evidence that good embankments can be produced without density tests. In fact, many competent soil engineers believe that observation of construction procedures is more important than density testing. However, observational quality control places heavy responsibility on the judgment of the inspector; hence, the quality of inspection personnel becomes very important. Also, observational quality control is most successful when experienced local contractors are placing soils with which they are familiar.

Recently one agency announced that density control would be eliminated on all future embankment construction because of ". . . inherent limitations of density testing as an effective control, staffing problems in providing inspectors to both observe and test embankment construction, undue emphasis on the frequency of density testing, both for control and record purposes, and other considerations." To provide a record of compaction, a daily entry in a diary will document the soil type, the amount and type of hauling, grading and compaction equipment, the general moisture conditions, the number of coverages of compaction equipment, the degree of rutting or displacement under equipment, and any unusual conditions encountered. A limited number of density tests currently are also made to document the end product.

CHAPTER SEVEN

MISCELLANEOUS TOPICS

The major aspects of the design and construction of highway embankments are presented in Chapters Two through Six. However, several materials problems that are of interest in limited regions of the U.S. have not been presented in the general discussions. Also, there remain several additional topics that either directly or indirectly affect embankment construction. These include environmental protection, construction of buried structures, field instrumentation, and the need for effective intra-agency communications.

SPECIAL MATERIALS PROBLEMS

Expansive Clays

Expansive clays are found in many parts of the United States, particularly in the Great Plains east of the Rockies and in the Southwest. Frequently their occurrence coincides with arid or semi-arid climates or regions with dramatic seasonal variations in rainfall. Because the behavior of expansive clays is primarily related to moisture changes, the primary treatments involve establishing and maintaining an equilibrium moisture condition.

The major problems with expansive soils occur in cut sections rather than in fills. In cuts, relatively unweathered material is exposed and tends to expand for two reasons. First, the soil is unloaded by the removal of the overburden soil. Second, much greater moisture variations occur at the exposed surface than at some depth beneath the surface. To reduce volume changes in cut sections, one practice is to undercut 3 to 6 ft of expansive clays and to backfill with select materials. In some instances, the expansive soil may be replaced under carefully controlled moisture and compaction conditions. Also, asphaltic membranes or chemical stabilization procedures are used at the surface to provide moisture barriers.

Expansive soils cause few problems as foundations for embankments: the weight of the fill counteracts the tendency for swelling. Also, the foundation soil no longer is directly exposed to the atmosphere; hence, it is subjected to less severe moisture variations. The higher the fill, the less likely there is of a problem due to an expansive foundation soil. For extremely low embankments, however, undercutting procedures similar to those described for use in cut sections may be required.

When expansive soils are used as embankment materials, they frequently are placed in the bottom of deep fills where they will be confined by the weight of the overlying fill. Alternately, expansive soils can be placed along the side slopes of embankments where expansion will not be detrimental to the pavement section. In some instances, highly expansive clays must be wasted. Less expansive soils sometimes can be placed in the upper portions of the embankment, but frequently select fill is required in the upper 3 ft. Lime treatment and/or asphaltic membranes may be applied to the upper surface to further inhibit moisture changes in the subgrade region. In addition, special drainage provisions and moisture barriers sometimes are used on exposed slopes as a means of maintaining moisture equilibrium in expansive soils. These provisions are perhaps more commonly used for cut slopes than for embankment slopes. However, special curbing sometimes is employed on embankments to reduce surface runoff on slopes.

Special provisions must be used to control the placement of fills containing expansive soils. Usually, the contractor is directed to place expansive soils at moisture contents 2 or 3 percent above optimum (AASHO T-99) because wet-side compaction minimizes the tendency for swelling. Because of the low permeability of many expansive soils, the procedures for adding moisture must be carefully controlled. Prewetting is commonly employed. In addition, discs, scrapers, graders, or other equipment may be required to break up the excavated soil so that more uniform moisture distribution can be achieved. In some instances, it is necessary to rip and prewater excavation material prior to placement of the fill.

The type and size of compaction and hauling equipment also should be controlled for placement of expansive soils. Heavy equipment tends to bog down at high placement water contents. Furthermore, excessive compactive efforts can produce overcompaction, and small increases in density are accompanied by significant reduction in strength. Because of the need for careful construction procedures and control, experienced contractors and inspection personnel are necessary on projects involving expansive soils.

In extremely arid regions, adequate sources of water may be unavailable or very expensive, and it may not be feasible to place expansive soils at moisture contents wet of optimum. Under such circumstances, expansive soils may be compacted dry of optimum; however, some type of chemical stabilization usually will be required. Dry-side compaction actually produces a stronger material than wet-side compaction; hence, it can produce a satisfactory embankment if constant moisture conditions can be maintained. In arid regions, full-depth asphaltic pavements sometimes are used to minimize moisture variations in the subgrade. If necessary, chemical treatment or impervious membranes can be applied to the surface to provide a moisture barrier that will minimize moisture changes. However, protective barriers may be extremely expensive, and the cost of such preventive measures must be balanced against the potential maintenance costs when swelling occurs.

Frozen Soils

Frozen soils cannot be compacted satisfactorily. This is true for granular soils as well as for cohesive soils. Figure 16 shows the effect of temperature on the densities produced by the AASHO T-99 and T-180 compactive efforts. The densities produced by both compactive efforts are reduced significantly at temperatures below freezing. Also, the optimum moisture content decreases and may

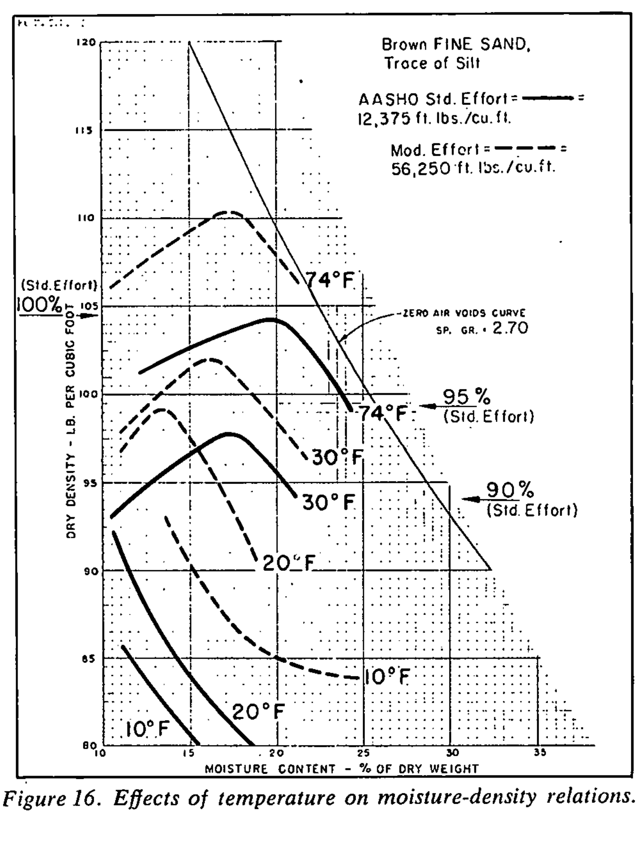

Figure 16. Effects of temperature on moisture-density relations.

disappear at freezing temperatures. Although these data represent laboratory compaction tests, the effectiveness of field compactive efforts will be similarly reduced at freezing temperatures. Furthermore, moisture cannot be added to borrow materials in freezing temperatures. For these reasons, embankment construction commonly is terminated during winter months in cold regions. The primary exception is the placement of clean rock fills, which can be satisfactorily placed at freezing temperatures if little moisture is present.

In some instances, frozen crusts may be produced by overnight freezing temperatures although daytime temperatures remain well above freezing. Under such circumstances, the frozen soil can be stripped away to expose the underlying unfrozen material, which then can be used for embankment construction. Usually a thin layer of frozen material must also be removed from the upper surface of the embankment each morning before additional lifts are compacted.

Permanently frozen ground, or permafrost, creates unique construction problems requiring special construction procedures. However, these procedures are not discussed herein because the occurrence of permafrost within the United States is limited to Alaska.

Foundation Voids

A number of potential problems result from voids within the foundation material beneath a proposed embankment. In some regions the voids are the result of sinkholes or solution cavities in natural rock; in other areas they are the result of old mine tunnels. If the voids occur at relatively shallow depths, they may be dug out and backfilled. Sometimes blasting is used to collapse the overburden into the void, and sometimes sinkholes are backfilled and capped. One of the problems in treating these shallow voids is the identification of their location and their potential for collapse. In some regions, subsidences due to the collapse of mine tunnels at depths of several hundred feet have been reported. Corrective measures for this problem appear to be extremely difficult. Prediction of locations at which subsidence is likely to develop due to the collapse of deep mine tunnels is difficult. Furthermore, the cost of initial corrective action may be greater than the maintenance cost should subsidence develop after construction. In this case, it appears that economy dictates that a calculated risk must be taken. However, at least one agency has a policy of purchasing mineral rights under major highway routes as a method of preventing future mining and decreasing the potential for subsidence.

Mine tailings and abandoned strip or open pit mines create a related foundation problem. These uncompacted materials generally must be removed, but frequently can be recompacted and used as a part of the embankment. When such deposits are too thick for economical complete removal, a partial thickness of the material may be removed and the bottom of the excavation proof-rolled.

Use of Industrial Wastes

A variety of industrial waste products have been proposed for use in highway construction, including mine tailings, blast furnace slag, fly ash, and various incinerator residues. In a few instances, embankments have been constructed of mine tailings, and slag has been used as a lightweight fill. To date, however, use of such materials for embankment construction has been extremely limited. Industrial waste products often are either unavailable in the areas of need or are more costly than the available soils. Hauling such materials to a distant embankment site usually is more costly than using the soils available at the site. Nevertheless, the potential for the use of industrial wastes in highway construction remains. As natural embankment materials become less available and the need for disposal of industrial wastes becomes more critical, the economics of using industrial wastes will undoubtedly become more favorable and an increased use of such products should result. Currently, the engineering properties of many waste products are being studied so that their future use may be more effectively planned.

BURIED STRUCTURES AND BRIDGE APPROACHES

Settlement of bridge approaches and dips in pavements over culverts are two common defects that quickly appear in many new highways. The special procedures and practices currently employed for the construction of bridge approaches have been summarized in *NCHRP Synthesis 2* and are not discussed further herein. However, special procedures for installation of culverts are discussed briefly in the following paragraphs.

Most agencies prefer to construct embankments and then to excavate a trench for the placement of culverts. In this way, the culvert is less likely to be damaged by construction equipment. The trench should be wide enough so that the culvert or pipe can be bedded properly. Where settlement is anticipated, culverts often are installed with an initial camber so that positive drainage will be maintained after some differential settlement has occurred.

Elimination of the pavement dip at culverts depends on closely controlled backfilling procedures. The backfill material should be placed in 6- to 8-in. loose lifts and compacted with power tampers or vibrators. Frequently, select granular backfill is required and for such material lift thicknesses of 12 in. or more may be permitted. The density of the backfill should be equivalent to that of the adjacent embankment so that uniform conditions are developed within the backfill and the embankment. If the backfill is made significantly stiffer than the embankment, an undesirable rise can result in the pavement over the culvert.

Many highway engineers believe that settlements occur over culverts because backfilling specifications are not followed in construction. If inspectors are not present, contractors sometimes will place extremely thick lifts of cohesive soils with little or no compaction, and problems are likely to develop later. Conversely, settlements rarely occur when backfilling specifications are followed. Consequently, even though relatively small quantities of material are involved, inspectors should be instructed to be present during the backfilling operations. Furthermore, contractors must be impressed with the importance of following backfilling specifications.

ECOLOGICAL CONSIDERATIONS

Today highway engineers share the growing concern for protection of the environment. Although construction practices directed toward protecting the environment have always been encouraged, improved practices being introduced will have a significant impact on highway construction. Some of the proposed changes will produce an increase in the cost of certain construction operations. With respect to embankment construction, environmental protection requirements are related to the disposal of waste materials and erosion control.

Waste Disposal

In many urban regions, anti-burning laws have been enacted as air pollution control measures. These regulations have had a significant effect on clearing and grubbing operations. Burning has long been the accepted method for disposing of brush, trees, and stumps, and this method of disposal generally is still used wherever it is permitted. However, in the growing number of areas where burning is prohibited, alternate methods of disposal are being developed. As mentioned in Chapter Three, there is an increasing trend toward leaving stumps in place beneath many embankments. In some instances, brush and stumps are being buried in the side slopes of embankments or placed at the toe of the slopes as erosion protection. If the debris is unsightly, it can be covered by flattening the design slopes. In some areas, brush and stumps, as well as large rocks or boulders, must be hauled from the right-of-way for disposal. Often the disposal of brush and stumps is the contractor's responsibility. If the contractor is not permitted to burn these materials, it is his responsibility to find alternate methods for disposal. Increasingly, waste disposal pits are being purchased just as borrow pits are purchased as materials sources.

Experimentation is under way with chippers that can create a mulch from brush and small trees (up to 20 in.) (Figure 17). Some power companies now are using this equipment for disposing of debris from trimming around power lines in urban areas. With rare exceptions, however, chipping is not being used currently in clearing for highway construction. Current use is limited by size of branch that can be handled in existing equipment. Also, the mulch, which is produced in the initial clearing operation, must be stockpiled until the embankment construction is completed.

Problems also are developing in relation to disposal of large rocks and boulders. Large rocks are being buried in the side slopes of embankments and in the lower regions of very high embankments. In the latter case, the boulders should be separated rather than nested together. In other instances, the contractor is required to haul large rocks from the right-of-way and to dispose of them much as he would dispose of stumps.

Erosion Control

During both excavation and placement of materials the earth is highly susceptible to erosion. Siltation of ponds or streams can result from the runoff from highway construction. When this has occurred in the past, the damage generally has been corrected at the expense of either the highway agency or the contractor. However, it has been suggested that much damage can be prevented by modifying some construction practices. With the encouragement of the Federal Highway Administration, most highway agencies currently are studying, revising, and generally upgrading erosion control practices.

One provision of a recent FHWA memo specifies that "Under no condition shall the amount of surface area of erodible earth material exposed at one time by excavation,

Figure 17. Twenty-inch-capacity chipper.

borrow, or fill within the right-of-way exceed 750,000 sq ft without prior approval by the engineer." This provision presently is undergoing considerable study and interpretation. Strict adherence to this maximum exposed area will result in significant changes in the scheduling of many construction operations.

As the embankment is placed, a variety of temporary erosion control measures are being introduced in various agencies. Mulches, temporary grass, and plastic membranes are being considered for temporary protection of side slopes. Temporary slope drains can be employed to carry runoff from the top of the embankment down the slopes. Both open and pipe drains are being proposed. In addition, various types of berms, dikes, and sedimentation basins are being constructed to control runoff from embankments and to prevent siltation of waterways outside of the construction site. As noted previously, the details of many of these erosion control procedures are currently under development. Although it is generally assumed that the introduction of these procedures will result in increased cost of construction, it is likely that this cost will, in part, be balanced by a reduction in the need to repair eroded spots in the side slopes of embankments.

INTRA-AGENCY COMMUNICATIONS

Successful earthwork construction requires the cooperative efforts of many highway engineers, at both the district and central office levels. Soils and materials engineers conduct the field exploration program, identify problem areas, and recommend corrective measures. Design engineers incorporate these corrective measures into the project design, and construction engineers must ensure the satisfactory implementation of the corrective measures during construction. Although these functions may appear separate and distinct, they must in fact be closely integrated and coordinated. For example, relatively minor design changes in alignment or grade may significantly affect the embankment foundation recommendations and require additional field exploration. Similarly, construction changes may also influence foundation behavior, and copies of construction changes usually are submitted for approval to both design and soils personnel. At the same time, because of the inherent natural variability of soil and groundwater conditions, unanticipated conditions frequently are to be expected during construction. Hence, embankment design should be considered flexible and subject to revision when unanticipated or changed conditions are encountered during construction. As a consequence, there is a need for continual communication among soils, design, and construction engineers from the initial planning stages until the completion of the construction.

Some agencies attribute many of their embankment problems to lack of communication and have taken positive steps to ensure that efficient channels of communication are available. In a recent internal review of its foundation investigation procedures, one agency cited inadequate interdivisional communication as a major problem area. To overcome this, it recommended that (a) the duties of design engineers and soils engineers be revised so as to place upon both a *definite mutual responsibility* for the coordination of the soils investigation and the foundation design, and (b) the duties of the construction engineers and soils engineers should be revised to place upon both a *definite mutual responsibility* for implementation, during construction, of the findings of the soils investigation. Another agency has formed a soils committee, composed of personnel from the materials, design, and construction divisions, to ensure adequate consideration of all aspects of potential soils problems during the planning, design, and construction phases of an embankment project.

excellent idea

FIELD INSTRUMENTATION

Field measurements are made for a number of reasons. Of foremost importance to the current discussion, field measurements can be used to control earthwork construction and to provide warnings of impending failures. In addition, field measurements are used to check and extend existing theories for soil behavior and hence provide a basis for extending the state of the art for design of earth embankments. These research-oriented investigations usually require much more extensive field instrumentation than is required for construction control. Only instrumentation that is commonly used for control of construction is discussed in this chapter.

The most common use of field instrumentation in embankment construction is the control of waiting periods or the rate of construction for embankments over soft foundations. Field measurements also are used to monitor slope movements and to warn of impending slides, particularly on sidehill fills. Thus, field instruments are used to measure horizontal and vertical movements of the embankment or its foundation or to measure porewater pressures within the embankment and its foundation. The instruments used to measure each of these quantities are discussed in the following sections.

Many of the most highly successful field instruments are simple devices, relatively insensitive to climatic conditions and damage by construction equipment. Accuracy often is sacrificed for durability. The instrument installation also should cause a minimum of interference with construction operations. Finally, the instrument should be easily read. Generally, because of the inherent variability of soil materials, instruments are installed at several locations so that measurements can be averaged. Multiple installations also are desirable because a single installation may become inoperative due to construction damage or other reasons.

To provide effective construction control, field observations must be made at regular intervals and the data must be analyzed immediately. The data should be plotted as a function of time so that trends in the measurement are readily observable. These results need to be reviewed by competent engineers who have the authority to alter or temporarily stop construction. Examples can be cited of failures that developed because field measurements that indicated the impending danger were not made available to the supervising engineers soon enough.

Vertical Displacements

Vertical displacement, or subsidence, is the most commonly measured quantity in earthwork construction. Often only the settlement of the upper surface of the embankment is measured. However, when the compression of soft foundations is involved, the settlement of the base of the embankment or top of the foundation material frequently is monitored. Sometimes, particularly for high fills, vertical displacements are measured at various elevations within the embankment.

Four simple devices for measurement of vertical displacements are shown in Figures 18-21. For measurements of the settlement of the surface of an embankment, permanent reference points are established on the surface. The reference point may simply be a wood stake, a bronze screw anchored in concrete, or a settlement rod as shown in Figure 18. Measurements of settlement or heave can be made to an accuracy of 0.01 ft with ordinary surveying instruments. If the settlement of the base of a fill is desired, settlement plates such as shown in Figure 19 can be used. The base plate is installed prior to placement of the fill and a vertical riser pipe is attached to the plate. As the fill is built up, additional sections of pipe are added and the elevation of the top of each new section is determined with respect to the base plate. When the embankment is completed, the displacement of the base can be determined by measuring the movement of the top of the pipe with surveying instruments. Because of the repeated measurements as each pipe section is added and potential disturbance due to construction of the fill, the accuracy of measurements is on the order of ½ to 1 in. During installation, each new section of riser pipe must extend above the surface of the fill. Hence, these pipes interfere with the placement of fill and hand compaction methods must be used within a few feet of the installation. The pipes also must be protected from damage by construction equipment.

Measurements of the vertical displacements at several elevations within a fill can be made with the device shown in Figure 20. Crossarms attached to vertical pipe sleeves are installed at 5- or 10-ft intervals and interconnected with pipe sections. The elevation of each crossarm is monitored by means of a settlement torpedo having a set of pawls that engage the bottom of each pipe sleeve. Measurements are made with a steel tape attached to the torpedo. This pipe installation also interferes with material placement and requires hand compaction of fill in the vicinity of the installation. The need for riser pipes, which interfere with placement of fill, is eliminated through use of a water-level gauge (Fig. 21). The cell is placed in the fill at the elevation for which the displacement is desired and connected to the observation point by means of three tubes. Vertical movements of the cell within the fill are indicated by changes in the water level at the observation station. Although the device is simple in principle, inaccuracies can arise from air bubbles within the system and temperature differentials. Also, care must be taken that the tubing is not ruptured by movement in the fill. If properly installed, water-level gauges can provide measurements to an accuracy of approximately ½ in.

All of the installations shown in Figures 18-21 have been

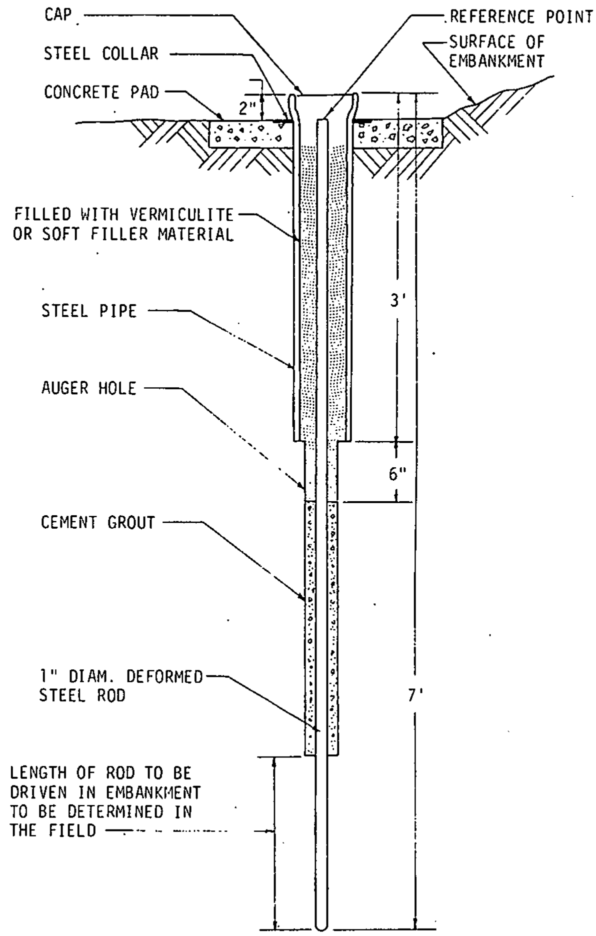

Figure 18. Surface settlement and lateral movement rod.

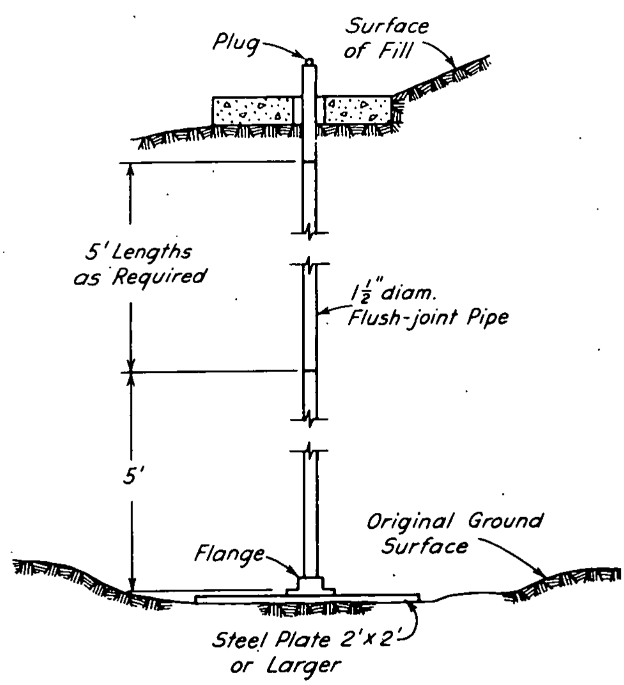

Figure 19. Settlement plate for determining settlement of base of fill. (From Terzaghi and Peck, Soil Mechanics in Engineering Practice, *2nd Ed., 1968, by permission.)*

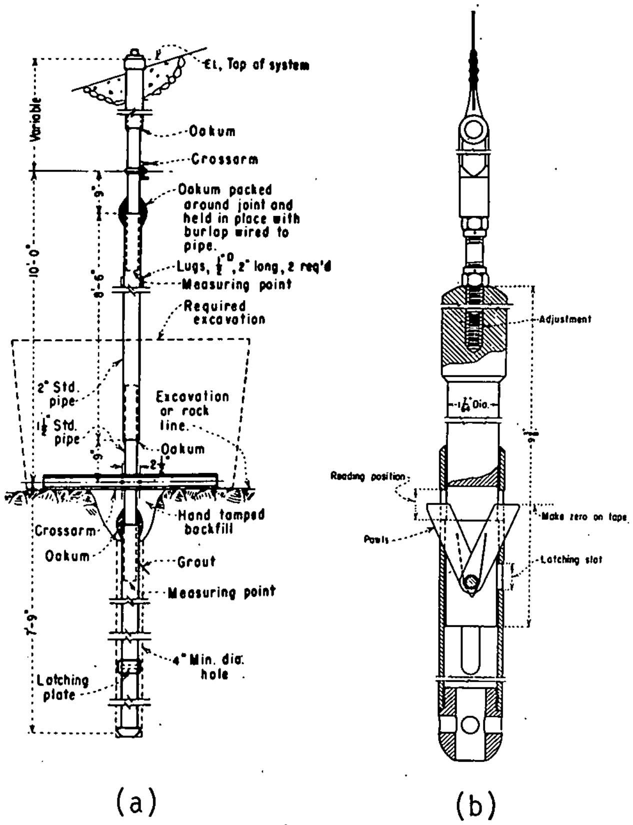

Figure 20. Vertical measurement device showing: (a) pipe installation, (b) measurement torpedo. (From U. S. Bureau of Reclamation Earth Manual, *1963.)*

used by some agencies to monitor rates of settlement. In addition, other instruments, such as mercury gauges and various electronic sensors, including one that uses an induction coil, have been used. However, the degree of success with any specific type of instrument varies greatly and no single type of installation appears to be more satisfactory than the others. Furthermore, it should be recognized that the reliability of all vertical displacement measurements depends on the establishment of permanent bench marks. Fixed bench marks may be difficult to establish and maintain at the sites of some embankment construction.

Lateral Movements

Lateral displacements may be measured to indicate impending instability of an embankment or its foundation. Movements of the surface of the embankment and its slopes can be monitored very simply with surveying instruments and the same reference points used to monitor vertical displacements. Creep within 3 to 5 ft of the surface of a slope can be monitored by observing the movement of pipe sections driven into the slope. However, the measurement of horizontal movements at greater depth requires more elaborate equipment. Usually vertical sections of flexible tubing are installed, and the deflection of the tubing is measured by lowering a pendulum-type device inside the tube. Perhaps

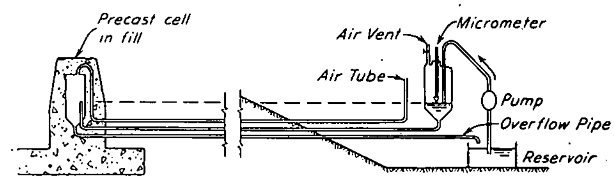

Figure 21. Water-level gauge for measuring settlement at point inside a fill—after Mallet and Pacquant, 1951. (From Terzaghi and Peck, Soil Mechanics in Engineering Practice, *2nd Ed., 1968, by permission.)*

the most commonly used of such devices is the Wilson slope indicator (Fig. 22). In this instrument, the orientation of the pendulum is controlled by a grooved casing and the tilt of the pendulum is measured by means of a Wheatstone bridge. However, the Wilson slope indicator and other similar types of inclinometers are relatively expensive installations. Hence, their use is relatively limited and often restricted to locations where movements previously have been observed.

Pore Pressure Measurements

Pore pressure measurements are used as an indication of the rate of consolidation of soft foundation materials, most commonly in conjunction with stage construction. In addition, pore pressure data can be used to obtain reasonably reliable slope stability analyses. The rapid buildup of pore pressures is frequently an indication that fill is being placed too rapidly and a stability failure may be impending. Similarly, if significant pore pressures are not developing, the rate of construction can be safely increased. Thus, pore pressure measurements complement vertical displacement measurements on projects involving consolidation of soft foundation materials.

Reliable pore pressure measurements are much more difficult to obtain than displacement measurements. In relatively pervious material, a simple observation well with a well point may be satisfactory. In this case, the pore pressure is determined by measuring the water level in the well. However, if the soil is relatively impermeable, measurements with simple open pipes become unreliable because of the quantities of water required to fill the pipe. This condition, which exists for most consolidation problems, requires the use of more elaborate piezometers. Some of the devices used are shown in Figures 23 through 27. Figures 23 and 24 show open-standpipe piezometers in which the quantity of water involved has been reduced significantly by using a small-diameter tube attached to a porous tube tip. The water level in the plastic tubing is sensed electrically. Figure 25 shows two types of tips used in closed-system hydraulic piezometers. In these gauges, water is circulated through the system to remove air and then the lines are connected to a Bourdon-type pressure gauge. An effective pneumatic piezometer is shown in Figure 26. In this device, air pressure is used to balance the pore pressure applied to a diaphragm in the transducer. The required air pressure is applied through a closed loop

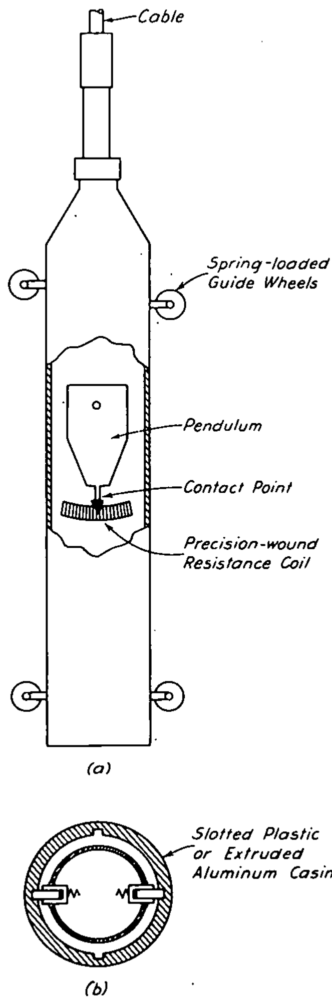

Figure 22. Wilson slope indicator: (a) cutaway view of instrument; (b) cross section showing instrument in slotted casing. (From Terzaghi and Peck, Soil Mechanics in Engineering Practice, *2nd Ed., 1968, by permission.)*

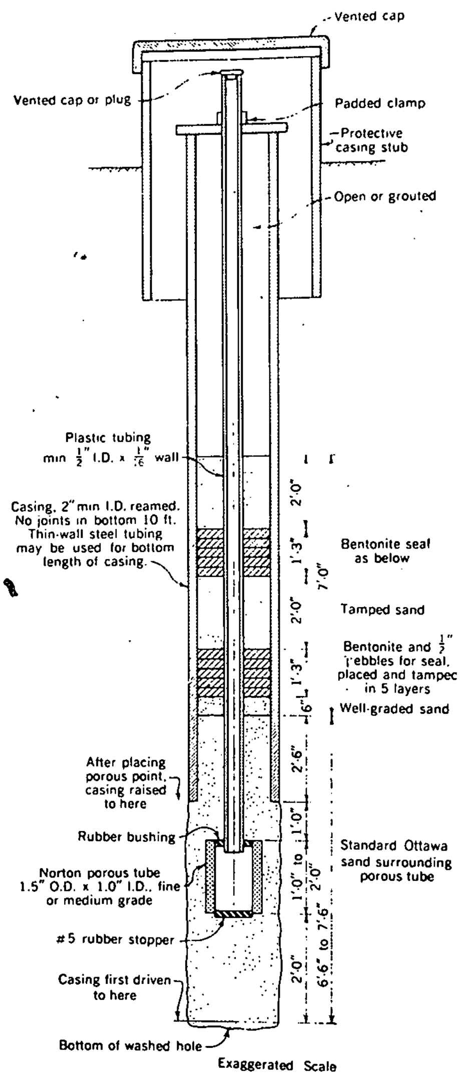

Figure 23. Casagrande porous-tube piezometer—adapted from Casagrande, 1958. (From Foundation Engineering *by Leonards. Copyright 1962 by McGraw-Hill Book Company. Used with permission.)*

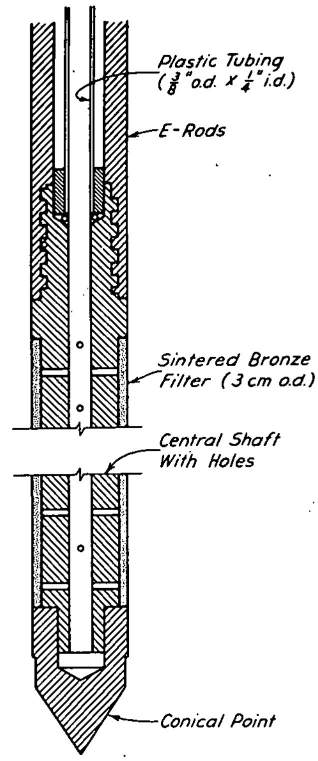

Figure 24. Open piezometer of Geonor type. (From Terzaghi and Peck, Soil Mechanics in Engineering Practice, *2nd Ed., 1968, by permission.)*

and monitored by pressure gauge at the surface. Figure 27 shows an electrically operated gauge. The pore pressure acting on the porous stone deflects the diaphragm. The deflection of the diaphragm is measured by means of a strain gauge that has been calibrated to indicate the pore pressure. These gauges may become unreliable or inoperative if the strain gauge becomes unstable or water leaks into the electrical chamber. This is likely to occur with long-term installations.

All piezometers require careful installation procedures to ensure that they are properly sealed and undamaged. Thus, all installations and measurements should be made under the supervision of qualified personnel. Furthermore, because the reliability of measurements frequently must be questioned, the interpretation of measurements must also be performed by properly trained personnel.

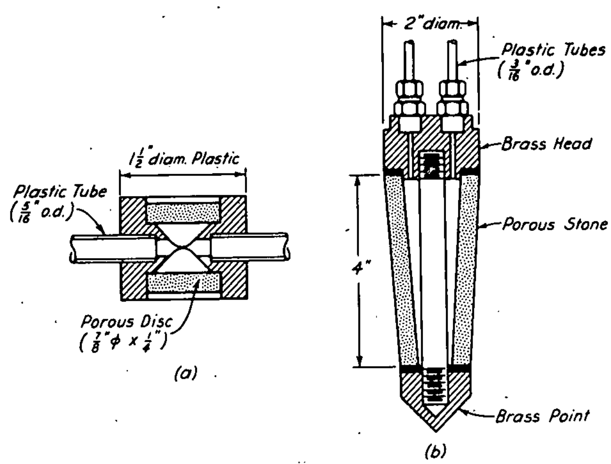

Figure 25. Closed-system hydraulic piezometers. (From Terzaghi and Peck, Soil Mechanics in Engineering Practice, *2nd Ed., 1968, by permission.)*

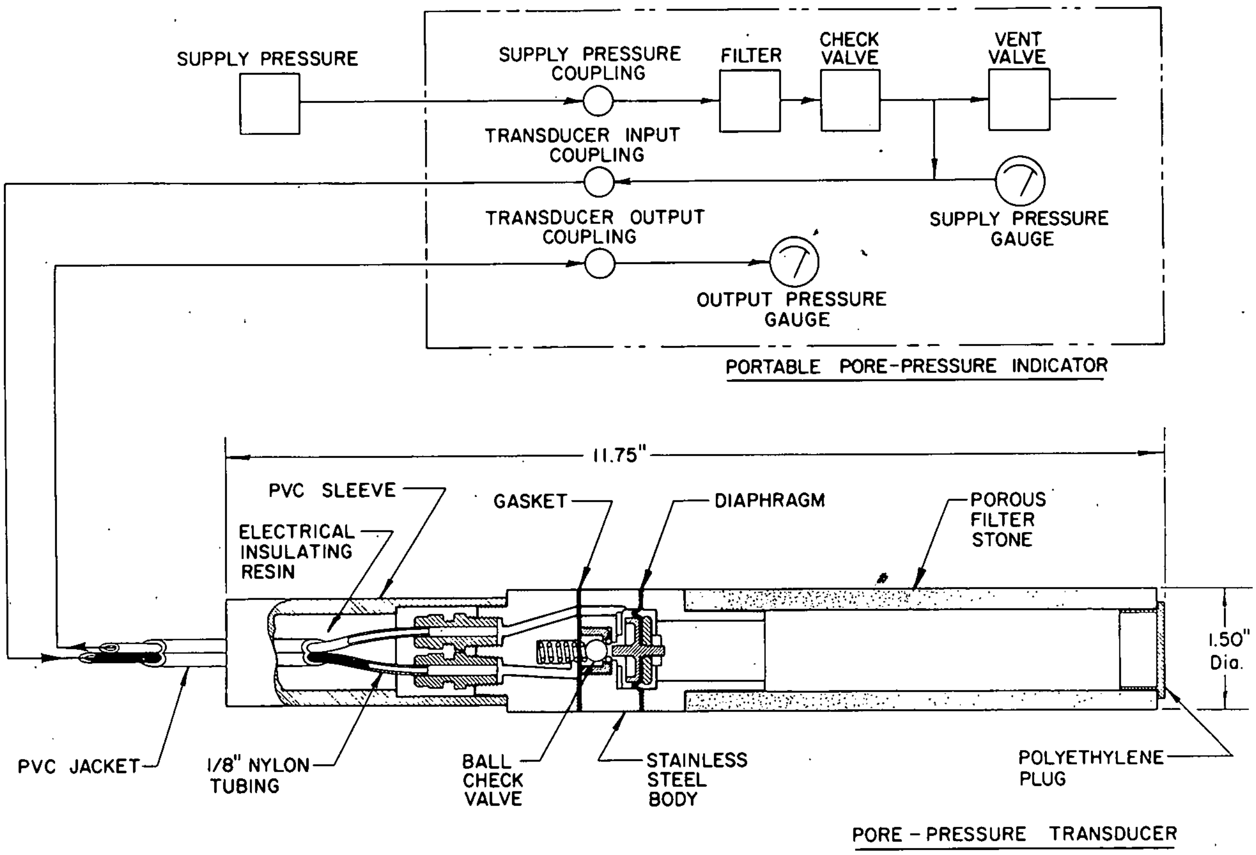

Figure 26. Pore-pressure system of pneumatic-diaphragm type. (Slope Indicator Co., Seattle.)

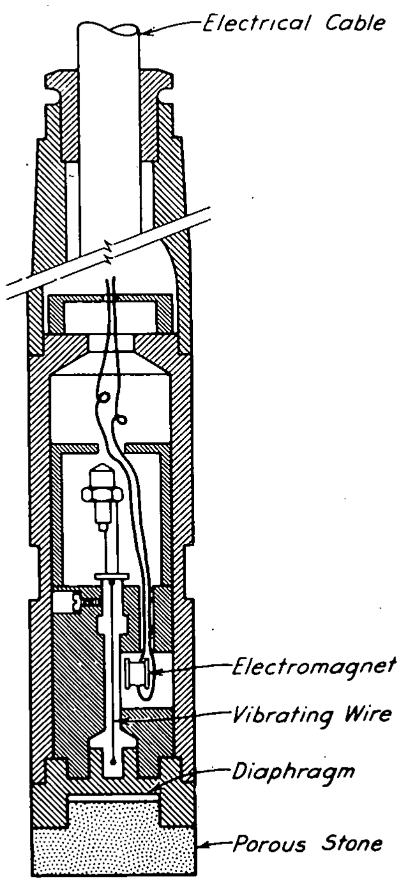

Figure 27. Electrically operated pore-pressure gauge of vibrating-wire type. (From Terzaghi and Peck, Soil Mechanics in Engineering Practice, *2nd Ed., 1968, by permission.)*

CHAPTER EIGHT

CURRENT AND FUTURE RESEARCH

CURRENT RESEARCH

A number of agencies are currently engaged in research related to embankment construction. Some of these are identified in Table 3. It is readily acknowledged that this is not a comprehensive list and that productive research is probably in initial or continuing stages in many highway agencies, universities, and other research-oriented agencies. Omission of these was not intended. It is hoped that they will make themselves and their research known, whether they be directly or indirectly related to construction of embankments.

FUTURE RESEARCH

Conversations and interviews with design, construction, and soils engineers indicate that there are a number of areas concerning embankment construction where additional research may be beneficial. These research needs are:

- Development of rapid quality control procedures for placement of fill, perhaps based on other than density criteria. Suggestions include:
  - (a) Quantitative method for measuring stability under construction equipment.
  - (b) Proof rolling at periodic intervals of embankment height.
  - (c) Expendable strain gauges to measure compression under equipment.
- Development of concepts and procedures for measuring quality of rock fills.
- Factors affecting the effective use of vibratory compaction.

TABLE 3

SUMMARY OF KNOWN RESEARCH ACTIVITIES RELATED TO EMBANKMENT CONSTRUCTION[a]

RESEARCH PROJECT TITLE	RESEARCH AGENCY	HRIP NO.[b]
Wet Fill Materials in Embankment Construction	Road Research Laboratory (UK)	33 062707
Development of Nuclear Methods for Quality Control of Highway Embankment Construction	Michigan Department of State Highways, Research Laboratory Division	34 001150
A Study of Active Clays as Related to Highway Design	Mississippi State Highway Department, Mississippi State University	62 019040
Prediction of Settlements and Pore Pressures Beneath a Highway Embankment Near Sydenham, Ontario	Queens University, Kingston (Canada)	62 050249
Settlement and Stability of Earthworks	Road Research Laboratory (UK)	62 060282
Stability of Embankments on the Texcoco Lake Area of the Mexico Valley	Projects & Laboratories Div., PWM (Mexico), Geotechnic. Dept.	62 060913
Materials for Earth Embankments Together with Construction Methods	Public Works Res. Inst., CM (Japan), Chiba Branch	62 064864
Study of Long-Term Deformation of Compacted Cohesive Soil Embankments	Purdue University	62 085143
Relationship Between the Height of an Embankment, Pore-Water Pressure and Stability	Materials Research Lab. (Turkey), Soil Mechanics Section	63 065007
Investigation of Landslides on Highways	Kentucky Department of Highways	63 201259

[a] As of July 1971. [b] Acquisition number assigned by the Highway Research Information Service of the Highway Research Board; HRIP = publication entitled *Highway Research in Progress* (current issue).

- Development of better or less expensive methods for controlling moisture in expansive soils.
- Cost-benefit studies of the compaction of side slopes.
- Corrective measures for eliminating creep and/or sliding along sidehills.
- Behavior of materials in high embankments, including strength, compression, and creep.
- New procedures for disposal of brush, debris, and stumps.
- Development of settlement tolerances for pavement systems; i.e., how much permanent settlement (total and differential) can be tolerated by the pavement system:
  (a) At structure-pavement interface.
  (b) Away from structures.
- The development of suitable control procedures for compacted materials with high rock content. Conventional impact compaction tests and field density tests are not designed for use with large particles. New principles for control of rocky materials are required.
- The study of the influence of compaction criteria on pavement design procedures. The behavioral requirements of compacted materials are related to the pavement design procedures, which determine the stresses applied to the compacted layers. Optimization techniques can be used to indicate the influence of the quality of compacted materials on the over-all cost of a pavement section.
- Development of rational procedures for correlating compaction requirements and embankment design methods, particularly for high embankments. The compaction requirements for embankments should be developed on the basis of the engineering properties used in embankment design.
- Determination of the effective depth of vibratory compaction as a function of material type and equipment parameters. Additional field data regarding vibratory compaction are required to enable more effective and economical use of vibratory compactors.
- Investigation of the variability of soil properties in (a) natural deposits, (b) newly compacted earthwork, and (c) existing satisfactory earth structures. Information regarding the normal variability of soil properties, how to determine it, and what effect it has, is essential to the establishment of compaction criteria based on statistical concepts.

APPENDIX

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Under the terms of its Congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the Federal Government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the Government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the Government.

THE NATIONAL ACADEMY OF ENGINEERING was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its Act of Incorporation, adopted Articles of Organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the Federal Government, upon request, on any subject of science or technology.

THE NATIONAL RESEARCH COUNCIL was organized as an agency of the National Academy of Sciences in 1916, at the request of President Wilson, to enable the broad community of U. S. scientists and engineers to associate their efforts with the limited membership of the Academy in service to science and the nation. Its members, who receive their appointments from the President of the National Academy of Sciences, are drawn from academic, industrial and government organizations throughout the country. The National Research Council serves both Academies in the discharge of their responsibilities.

Supported by private and public contributions, grants, and contracts, and voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

THE DIVISION OF ENGINEERING is one of the eight major Divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

THE HIGHWAY RESEARCH BOARD, organized November 11, 1920, as an agency of the Division of Engineering, is a cooperative organization of the highway technologists of America operating under the auspices of the National Research Council and with the support of the several highway departments, the Federal Highway Administration, and many other organizations interested in the development of transportation. The purpose of the Board is to advance knowledge concerning the nature and performance of transportation systems, through the stimulation of research and dissemination of information derived therefrom.

HIGHWAY RESEARCH BOARD
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