State of the Art Report 8

GUIDE TO EARTHWORK CONSTRUCTION

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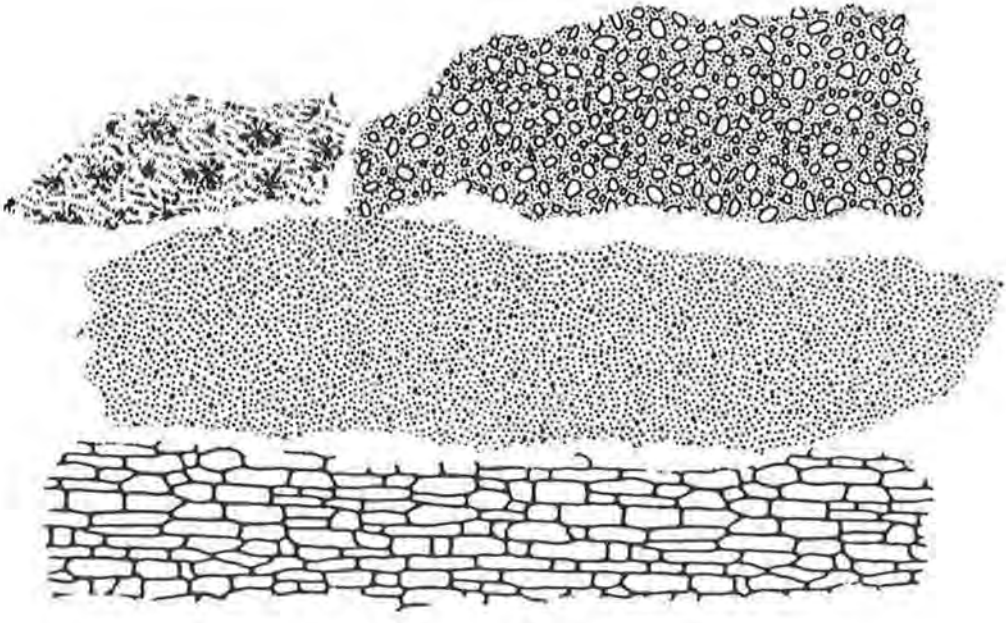

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Preface

IN RECENT YEARS, because of retirements and promotions, state transportation agencies have been gradually losing much of their earthwork experience gained during the construction of the Interstate highway system. Recognizing this problem, the TRB Committee on Transportation Earthworks, in cooperation with the Committees on Soils and Rock Instrumentation and on Foundations of Bridges and Other Structures, prepared this *Guide to Earthwork Construction* for field construction personnel at the project engineer and technician level, rather than for designers. The committee believed that it was better to concentrate on earthwork construction than design because ample information on the design of embankments, foundations, and slopes is available in reports published by the Transportation Research Board, National Cooperative Highway Research Program, and the Federal Highway Administration, as well as in other government publications such as the U. S. Bureau of Roads' *Earth Manual* and soil mechanics textbooks. It is hoped that this guide will provide useful information for construction personnel responsible for all aspects of embankment and earthwork construction.

This guide is the brainchild of William G. Weber, Jr., formerly with the California, Pennsylvania, and Washington State Departments of Transportation, and a long-time member of the TRB Committee on Transportation Earthworks. Weber believed that with the Interstate highway system essentially completed, embankment construction is now evolving toward rehabilitation and upgrading of facilities such as adding more lanes, widening embankments, and so forth. Such reconstruction would probably alter drainage patterns and systems. Retaining walls and abutments, along with recent developments in embankment stabilization and reinforcement techniques, would also influence embankment construction.

Because many embankments today are instrumented to obtain performance information and to check design assumptions, field personnel need to be aware of instrumentation in order to avoid damage and enhance the validity of measurements. Finally, environmental factors such as hazardous and other waste areas, swelling clays, and winter construction also need to be addressed. Unfortunately, William G. Weber, Jr., died August 2, 1989, just as the guide was being prepared for publication. Conse-

quently, he never had the opportunity to review the chapter he wrote—Chapter 2, History of Embankment Construction—or to see the result of his original idea.

The principal author or authors are given at the beginning of each chapter. Robert D. Holtz was responsible for the technical editing of the guide, and members of the three TRB committees mentioned earlier provided technical review of individual chapters. Thomas A. Belatty, Jerome A. DiMaggio, Richard E. Landau, R. G. Lukas, Victor A. Modeer, Jr., and Walter C. Waidelich critically reviewed the final draft and made many helpful suggestions. The efforts of all of the authors and reviewers are sincerely appreciated.

Robert D. Holtz

Contents

1

Introduction

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This *Guide to Earthwork Construction* has been prepared to provide construction engineers and technicians with information on all aspects of earthwork construction. Although it is not intended to be a design manual, it does contain considerable background on the design concepts that are necessary for good earthwork construction. Most of the sections contain information on specific field problems, and a number of references are included to provide construction engineers with additional detailed information.

As used in this guide, earthwork consists of roadway excavations and embankments and all associated items of work such as foundations, drainage, stabilization and reinforcement, environmental factors, and instrumentation. Earthwork encompasses all types of materials excavated and placed in highway embankments, including soil, rock, intermediate materials, and other natural and man-made materials including wastes. Also included in the broad category of earthwork are clearing and grubbing, scalping, removal of existing structures and obstructions, channel excavations, preparation of foundations and embankments, disposal of excavated material, borrow excavation, preparation of subgrade, proof rolling, and placement of granular subbase and base courses, structure backfills, and instrumentation.

Uniformity of earthwork throughout the project is necessary for adequate stability and satisfactory long-term performance. To obtain an acceptable product, the owner or agency is obligated to provide adequate design and specifications as well as competent construction inspectors. The contractor must provide proper equipment and construction procedures for all materials and conditions encountered during the project. Construction engineers and inspectors must work with the contractor to establish the most appropriate construction methods and procedures, consistent with the specifications, in order to obtain an acceptable product at the least cost. As noted by Johnson (1964), a key factor in achieving these goals is competent testing and monitoring during construction, a

primary responsibility of the construction inspection team.

The ultimate goal of earthwork construction is to make the final product or facility adhere as closely as possible to the designer's intent. The key to achieving this goal is the knowledge, experience, and understanding of all aspects of earthwork construction that field engineers and inspectors bring to the project. In order for a project to be successful, it is essential that field personnel recognize situations in which field conditions differ in important ways from the design assumptions, and in such situations, react quickly and effectively. It is hoped that this guide will facilitate recognition of these situations and encourage rapid and effective reaction by field personnel.

GENERAL REFERENCES AND BACKGROUND

A good basic reference on earthwork construction is Section 14 in Woods's *Highway Engineering Handbook* (Gregg 1960). Johnson and Sallberg (1960) present a detailed discussion of the factors that influence field compaction, and later they discuss the factors that influence compaction test results (1962). The Proceedings of the ASTM (1964) Symposium on Compaction of Soils contains much useful information, as does *NCHRP Synthesis of Highway Practice 8: Construction of Embankments* (Wahls 1971) and the *Earth Manual* (USBR 1974, currently being rewritten). Hilf (1975) has an excellent treatment of compacted fill. Monahan's (1986) book *Construction on Compacted Fills* contains information particularly useful for field engineers and inspectors. *Earthworks* (Horner 1981), an Institution of Civil Engineers (ICE) construction guide, discusses earth-moving and compaction practice, in addition to equipment commonly used in the United Kingdom. Related Federal Highway Administration publications include those by Konya and Walter (1985) on blasting, Christopher and Holtz (1985) on geotextiles, and Golder Associates (1989) on rock slopes.

Textbooks such as those written by Taylor (1948), Terzaghi and Peck (1967), Sowers and Sowers (1979), Holtz and Kovacs (1981), and Spangler and Handy (1982) also contain useful information on the theory and methods of compaction, earthwork and compaction equipment, and field compaction control and specifications.

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ASTM American Society for Testing and Materials
FHWA Federal Highway Administration
USBR U. S. Bureau of Reclamation

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2

History of Embankment Construction

WILLIAM G. WEBER, JR.*
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Primitive civilizations used soil to construct earth structures and embankments for dwellings, religious worship, burials, canals, roads, and fortifications. Simple trial-and-error procedures led to progress in the use of soil as a construction material, and this knowledge, initially passed on from generation to generation by word of mouth, later became part of the written record. One of the earliest of these is in the *Dschou-Li,* a book on the customs of the Dschou Dynasty, written about 3000 B.C. in China. It contains, among other things, instructions for the construction of roads and bridges (Speck 1950). According to AASHTO (1950), the oldest road in the world, "The Royal Road" across southwest Asia and Asia Minor, was used by wheeled wagons in about 3000 B.C. One of the oldest technical records of construction using soils is found in the Ten Books on Architecture compiled during the first century B.C. by the Roman engineer Vitruvius (Granger 1934).

In the mid-1600s, France undertook an extensive public works program, including construction of highways, canals, and fortification systems for the country's borders. The first engineering school in Europe, Ecole des Ponts et Chaussées, was established in Paris in 1747, and here engineers were educated in the principles of physics, mathematics, and mechanics as known at the time for the construction of highways, bridges, harbors, canals, and retaining structures. The founding of this engineering school had an important influence on the scientific development of civil engineering.

In the early 1800s in England, Thomas Telford and John MacAdam were constructing roads based partly on scientific principles. One principle was to raise the foundation above the surrounding ground so that water would not soften the subgrade. The embankments were seldom

* Deceased.

more than 10 ft high with maximum side slopes of about 3H:1V. Most of the roads followed the contours in hilly country, resulting in partial fill and cut; drainage blankets were common in these situations.

With the construction of railroads beginning in the 1830s, new problems developed because of the flat grades and long radius curves required and the higher loadings imposed on the subgrades. It was soon realized that moisture in the soil played an important role in compaction of fills. When the soil was too wet, construction equipment bogged down and failures occurred. When the soil was too dry, unanticipated settlements would occur during periods of rain.

A simple test was soon devised by construction personnel to gauge the proper moisture content in the soil. The soil was formed by hand into a compact ball; then a person would spit on the soil. If the moisture soaked into the soil, it was too dry; if the moisture balled on the soil, it was too wet. If the moisture slowly soaked into the soil ball, it had the proper moisture content. Although this test was crude and unscientific, it was quite effective as a method of moisture control, and ultimately it led to the development of the field moisture equivalent test.

After several disastrous landslides in Sweden, a Royal Commission of the Swedish State Railways was appointed in 1913 to develop procedures to avoid future disasters. The report of the Commission in 1922 was an important milestone in the understanding of soil properties and geotechnical analyses. Among the most important developments were the well-known and still commonly used Swedish Circular Arc method for determining slope stability, undisturbed soil sampling, and laboratory shear testing. The problems of embankment stability were now on their way to engineering solutions.

In 1925, in his book *Erdbaumechanik*, Terzaghi demonstrated the importance of the water phase in the long-term settlements that occurred under loadings on clay and developed the theory of consolidation. He also presented a new view of soils as materials with widely varying properties.

Expansion of the highway system in the United States during the 1920s and the construction of increasingly higher earth dams demanded improved compaction procedures. The California Division of Highways (Stanton 1928) used the first soil compaction test to determine the optimum moisture and maximum density before construction and relative density during construction. The Bureau of Water Works and Supply of the city of Los Angeles conducted an extensive study of the effects of soil compaction on the shear strength and permeability of compacted soils in earth dams. The results of this study were reported by Proctor (1933). About the same time and apparently independently of Proctor, Kelso (1934–1935) was performing experiments on the soil moisture–unit weight relationships during the construction of Silvan Dam in Australia.

The relationships between density, moisture, strength, compressibility, and other soil properties were fairly well established by the mid-1930s, and these factors were intensely studied for various soil types during the next 30 years.

Construction equipment used in the hauling and placing of earth for embankments has also undergone a similar development. In ancient times, soil was hauled by humans and animals. With the advent of wheeled vehicles, soil was hauled by animal-drawn carts. Animals were used to pull the Fresno scraper, which could load itself, and for compaction. A patent was issued to M. Louis Lemoine of Bordeaux, France, in 1859 for a steamroller. With the development of the gasoline engine in the early 1900s, gasoline-driven trucks, loaded with the use of a steam shovel, came into common use, and in 1906, a patent was issued for a horse-drawn sheepsfoot roller. Compaction was commonly limited to the surface of embankments, because the general feeling was that the fill would settle anyway.

Expansion of the highway system during the 1920s resulted in rapid development of heavier hauling and compaction equipment. During the 1930s, an extensive public works program was undertaken. Tractor-drawn scrapers came into common use, and new and heavier compaction equipment was developed. The requirement that fills be compacted from the ground to the surface became generally accepted by engineers. Embankments as high as 50 ft were common in mountainous areas.

In the 1950s when the Interstate highway system was begun, flat grades and long radius curves, previously encountered only in railroad construction, became standard. Whereas railroads often had used trestles to cross valleys, highways used embankments, and Interstate highway embankments often exceeded 100 ft in height. Zoned embankments, where specified materials were employed in different portions of the embankment, came into common use. As the pressures on culverts increased, their design required greater consideration, and foundation soils often were also of serious concern.

Construction of the Interstate highway system caused many changes for both engineers and contractors. The movement of large quantities of soil resulted in rapid advances in construction equipment. The use of self-propelled, rubber-tired scrapers capable of carrying 20 to 30 yd^3 of soil at high speeds became common, and larger tractors and blades were developed. Although the Interstate system is now essentially complete, many problems remain. Portions of the system are now carrying more traffic than they were ever designed for. Upgrading will require adding more lanes, which will mean widening existing embankments. Also, other means may be required to increase capacity. Many of the roads feeding traffic to or receiving traffic from the Interstate will need to be upgraded

or rebuilt. Safety improvements will also be required, and, of course, never-ending maintenance will present new challenges.

Environmental factors have become of increasing concern to engineers. Previously, potential embankment erosion was considered a maintenance problem; today it must be considered in design. Construction of sound barriers has become common in urban areas. Retaining walls are often required where limited right-of-way exists in reconstruction areas. Frequently, reconstruction will alter drainage patterns and cause other problems. Imaginative solutions are required, although the basic engineering principles will remain substantially the same.

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ABBREVIATION

AASHTO American Association of State Highway and Transportation Officials

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3

Compaction Concepts

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Often in highway engineering, the soils at particular sections along the roadway alignment are less than ideal for embankment construction. It may appear reasonable in such instances to simply relocate the facility. However, considerations other than geotechnical conditions often govern the location of a highway, and the engineer is forced to design for and construct on the site at hand. One possibility may be to adapt the foundation of the structure to the geotechnical conditions existing at the site. A good example of this approach is the use of pile foundations for bridges. Another possibility is to try to stabilize or improve the properties of the soils at the site, and depending on the circumstances, this approach often is the most economical solution to the problem. Stabilization is usually mechanical or chemical, but even thermal and electrical stabilization have occasionally been used or considered.

In this chapter, adapted from Holtz and Kovacs (1981, Chapter 5), the primary concern is with mechanical stabilization or densification, also called compaction. Chemical stabilization involves the mixing or injecting of chemical substances into the soil. Portland cement, lime, asphalt, calcium chloride, sodium chloride, and paper mill wastes are common chemical stabilization agents (Winterkorn 1975; TRB 1987).

Other methods for stabilizing unsuitable foundation soils include dewatering, which is the removal or reduction of excess groundwater pressures, and preloading, in which foundation soils are surcharged with a temporary overload so as to increase the strength and decrease anticipated settlement. Details of these and other methods are described in textbooks on foundation and highway engineering (see also Chapter 6 of this guide).

Compaction and stabilization are very important when soil is used as an engineering material; that is, when the structure itself is constructed of soil. Earth dams and highway embankments are typical examples of earth structures. If soils are dumped or otherwise placed at random in a fill, the result will be an embankment with low stability and high settlement.

Before the 1930s, highway and railroad fills were usually constructed by end-dumping soils from wagons or trucks. There was little attempt to compact or densify the soils, and failures of even moderately high embankments were common. As noted in Chapter 2, earthworks such as dams and levees are almost as old as man, but these structures, for example, in ancient China or India, were constructed by people carrying small baskets of soil and dumping them into the embankment. People walking over the dumped materials compacted and thus strengthened the soils. Even elephants have been used in some countries to compact soils, but research has shown that they are not very effective (Meehan 1967).

COMPACTION

As previously mentioned, compaction is the densification of soils by the application of mechanical energy. It may also involve modification of water content and gradation of the soil. Cohesionless soils are efficiently compacted by vibration. In the field, hand-operated vibrating plates and motorized vibratory rollers of various sizes are quite efficient for compacting shallow deposits of sand and gravel soils. Rubber-tired equipment can also be used efficiently to compact sands. Even large free-falling weights are used to dynamically compact loose granular deposits and fills. Some of these techniques are discussed in Chapters 6 and 9, as well as by Holtz (1989).

Fine-grained and cohesive soils may be compacted in the laboratory by falling weights and hammers, by special "kneading" compactors, and even statically. In the field, common compaction equipment includes hand-operated tampers, sheepsfoot rollers, rubber-tired rollers, vibratory rollers, and other types of heavy compaction equipment. Considerable compaction can also be obtained by proper routing of the hauling equipment over the embankment during construction (see Chapter 4, section on Compaction).

The objective of compaction is to improve the engineering properties of the soil mass; by compaction

- Detrimental settlements can be reduced or prevented;
- Soil strength can be increased and slope stability improved;
- Bearing capacity of pavement subgrades can be improved; and
- Undesirable volume changes, for example, caused by frost action, swelling, and shrinkage, may be controlled.

THEORY OF COMPACTION

The fundamentals of compaction of cohesive soils were developed by R. R. Proctor in the early 1930s. Proctor published a series of articles in *Engineering News-Record* (Proctor 1933) on the principles of compaction, and in his honor, the standard laboratory compaction test that he developed is called the Proctor test.

Proctor noted that compaction is a function of four variables: (a) dry density, ρ_d; (b) water content, w; (c) compactive effort; and (d) soil type. Compactive effort is a measure of the mechanical energy applied to a soil mass. In the field, compactive effort is the number of passes or "coverages" of the roller of a certain type and weight on a given volume of soil. In the laboratory impact compaction test, a hammer is dropped several times on a soil sample in a mold. The mass of the hammer, height of drop, number of drops, number of layers of soil, and the volume of the mold are specified. For example, in the standard Proctor test (also standard AASHTO T 99 and ASTM D 698), the mass of the hammer is 5.5 lb and the height of fall is 1 ft. The soil is placed in three layers into a $1/30$ ft^3 mold, and each layer is tamped 25 times. Compactive effort can be calculated to be 12,375 ft-lbf/ft^3.

The process of compaction for cohesive soils can best be illustrated by the Proctor test. Several samples of the same soil, but at different water contents, are compacted according to the standard Proctor test specifications given earlier. The total or wet density and the actual water content of each compacted sample are measured. Then the dry density for each sample can be calculated from

$$\text{Wet density, } \rho = \frac{\text{total mass or weight, } M_t}{\text{total volume, } V_t} \tag{3-1}$$

$$\text{Dry density, } \rho_d = \frac{\text{wet density, } \rho}{1 + \text{water content, } w} \tag{3-2}$$

When the dry densities of each sample are determined and plotted versus the water contents for each sample, then a curve called a compaction curve for standard Proctor compaction is obtained (Figure 3-1, Curve A). Each data point on the curve represents a single compaction test, and usually four or five individual tests are required to completely determine the compaction curve. This curve is unique for a given soil type, method of compaction, and (constant) compactive effort. The peak point of the curve determines the maximum dry density $\rho_{d\,max}$ at a water content

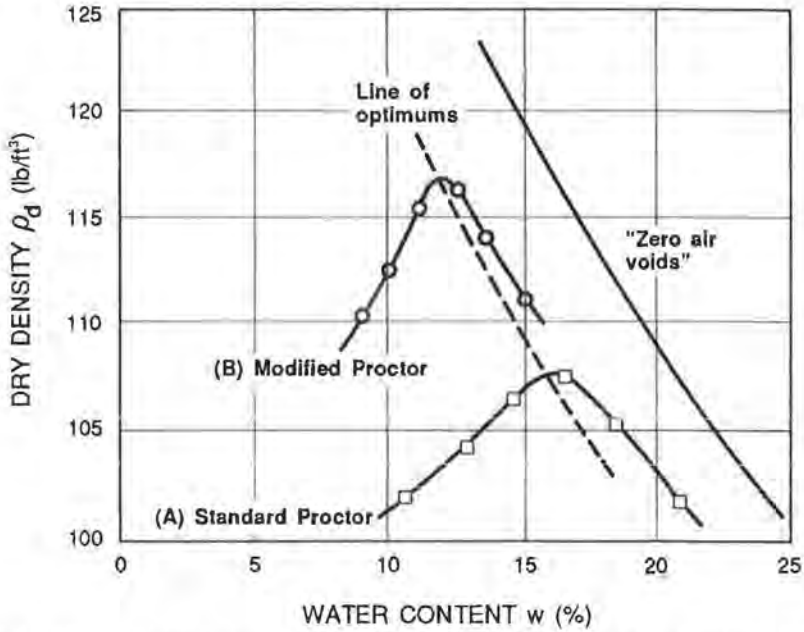

FIGURE 3-1 Standard and modified compaction curves for Crosby B till (Holtz and Kovacs 1981). (*Reprinted with permission from Prentice Hall, Inc.*)

known as the optimum water content w_{opt} [also called the optimum moisture content (OMC)]. Note that the maximum dry density is only a maximum for a specific compactive effort and method of compaction. This does not necessarily reflect the maximum dry density that can be obtained in the field.

Typical values of maximum dry density for inorganic cohesive soils are about 100 to 125 lb/ft^3 with the maximum range from about 80 to 150 lb/ft^3. Typical optimum water contents are between 10 and 20 percent, with an outside range of about 5 to 40 percent. Note that the compaction curve, even at high water contents, never actually reaches the zero air voids curve. This is true even for higher compactive efforts, for example, Curve B of Figure 3-1. Curve B is the compaction curve obtained by the modified Proctor compaction test (AASHTO T 180 and ASTM D 1557). This test uses a heavier hammer (10 lbf), a greater height of fall (1.5 ft), and five layers tamped 25 times into a standard Proctor mold. The compactive effort is 56,250 ft-lbf/ft^3.

The modified test was developed during World War II by the U. S. Army Corps of Engineers to better represent the compaction required for airfields to support heavy aircraft. The point is that increasing the com-

pactive effort tends to increase the maximum dry density, as expected, but it also decreases the optimum water content. A line drawn through the peak points of several compaction curves for the same soil at different compactive efforts will be almost parallel to the zero air voids curve.

Typical compaction curves for different types of soils are shown in Figure 3-2. Note how sands that are well graded (SW soils, top curve) have a higher dry density than most uniform soils (SP soils, bottom curve). For clay soils, the maximum dry density tends to decrease as plasticity increases.

Why do we get compaction curves such as those shown in Figures 3-1 and 3-2? Starting at low water contents, as the water content increases,

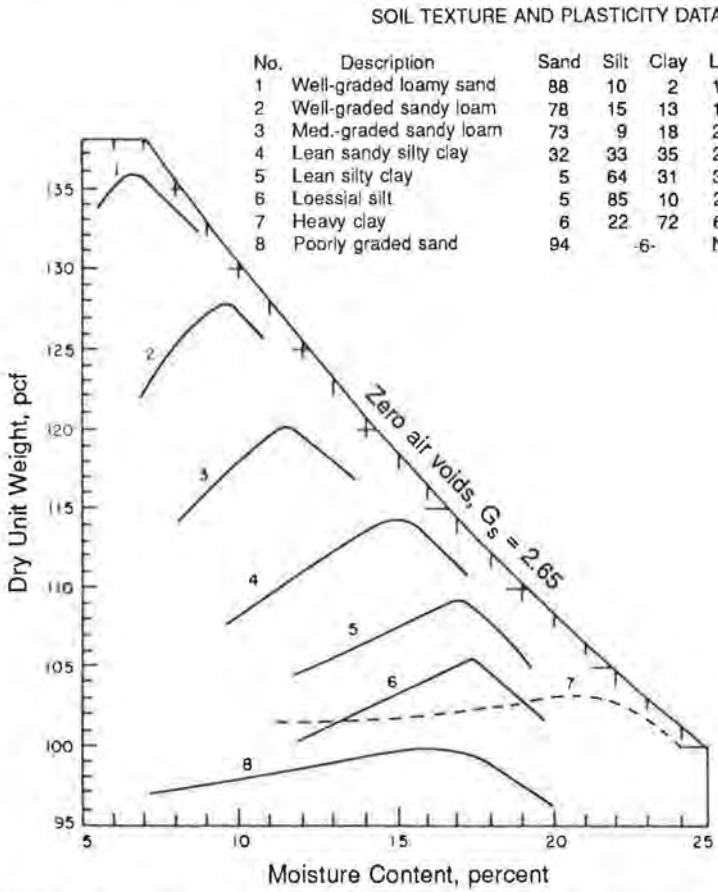

FIGURE 3-2 Water content–dry density relationships for eight soils compacted according to the standard Proctor method (Johnson and Sallberg 1960; cited by Holtz and Kovacs 1981). (*Reprinted with permission from Prentice Hall, Inc.*)

the particles develop larger and larger water films around them, which tend to "lubricate" the particles and make them easier to be moved about and reoriented into a denser configuration. However, eventually a water content is reached at which the density does not increase any further. At this point, water starts to replace soil particles in the mold, and because the density of water is much less than the density of the mineral grains, the dry density curve starts to fall off, as shown in Figure 3-3. Note that no matter how much water is added, the soil never becomes completely saturated by compaction.

Compaction behavior of cohesive soils as described in the preceding paragraph is typical for both field and laboratory compaction. The curves obtained will have different shapes and positions on the ρ_d versus w plot, but in general, the response will be similar to that shown in Figure 3-4, where the same soil is compacted under different conditions. The shapes of the curves are different because the types or modes of compaction in the laboratory are different from those in the field. Additional information on the properties of compacted soils can be found in soil mechanics textbooks and the references given in Chapter 1.

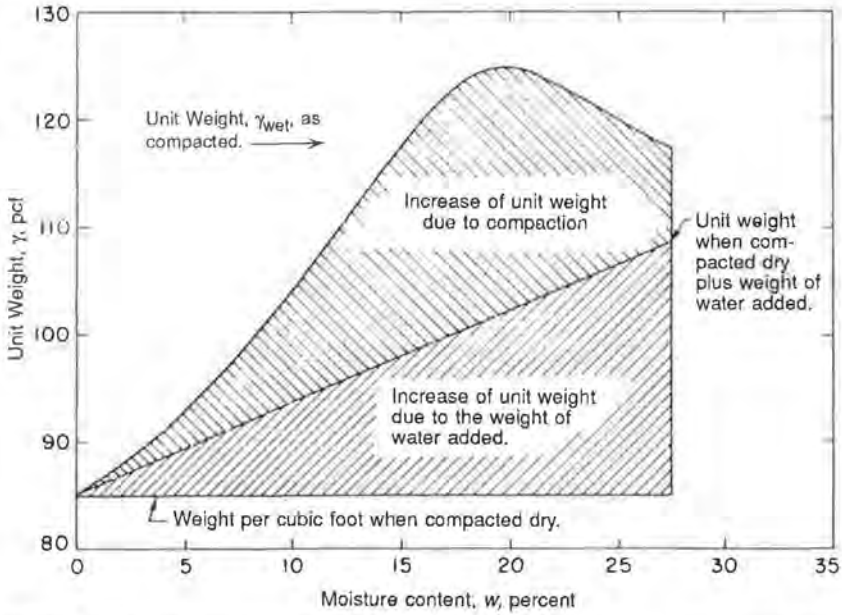

FIGURE 3-3 Water content–density relationship indicating the increased density resulting from the addition of water and the applied compaction effort. Soil is a silty clay, LL = 37, PI = 14, standard Proctor compaction (Johnson and Sallberg 1960; cited by Holtz and Kovacs 1981). (*Reprinted with permission from Prentice Hall, Inc.*)

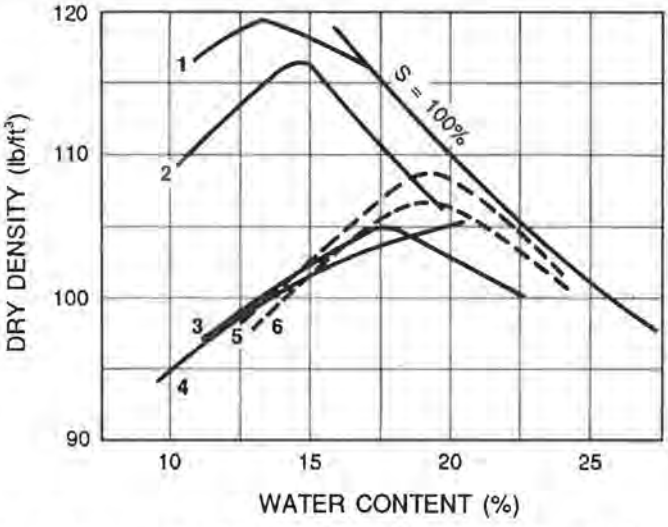

FIGURE 3-4 Comparison of field and laboratory compaction. (1) Laboratory static compaction, 2000 psi; (2) modified Proctor; (3) standard Proctor; (4) laboratory static compaction, 200 psi; (5) field compaction, rubber-tired load, six coverages; (6) field compaction, sheepsfoot roller, six passes.

Note: Static compaction from top and bottom of soil sample (Turnbull 1950, cited by Lambe and Whitman 1969 and Holtz and Kovacs 1981). (*Reprinted with permission from Prentice Hall, Inc.*)

SPECIFICATIONS AND COMPACTION CONTROL

Because the objective of compaction is to stabilize soils and improve their engineering behavior, many inspectors often forget that the desired engineering properties of the fill are important, not just its dry density and water content. Dry density and water content do reflect quite well the engineering properties, and thus they are convenient to use for construction control, but they are not the primary objective of compaction.

The usual design-construct procedure is as follows. Laboratory tests are conducted on samples of the proposed soil materials to be used to define the engineering properties required for design. After the embankment is designed, the appropriate compaction specifications are selected, field compaction control tests are specified, and the results of these tests become the standard for controlling the project. Construction control inspectors then conduct these tests to ensure that the contractor actually adheres to the compaction specifications.

The reasons for conducting such tests are outlined in the panel discussion in ASTM (1964, pp. 80–135); comments by Johnson and Sallberg (pp. 101–104) and Turnbull (pp. 104–106; 126–127) are particularly relevant.

Specifications

There are basically two types of earthwork specifications: (a) method specifications, and (b) end-product specifications. With method specifications, the type and weight of rollers, number of passes, lift thickness, and the like, are completely specified, and the responsibility for the quality of the earthwork rests with the owner or agency and owner's or agency's engineer. Although method specifications often provide considerable savings in unit costs for earthwork construction, they require such a large investment in preconstruction engineering and testing that they are generally used for large compaction projects such as earth dams.

End-product specifications are commonly used for highways and building foundations. A certain relative or percent compaction is specified. Relative or percent compaction is defined as the ratio of the field dry density, $\rho_{d\,\text{field}}$, to the laboratory maximum dry density, $\rho_{d\,\max}$, according to some specified standard test, for example, the standard Proctor or the modified Proctor test; or

$$\text{Relative or percent compaction} = \frac{\rho_{d\,\text{field}}}{\rho_{d\,\max}} \times 100(\%) \qquad (3\text{-}3)$$

How is relative or percent compaction determined? First, the test site is selected. It should be representative or typical of the compacted lift and soil material. Typical specifications call for a new field test for every 1,000 to 3,000 yd^3 or so, or when the soils change significantly. It is also advisable, if possible, to conduct the field test at least one or two compacted lifts below the already-compacted ground surface, especially when sheepsfoot rollers are used, or in granular soils, to be sure that loose materials near the surface are not included in the tested volume. Also, it is necessary to ensure that the materials the contractor has compacted are acceptable for the particular district or region according to the standard agency specifications or special provisions for the project.

Compaction Control Tests

Field control tests can either be destructive or nondestructive. Destructive tests involve excavation and removal of some of the fill material, whereas nondestructive tests indirectly determine by nuclear means the density and water content of the fill.

The steps required for the common destructive field tests are as follows:

1. Excavate a hole in the compacted fill at the desired sampling elevation (the size will depend on the maximum size of material in the fill). Determine the weight of the excavated material.
2. Determine the water content.
3. Measure the volume of the excavated material. Techniques commonly used to measure volume include the sand cone and balloon methods, or for rock fill and large holes, pouring water or oil of known density into the hole (Figure 3-5). In the sand cone method (AASHTO T 191; ASTM D 1556), dry sand of known dry density is allowed to flow through a cone-shaped pouring device into the hole. The volume of the hole can then easily be determined from the weight of sand in the hole and its dry density. In the balloon method (AASHTO T 205; ASTM D 2167), the volume of the excavated material is determined directly by the expansion of a balloon in the hole.
4. Compute the total density. If the total weight of the material excavated from the hole and the volume of the hole are known, the wet density can be computed. Because the water content is also known, the dry density of the fill can be determined.
5. Compare the field dry density with the Proctor density for that soil and calculate relative or percent compaction (see Equation 3-3).

There are several problems associated with the common destructive field density test. First, it is difficult and expensive to conduct a sufficient number of tests for a statistical analysis of the compaction test results. The volume of material involved in each test is an extremely small percentage of the total volume of fill being controlled (typically, one part in 100,000 or less). Second, oversize particles (gravel, cobbles, etc.) common in some soil deposits must be correctly accounted for, otherwise the laboratory test results will be less than those achieved in the field.

Oversize corrections are discussed in AASHTO T 224 and in ASTM D 698 and D 1557. Ideally, it is desirable to have the complete compaction curve for each field test, but this is time-consuming and expensive. Consequently, the laboratory maximum density may not be known exactly. It is not uncommon in highway construction for a series of laboratory compaction tests to be conducted on representative samples of the soil materials for the highway. Then, when the field test is conducted, its result is compared with the results of one or more of these standard soils from the project site. If the soils at the site are highly variable, this is a poor procedure.

Alternatively, a "family of curves" or a "one-point method" (AASHTO T 272) is often used. In this approach, a family of curves is

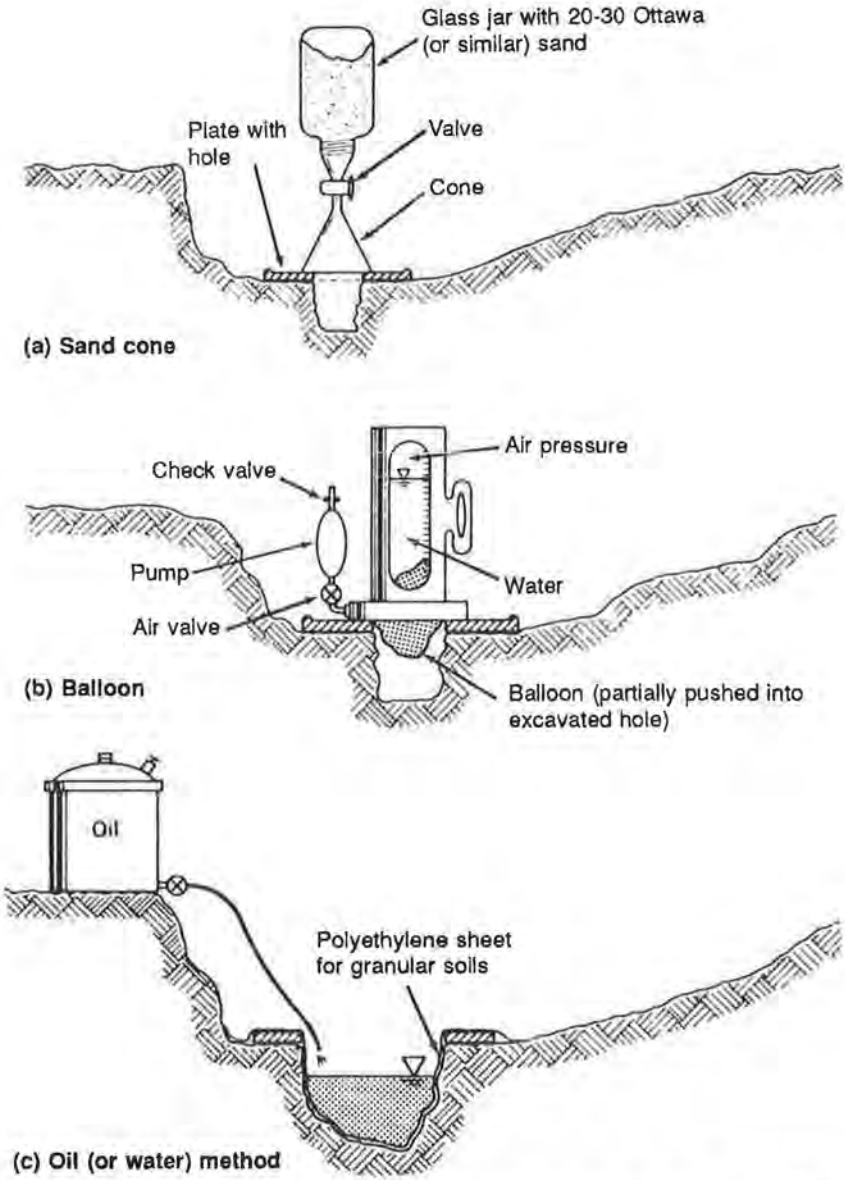

FIGURE 3-5 Some methods for determining density in the field. Note that Method (c) is suitable for large volumes, only as may be required for rock fill (Holtz and Kovacs 1981). (*Reprinted with permission from Prentice Hall, Inc.*)

developed by combining a series of Proctor curves for the various soil types common to a large area. One-point Proctor tests are conducted on the project soils and the results plotted on the family of curves. A Proctor curve having the same shape as the other curves is "fitted" into the family of curves by going through the one-point test results. Then the maximum density and optimum moisture for this one-point Proctor curve are determined, and the percent compaction is then calculated as usual by Equation 3-3.

The third major problem with the common destructive density test procedure is that determination of the water content takes time (several hours or overnight according to AASHTO T 265 and ASTM D 2216). Time is always of the utmost value on a compaction job, and if it takes a day or even several hours before the results are available, several lifts of fill may have been placed and compacted over the bad or failing test area. Then the engineer has to require the contractor to tear out possibly good fill to ensure that the relative compaction of the bad lift meets contract specifications. Contractors understandably are hesitant to do this, and yet how many zones of bad compaction should be allowed in an embankment? None, of course.

Because determination of water content takes the most time, several methods have been proposed to obtain it more rapidly. Pan drying or frying the sample over an open flame has been commonly used, but because it is difficult to control the temperature, this method gives poor results, especially for highly plastic clays.

Alternatively, a calcium carbide gas pressure meter (AASHTO T 217) can be used. The water in the soil reacts with carbide to produce acetylene gas; its pressure is proportional to the water content. Burning with methanol and the alcohol-hydrometer method are also sometimes used. For these methods, the correlation with standard oven drying is generally satisfactory for silts and lean clays.

If electricity is available at the field control laboratory, a microwave oven can be used to rapidly determine the water content. According to ASTM D 4643 (1989), microwave drying is not intended as a replacement for convection oven drying, but it can be used as a supplementary method when rapid results are required. The method appears to be satisfactory for most soils unless they contain significant amounts of halloysite, mica, montmorillonite, gypsum or other hydrated minerals, highly organic soils, or marine soils containing dissolved salts. Small, porous pebbles in the soil sample may explode when rapidly heated; therefore soil containers should be covered with heavy paper towels to prevent damage or injuries.

Another method for quickly and efficiently determining the relative compaction of cohesive soils was developed in the 1950s by the U. S.

Bureau of Reclamation (USBR 1974) and Hilf (1961) [see also Hilf (1975) and Holtz and Kovacs (1981)]. The procedure makes it possible to accurately determine the relative compaction of a fill as well as a close approximation of the difference between the optimum water content and the fill water content without actually oven drying the sample. Experience has shown that it is possible to obtain the values required for control of construction in about 1 hr from the time the field density test is performed.

Other problems with destructive field tests are associated with the determination of the volume of the excavated material. The sand cone method (AASHTO T 191, Figure 3-5a), often taken as the standard, is subject to errors. For example, vibration from nearby equipment will increase the density of the sand in the hole, making a larger hole volume; this results in a lower field density. A higher density will result if the technician stands too close to the hole and causes soil to squeeze into it during excavation. All of the common volumetric methods are subject to error if the compacted material is gravel or contains gravel particles.

Any unevenness in the walls of the hole causes significant error in the balloon method (AASHTO T 205, Figure 3-5b). If the soil is coarse sand or gravel, none of the liquid methods works well, unless the hole is very large and a polyethylene sheet is used to contain the water or oil (Figure 3-5c).

Density and Water Content by Nuclear Methods

Because of the problems with destructive field tests, nondestructive density and water content testing using radioactive isotopes has increased in popularity during the past few years. Nuclear methods (AASHTO T 238 and T 239; ASTM D 2922 and D 3017) have several advantages over traditional techniques. Tests can be conducted rapidly and results can be obtained within minutes. Erratic results can be easily and quickly double-checked. Therefore, the contractor and engineer know the results quickly, and corrective action can be taken before too much additional fill has been placed. Because more tests can be conducted, a better statistical control of the fill is provided. An average value of the density and water content is obtained over a significant volume of fill, and therefore the natural variability of compacted soils can be considered.

Disadvantages of nuclear methods include the relatively high initial cost of the equipment and the potential danger to field personnel of exposure to radioactivity. Strict radiation safety standards must be enforced when nuclear devices are used, and only properly trained and licensed operators are permitted to use nuclear density equipment.

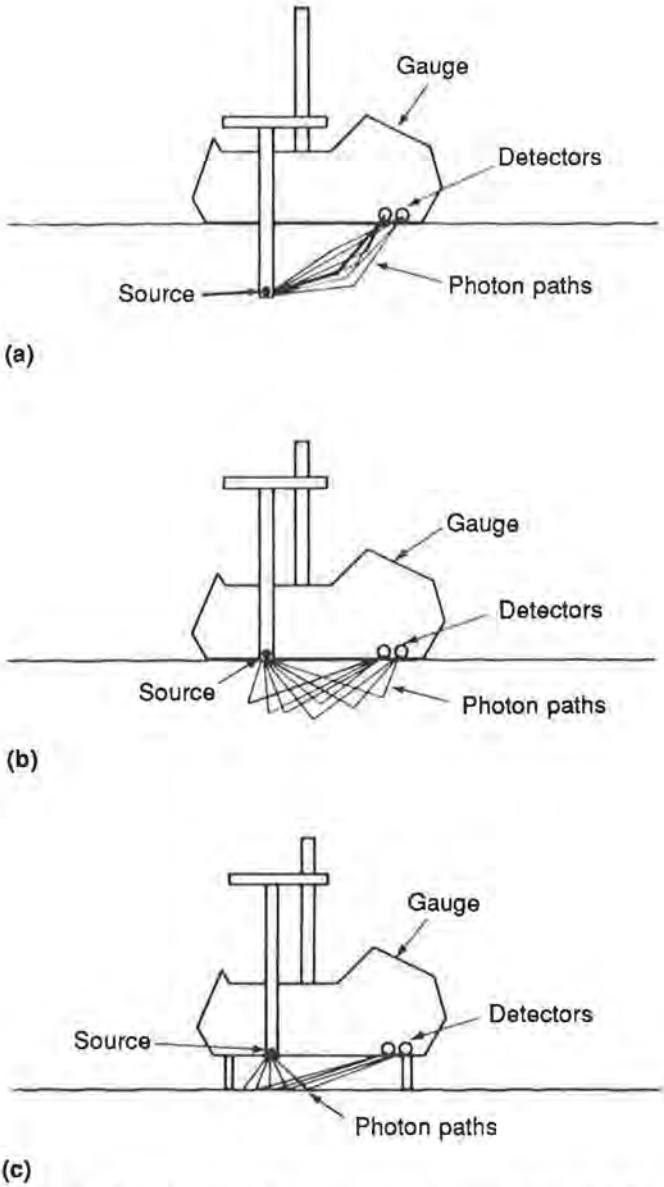

FIGURE 3-6 Nuclear density and water content determination: (a) direct transmission, (b) backscatter, (c) air gap (Troxler Electronic Laboratories, Inc.; cited by Holtz and Kovacs 1981). (*Reprinted with permission from Prentice Hall, Inc.*)

Two types of sources or emitters are necessary to determine both the density and the water content. Gamma radiation, as provided by radium or a radioactive isotope of cesium, is scattered by the soil particles; the amount of scatter is proportional to the total density of the material. The spacing between the source and the detector, which is usually a scintillation counter or a Geiger counter, is constant. Hydrogen atoms in water scatter neutrons, and this provides a means whereby water content can be determined. Typical neutron sources are americium-beryllium isotopes. Calibration against compacted materials of known density is necessary, and for instruments operating on the surface, the presence of an uncontrolled air gap can significantly affect the measurements. Filling the gap with dry sand helps reduce but does not eliminate this effect.

Three nuclear techniques are in common use. The direct transmission method is shown schematically in Figure 3-6a, and the backscatter technique is shown in Figure 3-6b. The less common air-gap method (Figure 3-6c) is sometimes used when the composition of the near-surface materials adversely affects the density measurement. For detailed test procedures, see AASHTO T 238 and 239 and ASTM D 2922 and D 3017.

REFERENCES

ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
USBR	U. S. Bureau of Reclamation

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4

Earthwork Construction

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This chapter is intended to acquaint field engineers and inspectors with good earthwork construction procedures and the reasons for developing them. It is not part of the contract documents and should not be used to supersede project plans, specifications, or special provisions. Field personnel must be thoroughly familiar with the project specifications. Questions and problems concerning earthwork construction should be referred to the project engineer and other staff specialists who are familiar with good embankment construction and can provide assistance when needed.

Good construction and materials will look good; and with reasonable care and effort, and thorough and competent inspection, an acceptable product will result.

PRELIMINARY WORK

Before the start of construction, the engineer and the contractor should review the topography of the project and the contractor's proposed erosion control plan for protecting the project from the elements. As the representative of the owner or agency, it is the engineer's responsibility to protect the property and facilities adjacent to the project from environmental pollution that originates from the project.

It is to the contractor's advantage to control surface runoff so that damage to completed work will be minimized and present and future operations will not be hindered. The benefits of erosion control are substantial. From the owner's or agency's viewpoint, the adjacent lands and waterways will not be polluted; from the contractor's viewpoint, surface water will be prevented from saturating the embankment foundation area, rainwater will run off rather than onto the embankment surface, and operations such as trench excavation and pipe installation will be protected.

Erosion control operations do not hinder the progress of the project significantly, and if they are performed, the contractor will not be hampered by a rainfall. At the most, some minor repair, such as removal of small quantities of mud, may be needed before full production can proceed. Items designated for temporary erosion control are usually in the contract to supplement the permanent features.

Good earthwork is most easily ensured by firm control of operations early in the contract when many seemingly more important operations also require attention. The amount of time spent closely inspecting the contractor's methods at this stage will be well spent because once the earthwork operation is correctly established, it generally runs smoothly. Therefore, an understanding of the contractor's proposed earthwork construction plan will permit realistic scheduling of the inspector's duties. This usually results in smoother and more efficient inspection that will require less field testing. Increased production for the contractor results in fewer delays and fewer problems.

CLEARING AND GRUBBING

Limits of clearing and grubbing are generally noted on the plans. Usually areas outside the work limits are to be left in their natural condition unless otherwise designated on the plans. In general, it is intended that the roadway fit into the landscape in a pleasing way. Natural features should be left undisturbed where possible.

Clearing is defined as the removal of all trees, brush, and so forth, and is required in all work areas. Grubbing is defined as the removal of stumps and roots. It is not always necessary to remove all stumps and root systems beneath embankments. Trees and brush should be cut off close to the original ground surface so that the initial layers of fill can be placed and compacted properly. Specifications should be read carefully to determine grubbing requirements.

SLOPE DITCHES

The specifications may require that top- or mid-slope ditches be completed before removal of materials from the cut. Some contractors prefer to ignore this detail as it is not a "production" item (such as excavation and fill placement), believing that they can complete the ditches whenever they have extra time. However, if the ditch work is not done in the beginning, problems may arise later that are detrimental to the work, and expensive corrective procedures may be required.

Two problems could occur if the ditches are not completed beforehand. First, the ditches are designed to collect surface runoff that otherwise would flow down the slope and into the excavation. This water can cause serious erosion as well as problems with cut-slope stability. Second, if the water is allowed to flow into the cut, the material to be excavated will become saturated, creating potential compaction problems when it is placed in the fill.

By delaying the ditch work, the contractor will create problems that not only are costly to correct, but also may delay other operations. Lack of accessibility to the work area, problems with disposal of the excavated materials, and improper placement of the ditch lining material, where required, result when ditch work is not performed before excavation is started. The engineer should insist that the contractor perform ditch work first. See Chapter 5 for additional information on drainage and erosion control.

EXCAVATION

Because the topsoil at the surface of earth cuts is usually unsuitable for use in compacted earth fills, it is normally stockpiled for later use in landscaping the project. The limits and depths of topsoil removal, where specified, are usually included in the plans.

As the excavation progresses it is good practice, depending on the topography and soil conditions, to keep the portion adjacent to the design slope at least 2 to 5 ft lower than the general level of the cut until the bottom is reached. In other words, providing a minimum of 2- to 5-ft-deep ditches along the sides of the excavation until the bottom "payment line" is reached helps to drain the section, resulting in a drier, more stable material for the contractor to excavate, transport, place, and compact. Because the cut is drained, the contractor will also have fewer equipment mobility problems.

Because of the highly variable properties of naturally occurring geologic deposits, specifications for roadway excavation often consider all soil and rock as unclassified excavation. This avoids the controversy, for example, of which bid price should apply to which material, or whether a deposit can be ripped or must be blasted.

ROCK EXCAVATION

If the material to be excavated is too tough or cemented to be ripped, then before drilling and blasting operations are begun, the engineer, contrac-

tor, and owner's or agency's geotechnical specialist experienced in blasting should hold a meeting at the project site to discuss all aspects of the blasting operations and set the ground rules for the contractor's operations and the basis for inspection of the work. The blasting specialist can help in determining the aspects of the work that need the most inspection, what to look for, and how it should look. A good reference on rock blasting is the FHWA manual (Konya and Walter 1985).

The owner's or agency's goal in the construction of rock slopes is that they require a minimum of maintenance and that they be hazard free. The contractor must thoroughly strip (overall removal of unsound material) or scale (removal of loose masses of rock) all rock slopes. If, after stripping or scaling operations, the engineer believes a hazardous rock slope situation still exists, a specialist should be requested to review the rock slope to determine if a hazardous condition exists and if so, propose solutions to correct it. For details on design and construction, the FHWA manual on rock slopes (Golder Associates 1989) should be consulted.

Some rock cuts will be designed with a broken rock trench to drain the section. The trench may be formed by extending the presplit or the production drill holes, or both, that fall within the typical section, or by drilling supplemental holes as necessary and fragmenting the rock. The limits and typical sections should be shown on the plans. Remember that this treatment must have a positive drainage outlet. Therefore, the plans and field conditions should be examined to confirm that positive drainage is provided.

EMBANKMENT FOUNDATIONS

The embankment foundation provides the base upon which the fill is constructed. After clearing and grubbing, the contractor should prepare the foundation in accordance with the plans, specifications, standards, and any special provisions. Preparation of the foundation may require stripping topsoil (particularly for fills less than 10 ft high), removing organic deposits and underwater backfill, compacting the ground surface, or placing a construction lift (a stable platform for later construction activities). In general, for problems identified during design, specific treatments will be shown on the plans or described in the contract documents. However, site conditions may be different, and some revisions to the design and construction plans may be required.

Embankment foundations are discussed in detail in Chapter 6. Construction procedures will depend on how firm or how soft the foundation is and whether the foundation soils require special treatment.

Firm Ground

No special foundation work is necessary in situations in which the height of embankment above firm ground is greater than the expected maximum depth of frost penetration. The topsoil may be left in place, especially under the outer edges of embankments. All embankment layers should conform to compaction specification requirements.

Depending on its natural in situ density, the surface on which the embankment or subbase, or both, is to be placed may need to be compacted to provide a stable platform upon which to place the subsequent embankment or subbase materials, or both. Proper compaction of each lift depends to a large degree on the density and stiffness of the surface upon which it is placed. If the foundation soil is in a loose state, the contractor may have difficulty compacting the initial embankment layers, unless the foundation is properly prepared and compacted first.

Clearing, grubbing, removal of the topsoil, and other special treatments will be noted on the plans when the embankment height is less than maximum depth of the expected frost penetration.

Soft Ground

Embankments that cross low, wet areas may require an initial stabilization layer, which is a thick lift of granular material, in order to provide adequate support for construction equipment. Usually specifications allow the engineer to permit a working platform up to 3 ft thick to be placed in one lift to bridge the soft areas. Experience has shown that sand, gravel, or well-graded blasted or crushed rock is excellent for this initial lift, especially when used with a stabilization geotextile (Christopher and Holtz 1985).

In some cases, low wet areas are anticipated during design, and a construction or stabilization lift with select materials is specified. Compaction of these initial lifts in soft areas should proceed with caution. In general, the use of vibratory rollers should be discouraged, as the vibration may cause the underlying soil to be pumped up into the granular fill. Separation geotextiles may be used to significantly reduce the thickness of this granular stabilization layer (Christopher and Holtz 1985).

Embankments placed on soft foundation soils may be designed so that the soft soils can be left in place. Methods of foundation treatment include the use of preload surcharges, sand, or prefabricated drains to accelerate the consolidation, stone columns, side berms, and so forth. [See Chapter 6 for a description of foundation stabilization techniques and additional references; for example, see NCHRP Synthesis of Highway Practice 147

(Holtz 1989).] These treatments are often critical and, when required, will be shown on the plans. They must be constructed strictly according to the specifications, and designers of these treatments should communicate their special concerns to the responsible construction personnel.

Unsuitable Materials

Unsuitable materials, such as peats and organic soils, mine and municipal wastes, swelling and collapsible soils, and so forth, are not generally appropriate for embankment foundations and must either be removed (undercutting) and replaced or specially treated. Limits and depths of removal or treatment of unsuitable materials will be shown on the plans; construction procedures will be detailed in the contract documents. Foundation treatment methods are discussed in Chapter 6 and by Holtz (1989).

Undercutting is the process of excavating below the usual subgrade cut limits to remove unsuitable soils. If these soils are anticipated during design, the undercut will be shown on the plans, backfill materials will be specified, and payment quantities provided. Unanticipated subgrade conditions encountered during construction that need corrective action should not be overlooked or ignored. The engineer should always be consulted in these cases.

The materials used to backfill undercuts and their placement requirements depend on the reason for the undercut and the site conditions encountered when the work is performed. Each case is different, and materials that are good for one case may not be satisfactory for another.

The two primary conditions for which undercut and backfill work are necessary are to (a) minimize damage caused by differential frost action, and (b) provide a stable platform to support the pavement (and construction operations).

In areas of cold winter weather, frost action, particularly differential frost heave, is probably the most dangerous situation encountered. As noted in the section on Frost Action in Embankment Design and Construction, Chapter 8, the conditions necessary for frost action are (a) presence of water, (b) freezing temperatures, and (c) frost-susceptible soils. Water may be controlled to some degree by side ditches and underdrains, but most frost problems occur because of capillary action, which is not helped by drainage. Frost-susceptible subgrade soils can be removed and replaced with clean, free-draining granular materials that are less susceptible to frost action. In this case, undercutting is designed to specifically reduce or eliminate frost susceptibility.

Sometimes problem soils are treated with lime to reduce the effects of frost heave. Areas that are selected to be undercut or lime modified should not be changed in the field without proper approval. Cold weather

construction is discussed in Chapter 4; environmental aspects of frost action are described in Chapter 8.

Instability of natural subgrade soils is the other reason for corrective undercutting and backfilling work. In cuts, the pavement subbase is generally placed directly on the natural soil or rock exposed by the excavation unless the natural soils have insufficient strength or are highly compressible. In such cases, the undercut and backfill is designed to provide a stable platform to support the pavement structure.

Wet, silty soils are responsible for most subgrade stability problems. Groundwater emerging on the floor of the excavation can saturate these soils and reduce their strength. Construction equipment can cause excess pore pressures and weaving or pumping (see section on Weaving and Pumping) of the subgrade, or rutting (see section on Rutting), and machinery vibrations tend to draw water to the surface that further reduces soil strength. Sometimes conditions get so bad that the wet soils have to be excavated with a drag line.

The requirements for the placement and compaction of backfill material in undercut areas depend on the condition of the soils at the bottom of the excavation. When the bottom is firm, backfill can be placed and properly compacted according to the specifications. For soft foundations, stabilization layers or geotextiles, or both, probably will be required, as described earlier in the section on Soft Ground. In any event, for undercut situations, lift thickness and compaction equipment do not necessarily have to conform to normal compaction specifications. For example, vibratory rollers should be avoided because the vibrations may cause the silt to pump up into the backfill.

Subgrade stability problems often depend on local water conditions at the time that the cut is made. Because stability problems are difficult to determine during design, the limits for undercut and backfill work are generally left to the engineer to determine in the field. Stability problems may be reduced in some cases if the cut is allowed to drain after it has been completed to the original pay lines. A waiting period may allow the groundwater table to stabilize and the material beneath the subgrade to dry out somewhat. This may allow the extent and depth of the undercut to be reduced, which would result in a significant cost saving. These treatments are critical and they must be constructed strictly according to the design. When required, they will be shown on the plans.

Disposal of Excessive or Unsuitable Materials

All excess or unsuitable materials should be disposed of in the areas designated on the plans, preferably within the right-of-way. Disposal outside the project limits may be necessary in some cases. Whenever

additional disposal areas are needed, special attention should be given to their location, method of construction, and final appearance. All disposal areas, inside or outside the right-of-way, should be under the direct control of the engineer.

EMBANKMENTS

Usually, materials that are to be used to construct the embankment are selected by the contractor and approved by the engineer. Many agencies have general requirements in their standard specifications for the soil types that are acceptable for embankment construction. These requirements or any special provision for the project should be strictly adhered to so that no unsuitable materials are used.

Suitable materials, if properly placed and compacted, will make satisfactory embankments. Water content in the natural state has no bearing on suitability. However, materials with excessive moisture will require drying before placement and compaction, or they must be replaced with materials having a proper water content. The location of the project will dictate which approach is appropriate, or perhaps the contractor will make the decision. During excavation if materials are uncovered that have an excessive water content, construction personnel should refer to the contract specifications. If there is still a question about whether to use the wet soils, the engineer should be consulted.

Some man-made materials may be suitable for constructing embankments (see Chapter 9, section on Waste Materials). Excavated pavement materials are inert and can be used. On the other hand, incinerator ash and other wastes may contain hazardous substances (see Chapter 8, section on Hazardous and Objectionable Materials) that could leach into the groundwater if placed in an embankment or cause a dust problem during construction. The use of waste materials and any related environmental regulations should be explained in the project soils report, which should be read carefully and made available to project personnel.

Swelling soils (see Chapter 9, section on Compaction Problems with Swelling Clays) are common in some parts of the country and can cause serious problems, particularly on pavements. Methods of treatment, use, or disposal of swelling soils will be detailed in the plans or specifications. Disturbance by excavation and placement in fills may change the stable environment in which the soils existed before construction to an unstable environment that allows the absorption of moisture. When these soils absorb moisture, they can swell tremendously or exert large, undesirable swelling pressure if movement is restrained. Methods of treating and

handling swelling soils are usually recommended by specialists in these materials.

Frozen soils (see Chapter 8, section on Frost Action in Embankment Design and Construction) are unacceptable for embankment fills because they are difficult to compact. Construction operations should be stopped whenever frozen materials are brought onto the fill. Cold-weather construction problems are discussed later in this chapter.

When rock is used in embankments, care must be taken to achieve dense fills. Otherwise, large voids or cavities may exist within a fill, and finer materials from above can settle into the voids. Eventually the subbase material supporting the pavement is lost and the pavement fails.

In constructing a rock fill, the proper sequence of operations is to dump the rock onto the lift under construction. The material is then pushed by a bulldozer over the leading edge of the lift, thoroughly wetted, and compacted with heavy equipment. Materials with sizes up to 2 ft may be placed in lifts up to 3 ft in maximum thickness. The larger sizes should be placed near the outer slopes, and very large (> 1 yd^3) boulders should be embedded in the slopes, broken down to smaller sizes, or wasted. Finer materials must be applied to the top of the layer being compacted to fill any voids.

Shales and other materials that break down during compaction present special problems; the use of shales in embankments is discussed in Chapter 9, section on Construction of Embankments of Shale.

COMPACTION

General

Compaction of a soil layer is probably the most important aspect of proper embankment construction. A uniform, densely compacted embankment will provide a satisfactory platform upon which to place the base courses and pavement. The word "uniform" is important in that uniform conditions during construction of the embankment will result in uniform behavior of the pavement, assuming that foundation conditions do not enter the picture. The benefits of good compaction are substantial and the consequences of poor compaction are severe. As noted in Chapter 3, compaction increases bearing capacity, slope stability, and resistance to frost action. It also decreases settlement and permeability. Inadequate compaction may result in general and differential subsidence, which causes depressions and perhaps premature failure of the pavement.

The contractor should be encouraged to route his hauling equipment as evenly as possible over the entire surface of the embankment during fill

placement. The purpose of this is to reduce the total amount of compactive effort required as well as to minimize localized rutting and damage that might be caused by heavy, repetitive, concentrated tracking by equipment. Loaded self-propelled scrapers may weigh in excess of 100 tons and may overload even a densely compacted embankment. Equipment operators have a natural tendency to follow the established track (path of least resistance), and some effort will be required to have them cover the entire surface. A good contractor will have a motor grader or dozer on the embankment to keep the surface smooth and to allow increased speeds over the entire surface. The performance of the embankment under the tires of scrapers will also give a good indication of the uniformity and quality of compaction.

Moisture Control

As discussed in Chapter 3, moisture acts as a lubricant and helps the soil particles to move relative to each other into a denser condition when compactive effort is applied. Thus dry soils must have water added and mixed thoroughly throughout the layer being compacted. Adding water in the cut rather than on the fill improves mixing and increases uniformity. However, adding water to the loosely deposited fill on the excavation is the usual method employed. The choice is ordinarily left to the contractor.

If a soil is too wet, the compactive effort only increases the pore water pressure and this tends to keep the particles apart. Using a heavier compactor results in a decrease in strength at the same water content (see section on Weaving or Pumping). Thus, wet soils must be dried; natural drying is the most widely used method. To hasten drying, the soil may be spread and mixed by the use of disks, harrows, or rotary tillers. Lime has also been used to dry soil, but it is expensive; however, it also stabilizes the soil (TRB 1987).

Each project should receive from the laboratory a set of laboratory compaction control charts or a family of curves (AASHTO T 272) developed from standard (AASHTO T 99) or modified (AASHTO T 180) Proctor values representing the soils being used in the embankment. These curves are plots of density (or dry unit weight) versus water content and provide a standard of acceptability for the field tests.

When field compaction control tests (Chapter 3) are performed, the results are compared with the laboratory compaction control chart values or the family of curves to determine the applicable percent of maximum density, called the relative compaction or sometimes the percent compaction (Equation 3-3). The specifications will give the percent compaction

required. If this value is not obtained, additional compactive effort or a change in water content will be required to reach the minimum specified value.

The specifications or laboratory compaction control charts will usually indicate a water content range associated with the maximum required density or percent compaction. To achieve the proper compaction, the moisture content can be varied depending on the embankment soils and compaction equipment. However, to do this requires specialized testing and analyses, and field personnel should not attempt such an analysis without proper training.

Moisture control becomes more critical as the particle size of the material being compacted decreases. Clays are greatly affected by changes in moisture content whereas sands are not. Small amounts of fines in granular soils will also affect moisture requirements. Well-graded materials will usually exhibit steep, sharp compaction control curves showing well-defined optimum moisture content (OMC) (Figure 4-2). Uniform materials, particularly sands, will exhibit flat curves with no well-defined OMC. These latter materials may be successfully compacted at a relatively large range of moisture contents, although they may experience bulking problems at some water contents.

Weaving or Pumping

Weaving or pumping is an elastic-type deformation of the soil. When loaded, the material deforms and as the load is removed, the material springs back to its original position (almost like a waterbed). The construction equipment looks as if it is riding on a wave as it travels over the fill. The soil will deflect and a wave will be created ahead of the wheel, but once the equipment moves on, the area looks the same, although there may be some cracking of the surface. Weaving occurs when there is excess moisture in the soil that does not have time to drain as the load is applied. The load is then borne partly by the soil structure and partly by the pore water pressure. This gives a temporary elasticity to the soil, thus creating the weaving or pumping effect. Note that in this condition the strength of the soil is substantially reduced.

Initially, weaving is not necessarily damaging to the embankment. The easiest solution is to simply stay off the area and allow the excess pore water pressures to dissipate naturally. The soil will then tend to regain its strength. If the fill is weaving under the action of compaction equipment, a lighter compactor will produce lower pore water pressures and thus reduce weaving and pumping. However, repeated loadings will continue to create cumulative pore pressures and may ultimately result in shear

failure or rutting. If a weaving condition exists, the engineer should be called for advice. The engineer should then explain to the contractor that continued compaction operations can only worsen the situation.

Rutting

Rutting is a surface shear or bearing failure. As the equipment moves across the embankment, the loads imposed exceed the shear strength of the soil, the wheels sink, and deep ruts occur. Rutting destroys the previous compaction and makes it impossible to place the next lift to a uniform thickness. The integrity of the work suffers and the contractor's operations are hindered. Corrective measures such as changing the method of operation, materials, or loading are usually the contractor's responsibility.

COLD WEATHER CONSTRUCTION

It is extremely difficult, uneconomical, and under some circumstances, virtually impossible to compact moist or wet soil while freezing temperatures exist and to obtain the densities necessary for proper performance of an embankment.

Experience has shown that, if adequate densities are not obtained during construction, significant differential settlement and sideslope instability will occur as the frozen portions of the embankment thaw. Depending on the dimensions of the embankment and the location(s) of the frozen portions, the thawing process may take several years. The resulting poor performance of the embankment may require substantial maintenance expenditures to correct.

Consequently, most agencies located in freezing climates and engaged in earthwork construction do not permit embankment construction during the winter months. The only exception is construction using blasted rock. The specifications of one northeastern state read: "Earthwork construction operations requiring compaction shall not be performed from November 1 through April 1, except with written permission of, and under such special conditions and restrictions as may be imposed by the Regional Director."

Temperature has a noticeable effect on soil compaction when the temperature of the soil is above freezing. Raising soil temperature increases the maximum dry density obtained from a given compactive effort. In contrast, lowering soil temperature causes the water in the soil to become more viscous, reducing the workability of the soil, and conse-

quently, lowering the maximum dry density for a given compactive effort (Johnson and Sallberg 1962).

Soil temperature becomes critical to the compaction process at approximately 32°F. As the soil temperature falls below 32°F, there is an immense decrease in the maximum dry density obtained from the application of any given compactive effort. When the water coating the soil grains freezes, the three-phase system of soil grains, water, and air that existed above 32°F becomes a two-phase system of soil grains, each coated with ice and air. This latter system does not occur at exactly 32°F but at somewhat lower temperatures, probably as a result of the heat energy imparted to the soil by the action of the compaction equipment or perhaps as a result of the pore water chemistry.

Compaction of soils at temperatures below freezing was investigated on a section of Interstate 47 in Albany, New York, by the Bureau of Soil Mechanics of the New York State Department of Public Works during the winter of 1957–1958. The roadway was on an embankment for the entire length of the project. Construction began in the spring of 1957, and excellent progress was made during the summer and fall months. In late fall the contractor requested permission to continue embankment construction operations during the winter.

Until this time, it was believed by many, but not all, of the department's construction engineers that noncohesive semigranular and clean granular materials could be properly compacted at temperatures below freezing with no significant problems. Because the only two types of soils to be compacted were a fine sand with a trace of silt (used for embankment construction) and a sand with some gravel and a trace of silt (used for trench, bridge, and culvert backfill purposes), permission was granted, provided that the contractor achieve at all temperatures and at all locations the dry densities stipulated by the specifications (AASHTO T 99, standard method). Because the project was located only a short distance from the Bureau of Soil Mechanics laboratory, it offered researchers an excellent opportunity to study in detail compaction of these soils at temperatures below freezing and resolve the controversy concerning compacting granular soils in freezing temperatures.

Construction continued into the winter of 1957–1958. The first discovery the bureau made was that it was essential that the frozen soil excavated from the test hole be thawed before it was compacted into the Proctor mold. As embankment construction proceeded into December, the contractor found it increasingly difficult to achieve the specified densities and finally ceased grading operations until spring.

Using the bureau's frost study facilities, the moisture-density relationships for each of the two soil types at 74°F, 30°F, 20°F, and 10°F under both AASHTO T 99 (standard method) and AASHTO T 180 (modified

method) compaction test procedures were established. The results are shown in Figure 4-1 for the brown fine sand with a trace of silt and Figure 4-2 for brown sand, some gravel, and trace of silt. The curves indicate that an immense increase in compactive effort is required to compact a soil at temperatures below freezing. This is particularly true of the finer-grained soils. Note that for the brown fine sand with a trace of silt (Figure 4-1), application of the standard compactive effort at 74°F achieved a higher density than application of the modified compactive effort, which has 4.5 times the energy of the standard, at 30°F.

The curves also show that at a temperature in the vicinity of 10°F to 20°F, depending on the soil type, the shape of the moisture-density curve changes completely, indicating that it may be practically impossible to properly compact a soil at or below those temperatures with presently available methods and equipment.

COMPACTION EQUIPMENT

The compaction equipment that the contractor selects should be determined by the type of soils encountered. Certain types of compactors work better with some types of soils than others. In many cases, however, the contractor will use whatever equipment he already owns or has leased. Whatever equipment the contractor uses must comply with the specification requirements and must be approved by the engineer. Minimum wheel loads and tire pressures for pneumatic rollers and minimum weight for steel wheel rollers are specified, whereas for vibratory drum compactors, a specific frequency range and a minimum dynamic force are usually specified. Length of feet, minimum weight per square inch of cross-sectional area of the tamping feet, and operating speed are specified for sheepsfoot rollers.

A maximum lift thickness should be specified, depending on the equipment being used or the project soils, or both. In this way, the inspector will know in advance the maximum thickness that the contractor will be allowed to place and compact. The contractor can place thinner lifts if he chooses.

Pneumatic-tired compactors achieve compaction by the interaction of (a) wheel load, (b) tire size, (c) tire ply, (d) inflation pressure, and (e) the kneading action of the rubber tires as they pass over the lift. Pneumatic-tired rollers should be ballasted to meet at least the minimum wheel load.

Vibratory drum compactors develop their compactive effort by vibrations. Four machine features must be known in order to rate vibratory rollers: (a) unsprung drum weight, (b) rated dynamic force, (c) frequency

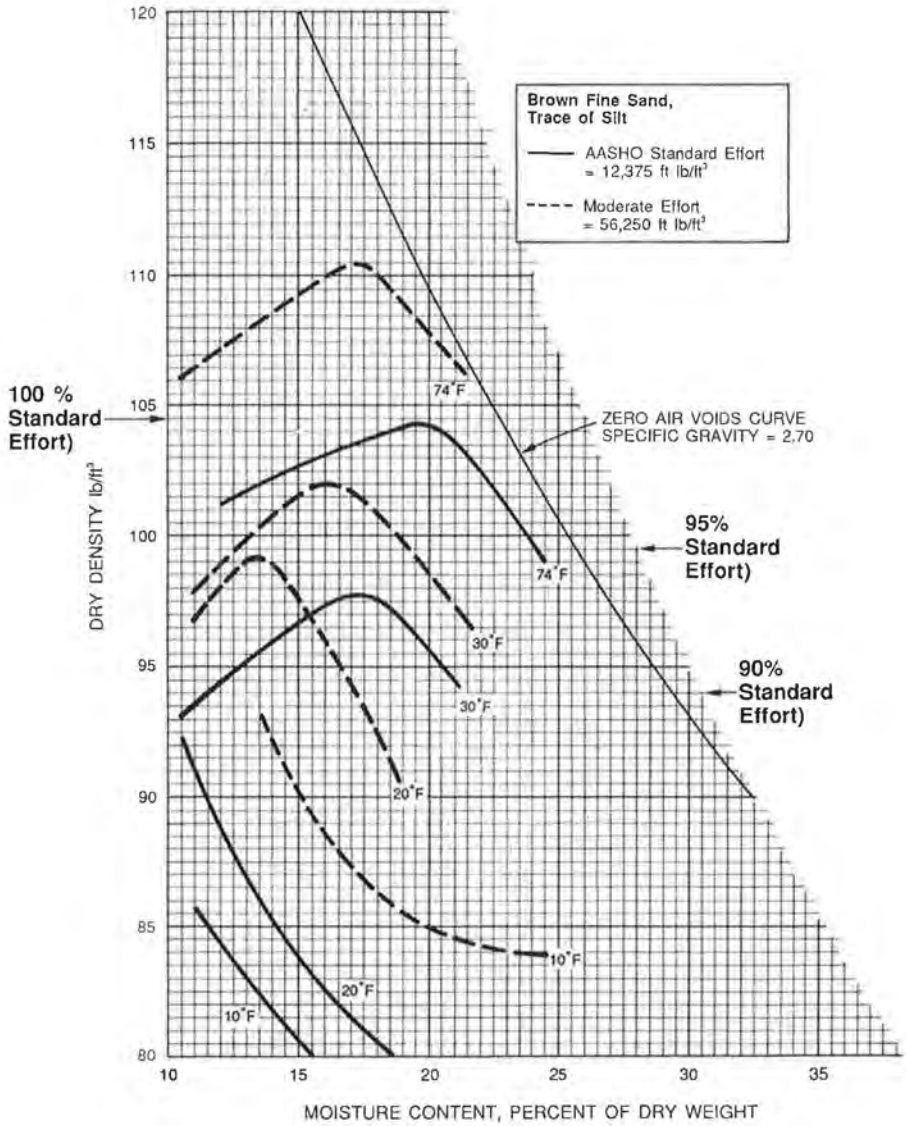

FIGURE 4-1 Compaction control curves (standard and modified AASHTO) for a brown fine sand with a trace of silt at different temperatures (*courtesy W. P. Hofmann*).

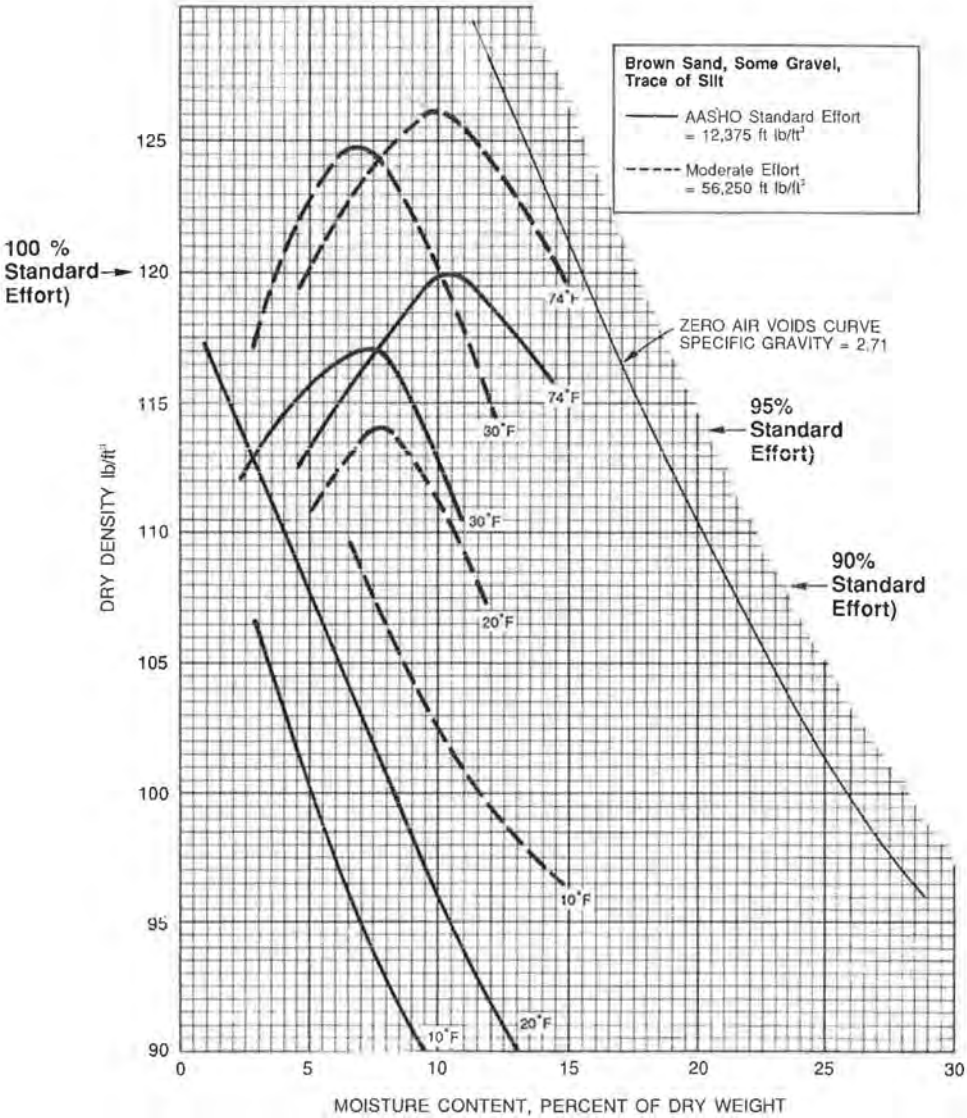

FIGURE 4-2 Compaction control curves (standard and modified AASHTO) for a brown sand with some gravel and a trace of silt (*courtesy W. P. Hofmann*).

at which the rated dynamic force is developed, and (d) drum width. The contractor or equipment supplier should have these data. Vibratory rollers should operate between 1,100 and 1,500 vpm, and the dynamic force at the operating frequency should be at least 2.5 times the unsprung drum weight (see the manufacturer's literature for the roller). Therefore, by using the machine data and the specification requirements, a range of acceptable frequencies can be determined.

The contractor should be required to provide at least one vibrating reed tachometer when vibratory rollers are used. This device is used to measure the frequency at which the machine is vibrating. The dynamic force is proportional to the square of the frequency. A reduction in the frequency will significantly reduce the compactive force. Therefore, the inspector should monitor the frequency often.

Compaction of granular soils is mostly due to the dynamic force created by a rotating eccentric weight. Vibratory compactors dramatically lose their effectiveness when the vibration is shut off because the compaction is due solely to the static weight of the machine. Satisfactory compaction of thick lifts cannot be accomplished in this case.

When sheepsfoot rollers are used, the criteria for job control can be determined by a test in the field. The feet must penetrate into the loose lift. If they ride on top, the machine is too light and the ballast must be increased. With succeeding passes, the feet should "walk out" of the layer. The number of passes required for the feet to walk out of the layer will then be used to control subsequent layers. If the feet do not walk out, the machine is too heavy and is shearing the soils, or the soil is too wet. The roller should be lightened and a new test should be performed for job control or the soil should be dried by the methods previously mentioned.

To be effective, smooth steel wheel rollers should weigh at least 10 tons and exert a minimum force of 300 lb per linear inch of width on the compression roll faces. These data can usually be obtained by referring to the manufacturer's specifications for the roller. At least eight passes over the lift at a maximum speed of 6 ft/sec is usually adequate. These rollers may be used on lifts of 8 in. or less of compacted thickness.

If the contractor wants to use equipment that cannot be placed into one of the preceding categories, a job site test should be performed to evaluate the effectiveness of the equipment and determine job control requirements. If such a situation exists, the engineer should be contacted for assistance.

COMPACTION IN CONFINED AREAS

There are two types of compaction: in large areas accessible to full-sized compactors and in confined areas accessible only to smaller, highly ma-

neuverable or hand-operated mechanical compactors. There is no precise definition of confined areas; each case should be considered on its own merits. For example, projects for which the pavement is being widened 2 ft on either side may be considered to be in a confined area, even though the job is 10 mi long, because the subbase material will be placed in a trench much narrower than the width of conventional embankment compaction equipment.

Compacting material behind a bridge abutment may require high maneuverability, which the usual fill compactors do not have. Pipe and conduit backfill and backfill behind retaining walls and minor structures are common cases that require confined-area compaction.

Any compaction equipment except hand tampers (unless mechanized) may be used in confined areas. Equipment that may not be acceptable in some situations may be acceptable in confined areas. For example, vibratory rollers with small diameter drums and operating at a high frequency should not be used to compact large areas, but they may be quite satisfactory in confined areas. The basic question for a confined-area compactor is, "Will it do the job?"

Material placed within confined areas should be limited to lifts not exceeding 6 to 8 in. before compaction. Some equipment may require even thinner lifts. The material should then be compacted with a number of passes sufficient to meet the specification density requirements.

INSPECTION

The field inspector has a number of aspects to check during compaction operations. There may be different density requirements in different areas of the fill. Certain types of compaction equipment may work better with one soil type than another. However, the contractor must have control over his own operations. If there are two ways to perform an operation, the contractor decides which method to employ. Continuous inspection can ensure that the work is performed in accordance with the specifications.

When specifications require certain equipment and methods of operation, the inspector will be required to verify that the equipment used complies with the specification requirements. If a job site test is required, the specifics of the test will be determined by the engineer. The inspector will probably be required to observe and measure the results, but it will be the engineer's responsibility to interpret them and determine whether the contractor has complied with the specifications.

It is extremely important that the layer or lift thickness not exceed the maximum allowable thickness. As the lift thickness increases, the effec-

tiveness of a compactor is reduced significantly because the compactive stresses at the bottom of the layer are too low to ensure proper compaction.

From tests using a pneumatic-tired roller, it is known that stresses are reduced by one-half at a depth of 12 or 14 in. The layer thicknesses given in the specifications should be established so that a desirable minimum stress level is exerted at the base of a layer, and together with the minimum number of passes, will result in proper compaction.

The inspector must also verify the compactive effort applied to the lift. This involves checking to ensure that the compactor applies at least the minimum number of passes at or below the maximum specified speed. The initial passes increase the density of the soil considerably. If the specified minimum number of passes is not applied, then the material at the bottom of the lift will not be compacted to the desired degree, and future settlement of the layer can be expected.

Verification of the number of passes becomes more significant because of limitations in the amount of density testing and the tendency toward thicker embankment lifts. The inspector is not going to test everything, but should base decisions to test on visual observations. In an end-result type of specification (see Chapter 3, section on Specifications), the type of equipment and number of passes probably will not be specified. To ensure that compactive effort is being obtained, much more density testing is usually necessary. Therefore, the inspector must use good judgment in such cases.

The stability of the lift under the action of the compactor will usually dictate the course of action to be taken. Corrective action should be taken when the lift shows significant weaving, pumping, or rutting under the action of the compactor (see sections on Weaving and Pumping and Rutting). This again is a judgment decision on the part of the engineer, but if a machine is merely leaving a tire print, this is not considered to be significant rutting. However, if the equipment displaces the soil laterally out of the wheelpath and leaves a visible rut, then something is wrong and corrective action must be taken before additional fill layers are placed. In this case, it does not matter what the present density is.

The main purpose of inspection of the compaction operations is to verify that the embankment is uniform and dense. A properly placed embankment will ensure acceptable performance throughout its useful life. As a general rule, a proper visual inspection of the compaction operations can ensure that this result is attained. Once the inspector knows how the compaction and hauling equipment affects a properly compacted layer, the contractor can be allowed to proceed with additional fill when the minimum specification requirements have been met.

In order to become familiar with soil conditions, it will probably be necessary to conduct more density tests at the beginning of the project and at the time that a new soil type is encountered. In this way, the compaction characteristics can be determined for the entire embankment lift rather than just at the test location. With the widespread use of the moisture/density nuclear gauges (see Chapter 3, section on Density and Water Content by Nuclear Methods), many more tests can be conducted in a shorter time and, therefore, specification compliance is more easily ensured today than in the past.

As described in Chapter 3, section on Specifications and Compaction Control, there are a number of methods for performing field density tests, such as the sand cone method (AASHTO T 191), balloon method (AASHTO T 205), and the nuclear gauge (AASHTO T 238). All of these tests provide acceptable results. Nuclear testing equipment requires a special license and may only be used by a licensed operator.

At the close of each day's work, the working surface of the fill should be crowned, shaped, and rolled with smooth steel wheel or pneumatic-tired rollers to ensure proper drainage, should it rain overnight. Thus, rainwater will be directed to the edges of the fill, and unless the rain is intense and of long duration, damage to the surface will be limited to the top few inches. The next day this wet material may be scraped off to the side to dry or be allowed to dry in place. Normal construction activities may then proceed. If, however, a significant rainfall occurs, the water may infiltrate the fill and require extensive corrective action. There will be instances in which this is unavoidable, but they will be minimized if the preceding preventive measures are taken.

STRUCTURE BACKFILL

The backfill of structures, pipes, and culverts must be performed correctly and compacted to specification requirements. Improper placement of backfill material or poor compaction can result in undesirable settlements and subsidence of the pavement.

Usually, a loose lift thickness of 6 in. and at least the same relative compaction as specified for the embankment are required. These specifications, together with the backfill gradation requirements, thorough inspection, and frequent density testing will generally ensure satisfactory results. Because the required degree of compaction is high and the areas are confined, the contractor must use small, highly maneuverable compaction equipment that does not exert high loads on the soil (see section on Compaction in Confined Areas). It is almost impossible to visually judge the adequacy of the compaction operations in these areas, and

much more frequent density tests are necessary. See Chapter 7 for additional information on structure backfills.

PROOF ROLLING

The specifications may require proof rolling of embankment fills in order to find areas of poor compaction, and in cuts, areas of the subgrade that are so soft that they will not satisfactorily support the proof roller. Proof rolling is done before placement of the subbase. If the compaction of the upper embankment layers is not uniform, or if the excavated soil conditions are not uniform and dense, these nonuniformities will be reflected in poor performance and high maintenance of the pavement.

In most jobs, the contractor has had complete control over the construction of the embankment, and proof rolling will provide a check on quality control. Because most proof rollers are ballasted with soil, a correlation can be determined between height of ballast in the roller box and different gross loads. An average density of 115 pcf for the ballast will generally give satisfactory results.

The proof roller should be operated briefly, and the response of the embankment under the action of the roller should be watched closely. If there is consistent lateral displacement of soil out of the wheelpaths, the proof roller may be loaded too heavily. Lateral displacement means rutting and shearing of the soil. Proof rolling is not designed or intended to cause the embankment to fail, but rather to point out areas of inadequate as well as nonuniform compaction. If the roller weight is reduced and consistent rutting still occurs, the proof roller should be further unballasted. This procedure may be followed until the roller does not consistently displace the soil.

Once a final acceptable weight has been determined, the roller should make two complete coverages on the subgrade surface within the outside edges of shoulders (roadway limits). Depressions should be filled with material similar to the subgrade soil so that uniformity will be maintained.

Major deficiencies must be corrected; the corrected areas are then recompacted in the normal manner and proof rolled again. All corrected deficiencies are at the contractor's expense until the subgrade surface shows a satisfactory uniform response to proof rolling. It is the earthwork contractor's responsibility to provide a suitable foundation for the pavement structure. Until this suitable foundation is provided, the earthwork construction is not complete, with two exceptions: proof rolling embankments may be eliminated in areas of limited access or maneuvering space

when it might damage adjacent construction or when the proof roller may come within 5 ft of a culvert, pipe, or other conduit.

Because correction of subgrade deficiencies in cuts is the owner's responsibility (see section on Soft Ground), once an area is undercut and backfilled by design, proof rolling should not be necessary. For example, an area ordered by the engineer to be undercut and backfilled because of conditions discovered during construction should not require proof rolling.

The proof roller used in cuts is normally loaded to 30 tons gross load and has tires inflated to 40 psi. Note that these conditions may be different from the ones normally used in embankment sections. It is not in the owner's interest to overload the roller because this may falsely show more areas requiring corrective undercut and backfill work.

Two complete passes are generally satisfactory. The engineer may require additional undercut and backfill work based on the action of the proof roller on the subgrade. As in embankment sections, proof rolling in cuts is not intended to destroy the subgrade, but to point out areas of inadequate subgrade support.

TRANSITIONS

As the name implies, transitions—both longitudinal and transverse—attempt to provide a gradual change from one subgrade support condition to another. Significant differences, if they are abrupt, will be adversely reflected in the finished pavement.

There are two main subgrade conditions in which transitions are necessary: (a) cut to embankment fill, and (b) rock cut to soil cut. Transitions are generally constructed with materials, as specified, that contain no particles that have a maximum dimension greater than 6 in. The intent is uniformity rather than a material with special properties. Therefore, specifying a high quality fill material should only be necessary when conditions warrant. For example, if the soil cut is unstable in a soil cut-rock cut transition, the high quality fill material should be continued into the rock cut.

Water in the subgrade may run along the surface of the rock and, unless it drains, may saturate the soil cut, causing an unstable condition. The rock may be naturally fractured or porous enough to remove any excess water. At times, blasting operations near this rock-soil interface will fracture the rock below the required subgrade level and provide an outlet for the water.

The necessity for longitudinal transitions depends on conditions at the site when the cut is completed; it cannot be determined during design. Therefore, each longitudinal transition from rock cut to soil cut must be

inspected and evaluated by the engineer, who will decide whether a longitudinal transition should be installed.

BENCHING

When embankments are to be constructed on slopes or where a new fill is to be placed against an existing embankment, the slopes of the original hillside or existing embankment normally are benched in order to key the new fill into the existing slope. Benching is usually specified for all embankments intersecting an existing earth slope that is one vertical on three horizontal or steeper, either transversely or longitudinally. Without benching, the sloping original ground surface creates a natural plane of weakness when the embankment fill is built against it. Benching breaks up the potential failure plane, thus increasing the stability of the entire system. The widths of the benches are variable, depending on the slope angle, with the height typically held to approximately 4 ft.

SLOPE PROTECTION

Often excavations for earthwork construction intercept the existing groundwater table, thus interrupting the natural flow of groundwater. This does not affect the highway until the groundwater flow emerges on the cut slope. If the flow is small, there may be no adverse effects. However, when the flow is significant and the conditions at the site are favorable, flowing water can cause seepage forces that will in turn cause the slope to slough or fail.

To stabilize slopes under these conditions, a heavy material is placed on the face of the slope. This material is heavy enough to hold down the existing soil even though seepage forces are acting in an outward direction. At the same time, it is open enough to carry all the water emerging from the existing soil. A coarse-graded stone, slag, or gravel blanket on top of a recommended geotextile has proven to be effective in these cases. The drainage blanket should be designed before construction, but weather conditions during construction will substantially influence the actual need for such treatment. Chapter 5 contains additional information on erosion control and drainage.

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ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
FHWA	Federal Highway Administration

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5

Drainage

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Water affects many aspects of earthwork construction. In the vicinity of the construction area, surface or subsurface water needs to be controlled by some type of drainage system. Controlling usually means removing, but there are some instances in which water is needed to facilitate construction, for example, to increase compaction efficiency (Chapters 3 and 4) or to maintain vegetative growth (Chapter 8).

Controlling water on the construction site is one of the contractor's major concerns, but it tends to be neglected because often the drainage work is not a pay item. However, good drainage helps the contractor produce a more acceptable project; therefore, the contractor's profit should be increased. A job that controls erosion and stays out of the mud is much easier to run and much more efficient. Drainage is important because

1. Soils that are too wet or too dry cannot be efficiently compacted to specified densities.
2. Most soils lose strength when they are wet, that is, above optimum moisture, and both the stability of cut and fill slopes and the operation of equipment at the construction site are adversely affected.
3. Erosion can damage adjacent property.

BASIC DRAINAGE PRINCIPLES

The basic principles of drainage are as follows:

1. Water runs downhill.
2. Gravity is cheaper than plumbing.
3. Erosion depends on water velocity.
4. Erosion problems are easier to prevent than to fix later.

The aspects of drainage that affect earthwork construction are discussed in this chapter. Included are control of erosion of side slopes,

ditches, and the basins at the ends of culverts; sedimentation control measures, including detention ponds; and subsurface water and underdrains.

SURFACE WATER

It is important to remove surface water from the embankments as quickly as possible to prevent it from being absorbed by the near-surface materials. Surface waters should also be controlled to prevent erosion, and if erosion occurs in spite of attempts to prevent it, the eroded soil particles should be encouraged to settle out of the runoff. The plans and specifications will almost always include permanent surface water control features (primarily ditches) to protect the finished facility; however, additional surface drainage systems will also be needed during construction. These as well as all other drainage items should be put in place as soon as possible, as they will simplify subsequent construction.

The area surrounding the site should also be considered because it may be possible to check erosion by retarding the flow of water onto the site or to prevent any water from encroaching on the site. This may not be easy to do on roadway widening projects, however. Runoff from the existing pavement and changes to drainage systems often make the problem worse on existing pavement than on new construction. It is false economy to allow slopes or drainage paths to erode badly, even though the contractor may be required to pay later for repairs as well as stabilization (HRB 1952).

Erosion

The detrimental effects of erosion can be minimized by understanding the erosion process and taking prompt and correct remedial action. The amount of erosion that takes place depends on (a) the velocity of the water that flows over the surface, (b) the type of soil or material that the water flows over, and (c) the vegetative cover. The relationship between the velocity of water and erosion is very important. Doubling the velocity of water will increase its erosive energy by four times, the size of particle that can be carried by 64 times, and the mass of soil that can be transported by 32 times (Israelson, 1980a). Thus, if job site erosion is to be prevented, certain characteristics that affect velocity must be kept in mind:

1. Slope—the steeper the slope, the faster water will flow.

2. Roughness of the slope channel—the smoother the channel, the faster water will flow.
3. Depth of flow—the deeper the flow over a surface, the faster it will flow.
4. The shape of the flow channel—the smaller the channel surface area in contact with the water, the faster the flow.
5. Quantity of flow—the larger the quantity, the faster the flow.

These characteristics lead to the prime rule of erosion control: do not allow water to concentrate; keep it dispersed whenever and wherever possible.The erodibility of a slope also depends on the type of soil and vegetation cover. Loose noncohesive soils tend to be more prone to erosion than dense cohesive soils. Fine sands and silts are the most erodible. Vegetation tends to decrease soil erodibility.

The contract plans and specifications will detail any required erosion control measures. The basic philosophy of some of the more common erosion control measures is explained in the sections that follow.

Control of Erosion on Side Slopes

The most critical time for slope erosion is immediately after the start of excavation or embankment construction. The soil on the surface is loose and is exposed to rainfall, and vegetation has not yet taken hold. A typical raindrop travels about 19 mph, and when it hits the soil surface, it displaces unprotected soil particles and starts washing them downslope (FHWA 1976; Israelson 1980a). The following measures should be taken to prevent side slope erosion:

1. Protect the newly exposed slope from the high velocity impact of raindrops. This should be done as soon as possible; one short, intense rainfall can ruin a well-designed and constructed slope. The most common method of dissipating the energy on newly exposed soils is by mulching. The mulch absorbs the impact energy and breaks up the raindrops. Common types of mulch are hay or straw, wood chips, crushed stone, and geotextiles. Details on the quantities of mulch to use can be found elsewhere (Israelson 1980b; CCSWC 1985).
2. Do not permit excessive quantities of water from outside the construction area to flow down cut slopes. Top-of-slope ditches are an important preventive measure (see Chapter 4, section on Slope Ditches). They should be put in promptly, as the slope is most vulnerable immediately after construction. Top-of-slope ditches need to be handled carefully,

however, as they tend to be quite steep (ditch erosion is discussed in the next section.

3. Keep the side slopes as flat as possible. (Cuts in rock, loess, or other lightly cemented permeable soils are exceptions.)

4. Be sure that the water arrives on the slope in a sheet and keep it in a sheet as long as possible. This is done by constructing continuous flat or slightly rounded slopes. Eventually adjacent slopes will intersect and the water will have to be concentrated into a ditch or stream, preferably under controlled conditions.

5. Establish a dense growth of grass or other vegetation as soon as possible. Vegetation slows down the water and lowers the erodibility of the slope surface. Mulching (Item 1) protects the slope and the seed or seedlings until the vegetation takes hold. After the vegetation is established, it ordinarily will dissipate the energy of raindrops.

Control of Erosion in Ditches

The following precautions should be taken to control erosion in ditches:

1. Ditches should carry no more water than necessary. Adjacent land owners should solve their own drainage problems, if possible.
2. Ditches should carry water only as far as necessary.
3. Avoid changes in direction of flow in a ditch. Use pipes for high flows and sharp turns.
4. Do not allow water to fall into a ditch. It should enter the ditch with as little impact as possible. Check dams can be engineered to handle large flows, but drop inlets into pipes or erosion protection linings are more appropriate for small ditches.
5. Do not allow water to pond in a ditch. When this happens the ditch is not carrying out its intended drainage function.

Well-sodded ditches should be used whenever possible; however, sod needs sun and cannot be maintained in channels that approach continuous flow. Also, it may not be practical or cost effective to sod a ditch during construction.

The factors that affect the velocity of flow in ditches are the same as those mentioned previously: (a) slope, (b) ditch roughness, (c) depth of flow, (d) ditch shape, and (e) quantity of flow. Slope is difficult to control because it is dictated by the geometry of the site. Nevertheless, slopes between about 0.5 and a maximum of about 2 percent should be maintained wherever possible.

Roughness of the ditch helps to decrease velocity at low flows, but it is not as effective at high flows. The quantity and depth of flow and the ditch

shape are all interrelated. The general rule is to have wide flat ditches and keep them as short as possible.

If after everything else has been considered, the water is still eroding the natural ditch lining, then an alternative lining is needed. One of the most effective linings for small ditches is a geotextile, either natural (jute) or synthetic. This design, if well constructed, will tolerate about the same water velocities as grass sod.

Stabilizing Eroded Ditches

Fast-flowing water in a ditch or stream is always turbulent, and this produces large traction forces that dislodge material from the stream bed and carry it downstream. A natural stream develops a filter system of successively smaller particles below the stream bed, which is relatively stable at normal flow velocities. Given time, ditches will also develop these same filter systems, but usually by the time a construction ditch stabilizes, large gullies or other problems will have occurred. By constructing a filter system using natural aggregate materials or aggregate materials and geotextiles, it is possible to stabilize an eroding ditch.

To be successful, filter systems must have the following characteristics:

1. The aggregate or stones in the top layer must be large enough to resist the water traction forces.
2. The particles in each successive lower layer are smaller than those in the layer above, but they are too large to pass through the voids in the layer above it.
3. Each layer must allow water to freely pass through it.

In many cases a two-layered system is all that is needed, although sometimes three aggregate layers are required. Geotextiles can be used in place of one or more of the finer filter aggregate layers (Christopher and Holtz 1985), but for permanent installations, the geotextile may need to be protected by a layer of gravel between it and large stones on the ditch bed. Riprap used to protect stream beds and slopes adjacent to water bodies require granular filters or geotextiles and should be constructed accordingly.

Culvert and Pipe Outlets

Because water leaving culvert and pipe outlets is concentrated and usually traveling at velocities that will erode the natural stream bed, these fea-

tures are of particular concern. In most cases, construction of a filter system that includes riprap from the outlet to a point downstream where the stream velocity is harmless will solve the problem.

In severe cases, stilling basins together with filters are needed. Care must be taken to protect the natural material at the edges of stilling basins until water velocities become tolerable. Placing large rocks or riprap at the ends of pipes (or in other parts of a stream bed that is experiencing erosion problems) without providing a filter underneath is not the way to solve the problem. Eventually the stream will form its own stilling basin and filter, but usually this will not happen until after some detrimental erosion has occurred.

SEDIMENT CONTROL

Sediment is the soil material that settles out of a dilute mixture of soil and water. Sediment will separate from the water when the velocity is slowed to a level that will no longer carry or move the soil particles. Environmental Protection Agency regulations require that material that has been eroded on a construction site must be retained within the bounds of the construction area.

Sedimentation control is needed on a construction site because of the inability of the contractor's erosion control practices to prevent all erosion. Sedimentation of material from water is not necessarily easy, and it is usually false economy to allow erosion to take place and then try to recover the materials.

All sedimentation control facilities must perform three functions: (a) decrease the velocity of the water to a level that will allow the suspended material to settle, (b) retain the water for a sufficient time for settling to take place, and (c) release the water without causing erosion or flooding downstream.

Because sedimentation control measures must decrease the velocity of the water in a stream or ditch, the size and extent of the facilities required are directly related to the quantity and duration of flow. The simplest sedimentation control measures, straw bales and geotextile silt fences, can be used on smaller swales and ditches when the flow is intermittent. Larger continuous flowing ditches or streams require detention ponds and barrier structures, as discussed in the next section.

Straw bales and silt fences are not complicated structures, but to function successfully they must be properly constructed and maintained. These barriers are designed to allow the water to seep through the material. Water must not be allowed to flow around the edges or under silt

barriers, or flow over the top in a concentrated high velocity flow. The channel downstream of the barrier should be constructed to carry the flow without allowing further erosion to take place. The ditch section just beyond the silt barrier may need erosion protection similar to that used at culvert outlets.

Any required periodic maintenance should be detailed in the contract specifications. For example, to ensure that the velocity is slowed and that there is sufficient storage in case of a storm, the backwater area must be cleaned if the deposited material comes halfway up the height of the barrier. Any damage or displacements in the barriers should be repaired immediately. Clogged barriers should be replaced.

Detention ponds should be treated as major features, and the plans and specifications for the barriers should be followed closely. The barrier is a dam and should be designed and constructed as such. Failures of these structures could cause major downstream damage. Areas of particular concern are

1. *Construction of the embankment.* Density and material requirements for different parts of the structure should strictly follow the plans and specifications.
2. *Location and construction of the emergency spillway.* This is essentially a large steep ditch and the necessity of including a good filter system cannot be overemphasized. Failures of spillways can cause rapid erosion and loss of the retention structure. A stilling basin or other method of dissipating the energy of fast flowing water is needed at the base of the spillway to reduce the velocity of the discharge to the receiving stream.
3. *Installation of pipe conduits that extend through the embankment and act as a spillway and pond drain.* Because a pipe through an embankment is a major discontinuity, seepage between the embankment and the pipe is a potential source of failure. It must be controlled by the careful compaction of select material around the pipe and by the installation of seepage collars at specified lengths along the pipe.

SUBSURFACE WATER

The emergence of subsurface water is of concern on the construction site because it affects the strength and load-carrying capacity of the soil; it is a factor in almost all failures of soil slopes and excavations. Because the original source of the subsurface water is rain that has fallen somewhere upslope of the site, careful observation of the area surrounding the site will usually indicate the source of the subsurface water problem.

Underdrains

Subsurface water is generally removed with underdrains such as trench or French drains. Although these drains are easy to design, they are difficult to construct without segregation of the granular filter or contamination of the filter and drain aggregate. As a result they are often not as effective as they should be.

Today, most underdrains are constructed using either geotextiles to replace the graded granular filter materials or with prefabricated geocomposite drains, which greatly simplify construction operations. To ensure that the geotextile or geocomposite will work as designed, however, several important points must be kept in mind when installing them. They should be protected from dirt and contamination, exposure to sunlight, and damage during shipping and storage. Installation and backfilling must be carefully done to avoid tearing or puncturing the geotextile. If the drain is a geocomposite, care must be taken to avoid damaging the core. Finally, the drainpipe and its outlets must be properly located. See *Geotextile Engineering Manual* (Christopher and Holtz 1985) for additional information about using geotextile filters and geocomposites in underdrains.

Common Subsurface Drainage Problems

Sometimes unanticipated subsurface water problems occur during construction or maintenance. For example, shallow slope failures can occur because of subsurface water seeping from a slope or moving parallel to and near its surface. Three possible solutions are

1. Install one or more subsurface drains to lower the groundwater surface. These drains may also help to prevent deeper slope failures. A drain at or near the ditch line will also tend to increase the strength of the roadway subgrade.
2. After undercutting, place a thick blanket of stone or rock on the slope. Although this will help to prevent shallow slope failures, it will not increase the strength of the roadway subgrade nor will it help prevent deeper slope failures.
3. Flatten the slope so that the wet slope is stable.

If the roadway is unstable because of free water at or near the surface, often it is possible to solve this problem by crowning the roadway and constructing deeper side ditches to remove the surface water and to lower the water table below the roadway. After crowning the roadway and

constructing side ditches, subsurface drains can be installed to further lower the water table. It may not be necessary to install these drains on both sides of the roadway. This method may be too slow to be helpful on a construction job because it may take too long to actually lower the water table. However, over the long term, this installation may be necessary to ensure a more stable roadway foundation. In this case, request advice from the engineer.

Another solution would be to install a geotextile and aggregate surface on the crowned and ditched roadway. This will permit the water to drain from the subgrade while preventing the subgrade material from intruding into the coarse stone subbase layers. The aggregate layers must be permeable so that they will remain stable when wet. Note that these solutions assume that corrective action can be taken without changing the grade of the roadway. If this is impossible, see Chapter 4, section on Unsuitable Materials.

If an excavation for a structure foundation is unstable because the original groundwater table is above the bottom of the excavation, side ditches can be constructed around the periphery of the excavation to lower the water table. The ditches must drain to a sump where the water can be pumped from the excavation.

A geotextile-aggregate mat thick enough to provide a stable working surface can be installed. The peripheral ditches with the sump pump are also needed. The aggregate mat needs to be permeable to internally drain water to the peripheral ditches and to remain stable when wet.

Well points around the periphery of the base can be installed to lower the free water surface to produce a stable base. The required spacing, usually between 3 and 12 ft, depends on the type of soil and the desired depth of groundwater lowering (Cedergren 1989). Again, these solutions assume no change in the elevation of the bottom of the structure foundation.

REFERENCES

ABBREVIATIONS

CCSWC Connecticut Council on Soil and Water Conservation
FHWA Federal Highway Administration

CCSWC. 1985. *Connecticut Guidelines for Soil Erosion and Sediment Control.* Connecticut Council on Soil and Water Conservation, State Office Building, Hartford.

Cedergren, H. R. 1989. *Seepage, Drainage, and Flow Nets,* 3rd ed. Wiley, New York, N. Y., pp. 254–291.

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6

Embankment Foundations

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New roads are increasingly being located on poor foundation soils. Thus, comprehensive geotechnical embankment foundation design studies are required to identify and solve potential stability, settlement, and construction problems. The results of these studies are normally incorporated into the construction plans and specifications.

The success of a foundation design is generally judged by embankment performance, although rate and cost of construction may also be important factors. Performance is reflected in pavement rideability, which includes smooth transitions at structures, effects on buried utilities, and frequency of maintenance required during the design life of the roadway. These postconstruction factors are dependent on the amount and rate of settlements, assuming that foundation stability requirements are met satisfactorily during construction. The success of an embankment construction project is directly proportional to the design and construction effort, and it requires good predictions of the behavior of the foundation in response to the applied loads.

With proper specification and construction controls (Chapter 4), incidences of faulty embankment construction are rare. Poor embankment performance is usually a consequence of unexpected variations in subsurface or foundation conditions, inadequate design for site-specific soil conditions, or in cases where a proper foundation design was provided for in the contract documents, lack of strict adherence to the construction specifications.

Project design starts with a preliminary site evaluation. Next, exploratory borings and sampling are conducted, followed by a laboratory testing program, design analyses, and design report. Finally, recommendations for foundation treatment, if any, are incorporated into the contract plans and specifications.

This chapter contains background information on embankment foundation design as it affects construction operations and inspection procedures. Briefly discussed are the several phases of the embankment

*Retired.

foundation design process. Possible treatment alternatives, which may be required for particular problem foundations sites, are also mentioned. Specific problem foundation soils are discussed in Chapter 9.

Good general references to the design and construction of embankment foundations are books by Terzaghi and Peck (1967), Winterkorn and Fang (1975), Cheney and Chassie (1982), and U. S. Navy (1982), as well as other textbooks on foundation engineering. Information on methods of treating problem foundation soils can be found in works by Sinacori et al.(1952), Moore (1966), and Welsh (1987). Also recommended are *NCHRP Syntheses of Highway Practice 2, 8, 27, 33, 89, 107,* and *147.*

DESIGN

Preliminary Data Acquisition Activities

Every subsurface exploration program and subsequent design analysis should be preceded by a site inspection followed by a review of all available information pertinent to the project. The latter includes, for example, data from previous projects in the area, geological and pedological reports and maps, well logs, U. S.Geological Survey maps, aerial photographs, and any existing subsurface exploration data. From this information, such items as old slope failures and landslides, swamps and bogs, different soil types as revealed by landforms, buried stream channels, sinkholes, landfills and dumps, mining activities, and poorly drained areas may be located. All pertinent information should be available to the construction engineer, usually in the project soils report.

Exploration Programs

The boring and sampling requirements for a highway project depend on the size, complexity, and location of the project. Exploratory borings (auger, split spoon), undisturbed sampling for subsequent laboratory testing, or in situ tests may all be used in the boring program. For additional information on this phase of the design process, consult the AASHTO (1988) *Manual on Subsurface Investigations.*

Foundation Design Procedures

Granular soils such as sands and gravels generally provide stable embankment foundations. Settlements on these soils are usually small and occur as the embankment is built.

Soft compressible soils such as clays, organic silts, marls, and peats cause embankment stability and settlement problems. First, a model of the subsurface conditions (soil profile) is established, followed by determinations of the strength and settlement design parameters from interpretations of laboratory test results on undisturbed samples or possibly from in situ tests performed during the subsurface exploration program.

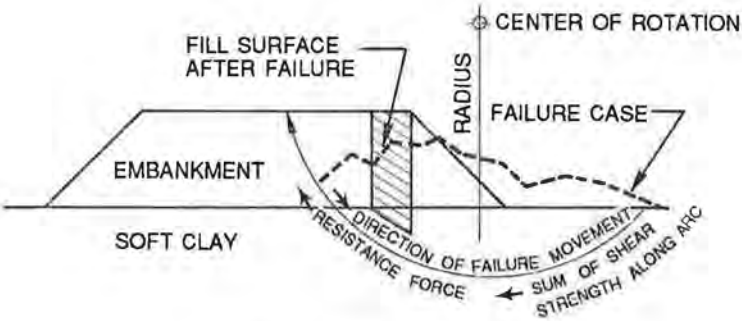
Stability Studies

Failures can occur in situations in which embankments are built on weak soils, such as soft clays, organic silts, and peats, without special foundation treatment. Foundation soils will provide adequate support if the additional stress from the embankment does not exceed the shear strength of any of the underlying strata. Overstressing the foundation soil may result in dramatic embankment failures, which generally occur in one of the ways shown in Figure 6-1. It is important for field engineers to be aware of these possibilities so that should unusual movements appear to be occurring or, for example, cracks start to appear in the embankment, the agency's geotechnical specialist should be contacted immediately. On critical projects or those in which the calculated factor of safety is marginal, the project soils report, or sometimes the project specifications, will state the acceptable limits of settlements or lateral movements of the embankment and foundation. In this case special geotechnical instrumentation (Chapter 10) is used to facilitate these performance observations.

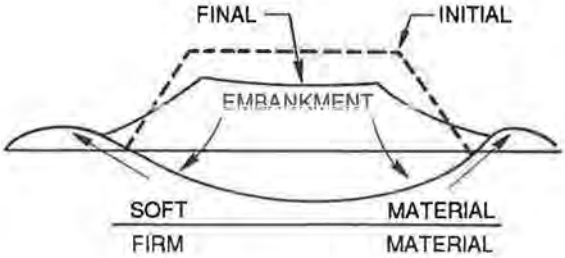
As mentioned earlier, granular foundation soils generally produce small settlements, and because they take place rapidly, usually as the load is placed, they ordinarily pose no particular difficulty in embankment design or construction. On the other hand, foundation soils such as soft clays or organic soils, or both, are capable of large continuous settlements, depending on their geological and loading history and the magnitude of the embankment load. In organic materials especially, settlements may continue almost indefinitely after a project is built. Unusual settlement problems, if anticipated, will be mentioned in the project soils report.

METHODS OF FOUNDATION TREATMENT

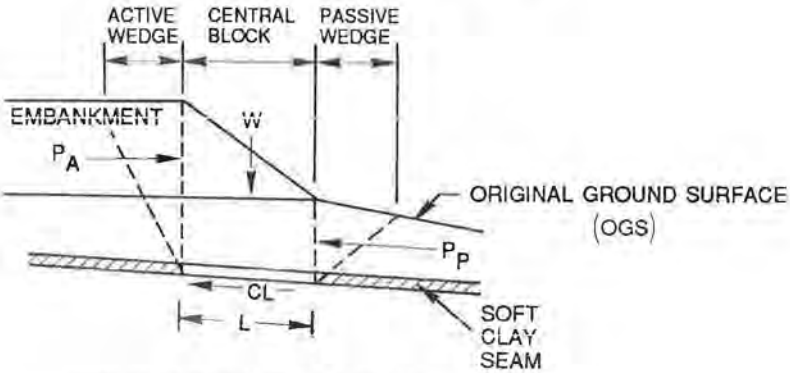
If the designer determines that the calculated settlements are too large or that stability problems are likely to arise from construction of the em-



(a) ROTATIONAL FAILURE



(b) DISPLACEMENT FAILURE



(c) TRANSLATORY FAILURE

where

P_A = ACTIVE FORCE (Driving) CL = RESISTING FORCE DUE TO COHESION OF CLAY

P_P = PASSIVE FORCE (Resisting) W = WEIGHT OF CENTRAL BLOCK

FIGURE 6-1 Typical embankment failures (*courtesy New York State Department of Transportation.*

bankment, it may be possible to lower the grade or shift the alignment to avoid or minimize potential problems. Stability and settlement problems are often interrelated and time dependent. Finding the most appropriate procedure for ensuring stability and minimizing settlements requires an analysis of various foundation treatment techniques. The two most important factors to consider when selecting a treatment method are economics and construction time, while taking into consideration the sequence of operations and the duration of the contract.

Basically, problem foundation soils can be improved by

1. Reducing the load,
2. Replacing the problem materials with more competent materials,
3. Increasing the shear strength and reducing compressibility of the problem materials,
4. Transferring the loads to more competent layers, and
5. Reinforcing the embankment or its foundation, or both.

For treating problem embankment foundation soils, these general concepts are actually accomplished by the following specific methods: (a) berms or flatter slopes, (b) lightweight fill materials, (c) pile-supported roadways and embankments, (d) removal of soft or problem materials and replacement with suitable fill, (e) stabilization by consolidation of soft foundation materials, (f) chemical alteration/stabilization, (g) physical alteration/stabilization, including densification, and (h) reinforcement. These methods and their variations are listed in Table 6-1. All have been used singly or in combination in the United States, although some methods are much more popular than others, and some have only been used on an experimental basis or for structures other than highway embankments. Variations and combinations of the methods listed in Table 6-1 can be considered applicable, but not necessarily the most economical, for virtually any thickness or type of problem soil.

Berms and Flatter Slopes

Embankment instability in the case of a rotational failure (Figure 6-1a) can be improved by adding a counterweight or stabilizing berm to the lower portion of the embankment (Figure 6-2). Berms often necessitate additional rights-of-way. The berm is normally constructed at the same time as the embankment, not afterward, as has been discovered too late in a few embarrassing cases.

TABLE 6-1 FOUNDATION TREATMENT ALTERNATIVES (Holtz, 1989).

Method	Variations of Method	Generally Applicable to		Is Treatment Generally Time Dependent?		
		Stability Problems	Settlement Problems	Yes	No	Possibly
1. Berms; flatter slopes	—	X	—	—	X	—
2. Reduced stress method	Lightweight fill.	X	X	—	—	X
3. Pile-supported roadway	Elevated structure supported by piles driven into suitable bearing stratum.	X	X	—	X	—
	Swedish method of supporting embankment on piles driven into suitable bearing material. Piles have individual pile caps covering only a portion of base area of fill.	X	X	—	X	—
4. Removal of problem materials and replacement by suitable fill	Complete excavation of problem materials and replacement by suitable fill.	X	X	—	X	—
	Partial excavation (the upper part) of soft material and replacement by suitable fill. No treatment of soft material not removed.	X	X	—	—	X
	Displacement of soft material by embankment weight, assisted by controlled excavation.	X	X	—	X	—
	Displacement of soft material by blasting, augmented by controlled placement of fill.	X	X	—	X	—

5. Stabilization of soft materials by consolidation	Consolidation by surcharge only.	—	X	X	—	—
	Consolidation by surcharge combined with vertical drains to accelerate consolidation.	—	X	X	—	—
	Consolidation by surcharge combined with pressure relief wells or vertical drains along toe of fill.	—	X	X	—	—
6. Consolidation with paving delayed (stage construction)	Before paving, permit consolidation to occur under normal embankment loading without surcharge; accept postconstruction settlements.	—	X	X	—	—
7. Chemical Alteration and Stabilization	Lime and cement columns; grouting and injections; electro-osmosis; thermal; freezing; organic.	X	X	—	—	X
8. Physical alteration and stabilization; densification	Dynamic compaction (heavy tamping); blasting; vibrocompaction and vibroreplacement; sand compaction piles, stone columns; water.	X	X	—	X	—
9. Reinforcement	Geotextiles and geogrids; fascines; Wager short sheet piles; anchors; root piles.	X	—	—	X	—

NOTE: Some combinations of methods are feasible.

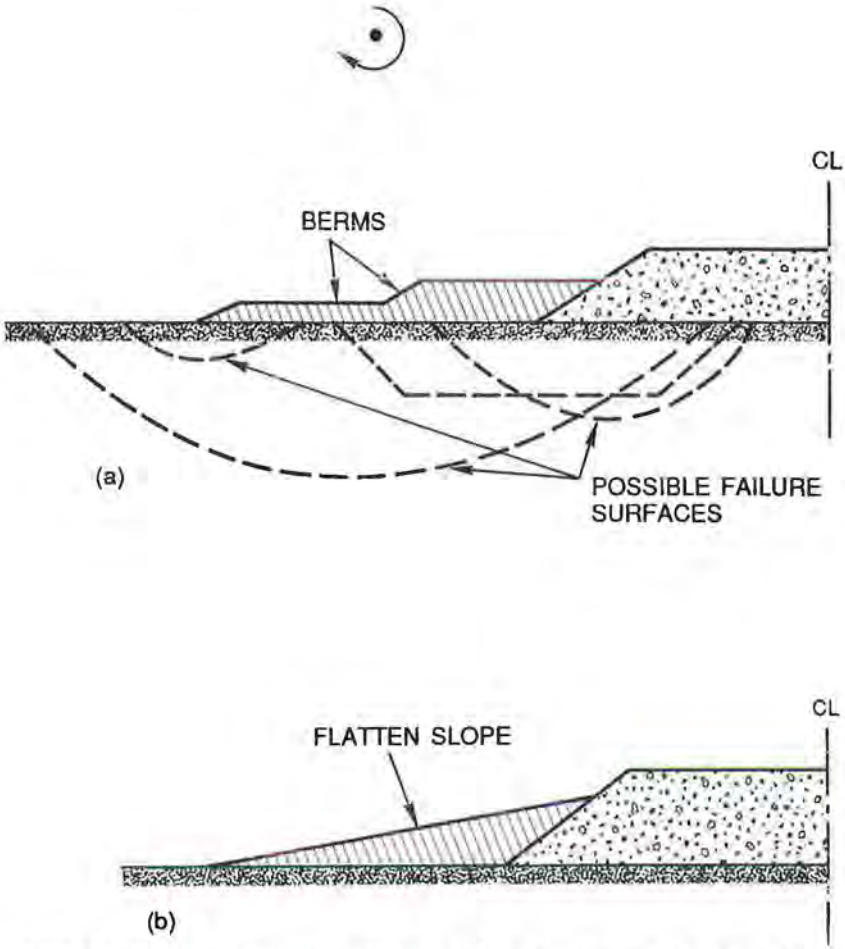


FIGURE 6-2 Embankments stabilized with (a) berms or (b) flatten slopes (*courtesy New York State Department of Transportation*).

Load Reduction

Where the roadway profile cannot be lowered, the use of lightweight embankment fill materials such as cellular concrete, expanded shale, slag, ash, cinders, sawdust and bark, shells, or expanded polystyrene may be considered to reduce the load on the foundation soils (Figure 6-3).

Special construction procedures for placing lightweight fill materials will be given in the special provisions of the project specifications (see Chapter 9, section on Lightweight Fill Materials).

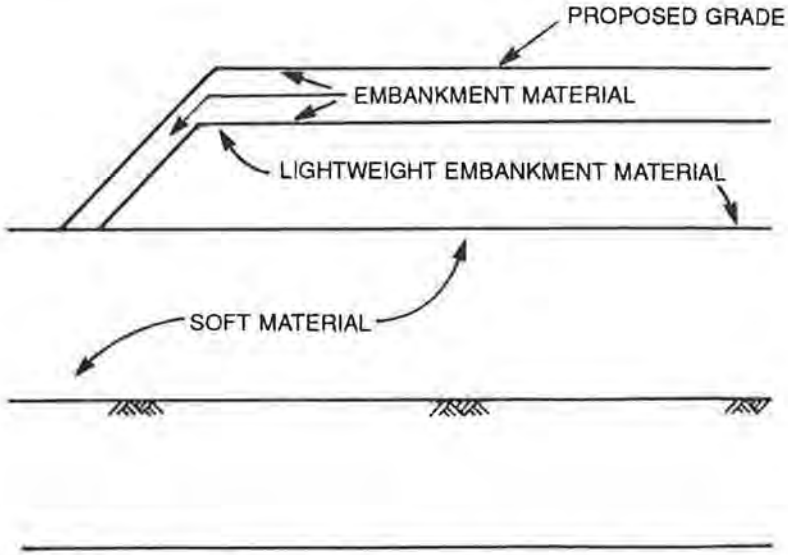


FIGURE 6-3 Use of lightweight embankment fill materials (*courtesy New York State Department of Transportation*).

Pile-Supported Roadways

Pile-supported roadways include elevated structures, such as bridges and viaducts, that are supported by pile foundations and earthen embankments that are supported by relief piles driven to firm bearing layers. The latter system is commonly used to support highway embankments in Scandinavia (Holtz 1989).

Excavation and Replacement

Where feasible, excavation of surface deposits, such as organic material or very soft clay, and replacement with select granular material is an effective means of solving foundation problems. As noted in Table 6-1, the excavation process may be either partial or complete. When the material to be removed is underwater, excavation and backfilling is usually carried out underwater to avoid collapse of the sides of the excavation.

Complete excavation (Figure 6-4) is appropriate where the depth to the bottom of the soft material is fairly shallow, that is, to 20± ft, making removal easy and economical. Partial excavation may be possible in areas where the very soft surface deposit is either quite deep or is underlain by a significantly stronger material. Sometimes the soft materials are displaced by the weight of the fill, as shown in Figure 6-5 (see Chapter 4,

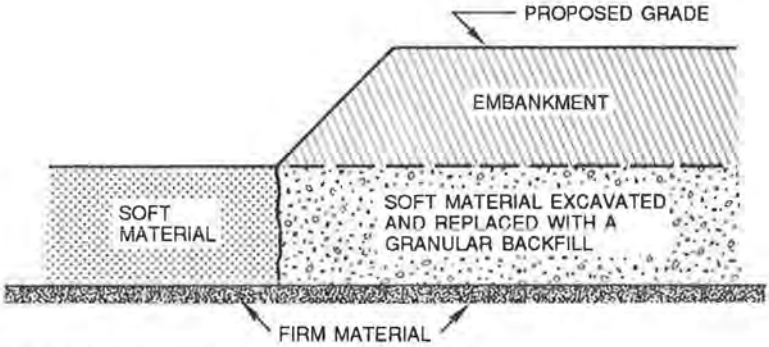


FIGURE 6-4 Complete excavation and replacement.

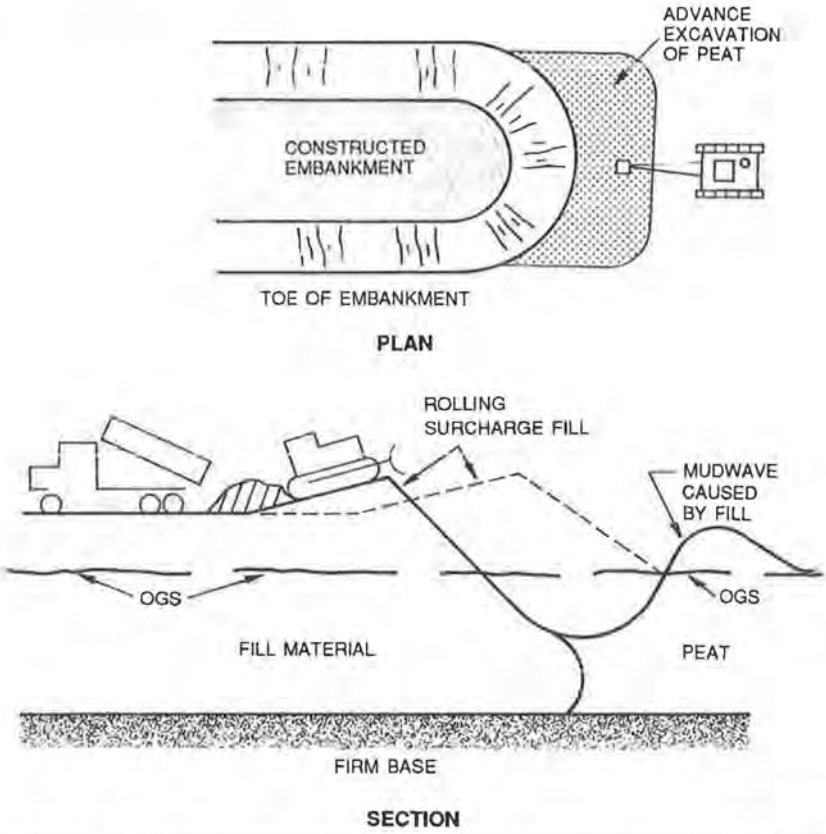


FIGURE 6-5 Gravity displacement method of fill using rolling surcharge and relief excavation at front (MacFarlane 1969; *reprinted with permission from University of Toronto Press*).

section on Unsuitable Materials). All these methods require very careful construction supervision and inspection. Close coordination with the geotechnical specialist is also necessary.

Stabilization by Consolidation

The basic concept of stabilization by consolidation is to force possible detrimental settlements that would otherwise occur after construction to occur during construction when they can be tolerated. This way, corrections can be made before opening the embankment to traffic. A temporary surcharge of additional fill material placed above grade combined with a waiting period causes more settlement in a given time period than would occur without a surcharge. With this procedure, the rate of embankment construction, including surcharge placement, is coordinated so that the surcharge is removed when field settlement and pore pressures equal the predicted values. The criteria given in the project soils report or in the special provisions are ordinarily based on the results of geotechnical instrumentation (Chapter 10) and surveys of line and grade. Although the additional cost of the surcharge fill is usually small, a surcharge may create potential stability problems in very soft foundations. Therefore, modifications in embankment design, such as slope flattening or berms, may also be required, as shown in Figure 6-6.

Also shown in Figure 6-6 are vertical sand drains or prefabricated "wick" drains, which are used to accelerate the consolidation settlements.

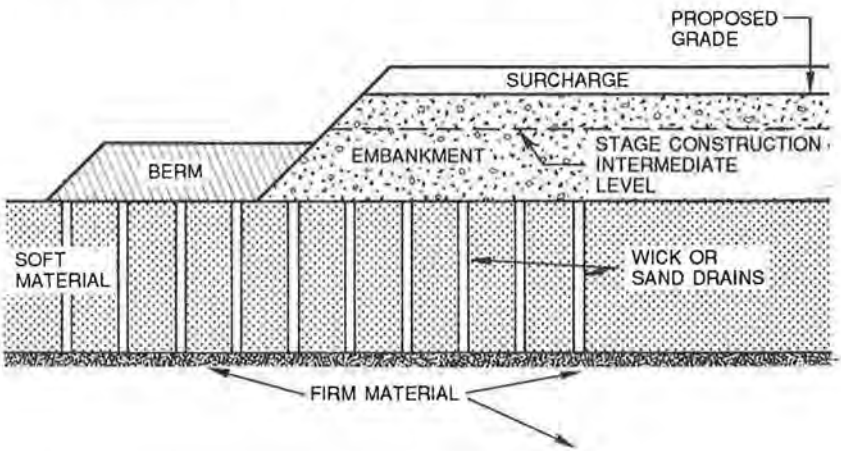


FIGURE 6-6 Stabilization by consolidation with a surcharge fill and wick or sand drains (*courtesy New York State Department of Transportation*).

Because the rate of pore water pressure dissipation increases as the square of the drainage distance decreases, vertical drains installed at typically 5- to 10-ft center-to-center spacings can dramatically reduce the time of consolidation. The corresponding soil strength increase that occurs with consolidation allows the embankment to be safely constructed, frequently in conjunction with stage construction of the fill. Again, these projects usually require monitoring with geotechnical instrumentation (Chapter 10).

Information on prefabricated vertical drains can be found in TRB (1986) Circular 309, and work by Rixner et al. (1986) and Holtz (1987).

Stage Construction with Delayed Paving

With programmed waiting periods between stages, the foundation soils can dissipate excess pore water pressure and settle without surcharge. Field instrumentation (Chapter 10) in the form of piezometers, settlement gauges, and optical survey stakes are required to monitor the foundation performance and regulate waiting periods. Criteria are ordinarily given in the project soils report.

Chemical and Physical Stabilization

Although most chemical and physical techniques have not been extensively utilized in the United States for highway embankments, they may be technically feasible and economical in some situations. Chemical stabilization techniques include lime and cement columns, grouting, electroosmosis, and thermal (heating, freezing) techniques. Physical stabilization and densification techniques such as blasting, dynamic compaction, vibro-compaction and vibro-replacement, jet grouting, and stone columns have been utilized occasionally and quite successfully at some highway sites. Figure 6-7 shows a schematic diagram of a stone column installation. Details on design and installation of most chemical and physical stabilization techniques can be found in work by Welsh (1987) and Holtz (1989).

Reinforcement

Reinforcement involves the inclusion of some type of reinforcing elements at the interface between the embankment and the ground to increase the stability of the embankment. The most common types of embankment reinforcement are geotextiles and geogrids, although bamboo and brush fascines or mats, corduroy, short sheet piles, tie rods, and

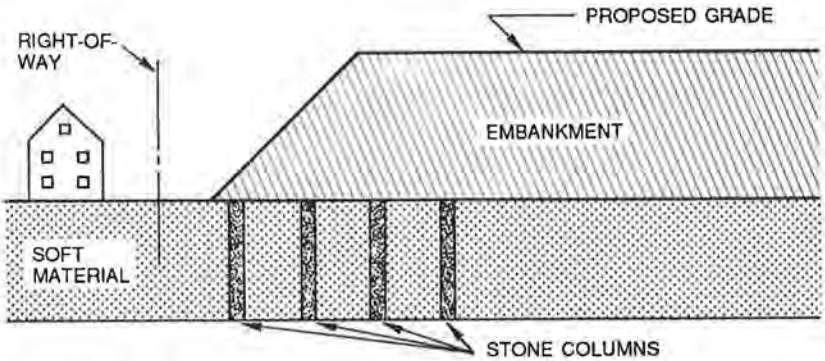


FIGURE 6-7 Stone columns used to stabilize highway embankment (*courtesy New York State Department of Transportation*).

the like have also been used. Common systems are shown in Figure 6-8. See *Geotextile Engineering Manual* (Christopher and Holtz 1985) and NCHRP Synthesis of Highway Practice 147 (Holtz 1989) for a discussion of the use, design, and construction of geotextiles to reinforce embankments on soft foundations.

Construction and Performance Monitoring

To ensure satisfactory construction and performance of the completed embankment, careful, competent inspection during construction is essential, especially for embankments in which some type of soil improvement and foundation treatment has been carried out. Visual observations and physical testing are obviously important components of construction inspection; perhaps not so obvious is that geotechnical instrumentation for taking measurements during construction is also an important aspect of construction monitoring. With a number of foundation treatments such as consolidation with vertical drainage, reinforcement, and chemical alteration, it may be desirable for foundation instrumentation and monitoring to continue for many years after construction is complete, especially if the particular treatment is considered experimental or if the stability of the site is marginal. Embankment instrumentation is discussed in Chapter 10 of this guide.

Inspection During Construction

The importance of well-trained, competent, and conscientious field and inspection personnel cannot be overemphasized. This is the only way to ensure that the essential features of the design are actually carried out in

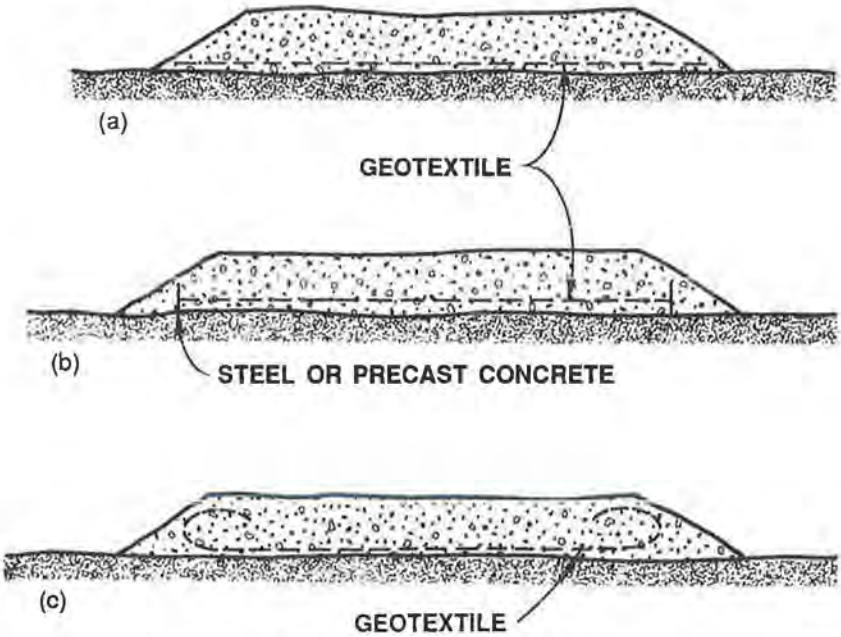


FIGURE 6-8 Concepts for using geotextiles to reinforce embankments on soft foundations (Christopher and Holtz 1985).

construction. With most, if not all, foundation improvement techniques, the success of the entire project is directly dependent on the success of the treatment, and competent inspection is the key element of the project.

To ensure that construction procedures for the treatment method are carried out properly, the designer should inform project engineers and field inspectors, by means of the project soils report and personal meetings prior to construction, about the important design concepts and key construction details of the treatment method. The on-site project engineer must be knowledgeable about the design assumptions to be able to make correct decisions about problems that will inevitably arise during construction. Uninformed construction decisions often result in cost overruns, contractor claims, or even failures.

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ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
FHWA	Federal Highway Administration

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7

Earthwork for Retaining Structures and Abutments

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Structural elements are often incorporated in earth embankments to retain or reinforce the soil mass. This chapter outlines the general design concepts and construction considerations for conventional earth-retaining structures and abutments as well as those with reinforced or mechanically stabilized backfills. Because such structures are often critical features of the highway system, details of design and construction that, if overlooked, may compromise their safety or reduce their life expectancy deserve careful attention.

CONVENTIONAL EARTH-RETAINING WALLS AND ABUTMENTS

Design Concepts

Conventional earth-retaining structures may be classified into two broad categories: rigid and flexible. Rigid retaining structures are commonly constructed of concrete or masonry. Rigid retaining structures used in highways include gravity, semi-gravity, cantilever, counterfort, and buttressed walls. All have occasionally been used for bridge abutments, some more commonly than others. Flexible retaining structures may be unbraced, as in the case of interlocking sheet piling, gabion walls, and crib walls. Alternatively, they may be braced or anchored, as in sheet pile walls, bulkheads, or tieback walls. Flexible walls are not commonly used for abutments. See *Foundation Engineering* (Peck et al. 1974) and *Design Manual 7.2* (U. S. Navy 1982) for information about the design of conventional retaining walls.

A properly designed conventional retaining structure must satisfy the following requirements:

1. The structural components of the wall or abutment must be capable of resisting the lateral earth pressures as well as any other loadings such as surcharges and hydraulic forces acting on it.
2. The wall or abutment must be safe against overturning and sliding at the base.
3. The foundation soil must have sufficient bearing capacity to avoid bearing failure due to both horizontal and vertical loads.
4. The wall and the soil mass it supports must be safe against an overall slip failure.
5. The structure must be able to tolerate the total and differential settlements caused by compression of the foundation soil.

All factors relating to the stability of a retaining structure are affected by the magnitude and distribution of the lateral earth pressures acting on the wall. In the design of rigid retaining walls with cohesionless backfills, it is standard practice to assume that the minimum or "active" earth pressure state exists because wall movements of less than 0.5 percent of the wall height are sufficient to mobilize the active earth pressures. On the other hand, movements less than this will result in greater earth pressures than assumed in design. Construction engineers need to remember this in case conditions occur during construction that effectively reduce or prevent wall movement. Examples of these conditions include the use of temporary bracing during backfilling and the discovery that the foundation of the wall is partially or entirely on bedrock instead of soil, as assumed in design. Abutments, on the other hand, are often designed assuming at-rest or greater earth pressures, especially if they are an integral part of the bridge.

The magnitude of the lateral earth pressures acting on the wall depends on the backfill, soil type, and placement density, as well as the compaction operations. Clean, free-draining granular soils should be used whenever possible. Backfills containing clay, silt, or organic matter are susceptible to swelling, shrinkage, creep, and frost action, all of which may cause excessively large earth pressures and detrimental settlements. For example, shrinkage cracks in clay may become filled with water and create undesirably large pressures against the wall. Particular care should be taken to prevent the use of swelling clays as backfill (see Chapter 9, section on Compaction Problems with Swelling Clays). Silt is also sensitive to moisture changes. Increase in moisture may cause a collapse of the soil structure and result in significant settlements. To reduce detrimental

settlements behind abutments supported on piles, some states use free-draining select granular material as backfill.

Partial or total submergence of a backfill results in an undesirable increase in the active thrust acting on the wall. Seepage pressures are one of the most common causes of retaining wall failures. Reduction of water pressures can be enhanced by the use of free-draining backfill and by providing effective drainage of the backfill. Unexpected surcharge loadings, including traffic and temporary construction loads, can also be very detrimental to the wall. Although it is ordinarily desirable to achieve good compaction of the backfill, heavy compaction equipment operating near the wall can induce lateral stresses on the wall much greater than the active earth pressures assumed in the design.

Construction Considerations

Walls are normally constructed by first erecting the wall and then backfilling behind it.

Excavations

To provide room for wall construction, it is common to over-excavate the soil back of the wall. Whenever an open excavation is needed, a safe slope or temporary shoring is required for the excavation. The maximum safe inclination of the slope depends largely on the shear strength of the soil, but the Occupational Safety and Health Administration (OSHA) requires that all trenches exceeding 5 ft in depth be properly shored.

If a retaining structure is constructed near a stream or river, the excavation may be below the groundwater level and special precautions are needed to protect the construction. Temporary flooding may leave soft muck in the bottom of the excavation that must be stabilized or removed before backfilling.

Control of Water During Construction

Surface water can cause erosion and deterioration of a slope, or even induce a slope failure. It can also reduce the capability of the soil to support structures or construction equipment. As discussed in Chapter 5, section on Surface Water, surface runoff should be directed away from the site during construction. In addition, surface runoff from adjacent areas should be prevented from encroaching on the site. The simplest way to

control surface water is to excavate a trench or construct a dike or curb around the perimeter of the site and dispose of the water by gravity or by pumping from sumps.

Retaining walls are sometimes constructed below the groundwater table, and dewatering may be required to provide a working platform (see Chapter 5, section on Subsurface Water). Although there are many methods available for this purpose (well points, horizontal drains, and the like), the simplest technique is to construct perimeter trenches and connect them to sumps. This method is most effective when the excavation is in cohesive material and the groundwater is not too high. The trench should be installed as far from the location of the wall base as practical to prevent disturbance due to groundwater seepage. In certain cases, impermeable barriers to reduce or eliminate the inflow of groundwater into the work site may be more effective than dewatering. Usually the selection of the method is left to the contractor.

Backfilling

Backfilling is generally the most important single aspect in the construction of walls. This is especially true when the space for compaction equipment is restricted. Inadequate compaction may cause excessive settlements or even failure of the structure. This is especially important when the abutment supports a spread footing foundation for a bridge (Cheney and Chassie 1982; Wahls 1983).

If possible, the backfill materials should be compacted at their optimum water content (Chapter 3). Backfill should be compacted in layers or lifts, which should slope away from the wall. The lift thickness depends on the compaction equipment and the backfill material, but typical lift thicknesses are 6 to 8 in. Thicker lifts may be used for coarser granular soils. When hand-held compactors are used, the loose lift thickness should be about 4 in. The recommendations of Chapter 4, particularly the sections on Compaction, Compaction in Confined Areas, and Structure Backfill, should be followed.

Constant supervision is necessary to obtain the proper lift thickness, especially in areas of limited working space. If the fill material is dumped in a pile and spread by hand, considerably thicker lifts than specified may result, leaving pockets of poorly compacted backfill behind the wall.

The specification for the gradation and density of the backfill should be adhered to strictly. Do not permit the contractor to substitute materials for the backfill without the prior approval of the engineer in charge. If clean granular backfill is specified, do not allow materials to be placed with clay or silt fines, organic materials, or any other material that does not meet the specifications.

The use of frozen materials in backfills is generally recognized as bad practice (see Chapter 4, section on Cold Weather Construction and Chapter 8, section on Frost Action in Embankment Design and Construction). Frozen backfill may look quite satisfactory when placed, but it can be extremely troublesome and totally unstable after it thaws. Care is needed during backfilling to prevent damage to any geotextile or geocomposite drains installed on the back of the wall or, in the case of anchored sheet pile walls, to tie rods. The soil in front of the toe and anchorage must also be adequately compacted.

Drainage

Conventional walls built above the groundwater table are normally designed with the assumption that no significant water pressures exist behind the wall. To ensure that this is the case, through-the-wall weep holes or a collector-drainage system, or both, are commonly provided and will be shown on the plans. Today a combination of granular drain materials and geotextiles, or a geocomposite drain, are commonly used (Christopher and Holtz 1985).

During installation, contamination of the drainage materials and system must be avoided. The drain outlet pipe, which connects to the drain, must also be carefully installed. Because proper drainage is very important to the long-term performance of the wall, all aspects of the drainage system construction should be carefully inspected.

To reduce percolation of surface water into the backfill, the site should be graded to direct runoff away from the back slope. Sometimes interceptor drains on the back slope are used (see Chapter 5, section on Surface Water). Periodic maintenance is also necessary to minimize runoff infiltration.

Scour

If a retaining wall is located adjacent to a stream or river, it is susceptible to scour during floods. Consequently, the erosion protection system is very important and must be constructed strictly according to the plans.

WALLS WITH REINFORCED BACKFILLS

Design Concepts

It is becoming increasingly common to use some type of tensile reinforcement in backfills behind retaining walls in order to reduce the earth

pressures acting on the wall face. A variety of reinforcing materials such as steel strips, sheets of geotextiles or geogrids, welded wire mats, metal grids or bars, and various anchor systems have all been successfully used for this purpose. Although soils are relatively strong in compression and shear, they are very weak in tension. By incorporating a material of high tensile strength in the soil, the composite soil mass will exhibit greater strength and be able to tolerate larger movements without distress. The mechanisms of reinforcement for the different types of materials have been summarized by Mitchell and Villet (1987) in NCHRP Report 290. The reinforced retaining structure must satisfy both external and internal stability requirements. For external stability, all the requirements described in the section on Design Concepts for conventional retaining walls must be met. Internal stability must satisfy two criteria: (a) the tensile reinforcement must not break, and (b) there must be sufficient friction or bonding between the soil and reinforcement so that it does not pull out from the backfill. Many of the reinforcing systems commonly used today are proprietary, and designs and contract specifications are often prepared by the individual material suppliers or contractors.

The inclusion of tensile reinforcement in permanent highway structures requires that the reinforcement be sufficiently durable throughout its design life. Examples of problems include creep and chemical degradation of geosynthetics and the corrosion of metals.

Construction Considerations

Earthwork construction control for reinforced structures is essentially the same as that required for conventional retaining structures, but with a few additional details that require special attention. Several of the proprietary firms have published quality control procedures and manuals (for example, Reinforced Earth Co., 1987). The contractor should obtain a copy from the company or the design engineer and follow the recommendations as closely as possible. Field substitutions of backfill materials or changes in construction sequence, procedures, or details should *only* be permitted with the express consent of both the responsible geotechnical or preconstruction design engineer and the proprietary system material supplier.

Site Preparation

Before placement of the reinforcement, the ground should be graded to provide a smooth, fairly level surface. The surface should be clear of

vegetation, large rocks, stumps, and the like; depressions should be filled; soft spots should be excavated and replaced with backfill material; and the site should be proof rolled (see Chapters 4 and 6).

With reinforcing systems utilizing precast concrete facing panels, a small strip footing is commonly employed as a foundation under the facing panels.

Handling of Reinforcement Materials

Specific material-handling instructions for proprietary reinforcement materials are generally provided by the individual material suppliers. Geosynthetics, especially geotextiles, should be protected from sunlight and extreme temperatures. Concrete facing panels should be handled carefully to prevent cracking and chipping. Damaged or improperly handled reinforcing materials should be rejected.

Placement of Reinforcement Material

After the reinforcement is in place, it should be examined carefully. Any damaged or torn materials should be removed or repaired as detailed in the specifications. In no case should construction equipment be allowed to operate directly on any reinforcement before fill is placed. In the case of geosynthetic reinforcement, it should be unrolled transverse to the alignment of the embankment or wall, and wrinkles and folds should be eliminated. Procedures for seams and overlaps detailed in the plans and specifications should be adhered to strictly.

Fill Placement and Compaction

Special attention should be given to ensuring good compaction of the backfill, especially near the face of the wall. Otherwise detrimental settlements behind the face may cause a downward drag on the reinforcement, which might induce excessive tensile stresses, particularly near the face where reinforcements are attached to concrete panels (see Chapter 4, sections on Compaction in Confined Areas and Structure Backfill).

At the end of each day's backfilling operation, the last lift of fill should be sloped away from the wall facing to direct any possible runoff away from the face.

Alignment of Facing Panels

Alignment of the structure is usually established by initial layout of the foundation wall section and strip footing, if required. In addition, some type of external bracing, formwork, or scaffolding, usually erected in front of the wall face, is often used to maintain the alignment of especially the first lift. For all reinforced structures, particularly for those that do not use precast concrete facing panels, care should be taken not to allow heavy construction equipment to operate too close to the face. Otherwise undesirable bulging of the face may result.

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ABBREVIATION

FHWA Federal Highway Administration

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8

Environmental Considerations

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This chapter discusses some of the environmental issues commonly encountered during construction operations. It is a brief overview and does not include all the possible environmental issues that may be encountered on a specific project. Fortunately, most environmental construction problems are anticipated during the design phase and are adequately addressed in the contract documents and specifications. However, specifying how a contractor shall operate to withstand the forces of nature and getting the contractor to actually comply with the contract and achieve the project objectives can be a very difficult and drawn-out process. For example, prevention of erosion during and shortly after construction is particularly difficult. Achieving satisfactory results requires diligence and cooperation by both the contractor and the construction engineering team.

This chapter discusses general and site-specific environmental considerations, soil erosion, the influence of construction on the local environment, hazardous and objectionable materials, long-term issues and considerations, and frost actions.

GENERAL ENVIRONMENTAL CONSIDERATIONS

It is important to recognize environmental problems and know how to solve them in a prompt and effective manner. Failure to properly identify

and mitigate environmental problems often leads to construction delays and increased costs. Environmental problems rarely go away by themselves and usually get worse with time. A severe environmental problem, such as malodorous or highly toxic waste material, could stop construction, and in an extreme case, even force the project to be abandoned. In recent years, growing public concern about the environment has exerted considerable influence on some construction projects. This trend is likely to continue.

SITE-SPECIFIC CONSIDERATIONS

Each construction site has its own unique environment. Some of the items that must be taken into account before the start of construction include the immediate local surroundings (schools, parks, airports, industrial plants, and the like) and special local events (county fairs, races, music and art festivals, and so on). Another consideration is the potential effects of construction on the local environment: strong winds blowing soil and dust, heavy rains causing temporary flooding and silting of streams and drainage structures, and excessive traffic delays due to construction operations.

Once potential problems have been identified, solutions and procedures should be incorporated into the contract.

Recognize the specifics of the local climate. For example, in areas of significant snowfall, recognize that drifting snow can potentially clog temporary drainage channels. Vegetation should be planted during a period when there will be adequate moisture, heat, and light. The plans and specifications generally cover project landscaping requirements. If questions arise, consult the resident engineer.

SOIL EROSION

The most common environmental problems encountered in construction are erosion by wind and water. Normal embankment construction activities require destroying the natural vegetative cover and the natural balance of environmental forces near the ground surface. The best way to prevent local problems is to reestablish the natural vegetative cover and natural environmental balance as quickly as possible after construction. There are several methods that can be used, and combinations and variations to fit local site conditions are usually the most effective. A number of new products are available that can help reduce the effects of wind and water during construction (see Chapter 5, section on Surface Water). Although some of these products can be very effective in local

areas, generally they are quite expensive. Construction erosion may often be more economically taken care of by controlling the contractor's operations so that environmental damage is avoided. Be careful not to order the contractor to use various erosion control measures unless the items are specifically mentioned in the contract. Consult the resident engineer if there are any questions.

Many states specify in the contract documents the size of the area that may be opened to construction at any one time before the permanent erosion control treatment is applied. The contractor should not be permitted to destroy the natural vegetative cover on extended lengths of the site unless erosion control and prevention measures can be installed at about the same time. The contract may require the contractor to submit an erosion control plan. State and federal soil conservation agencies can often provide valuable information and guidelines. As part of each contract, many states include standard plans for temporary erosion control features and/or devices.

Water Erosion

As noted in Chapters 4 and 5, one of the first requirements of construction is to prevent damage to local water bodies by not permitting the construction runoff water to mix with any local stream, lake, or other nearby water body. This can best be achieved by reducing soil erosion caused by surface runoff where possible and by preventing unavoidable soil erosion from leaving the construction site.

Normal rainfall will cause erosion of exposed soil, if not protected. During construction, it is nearly impossible to prevent rainfall from eroding the work area. Therefore, to protect the environment, all the runoff and all the eroded soil must be kept on the project site until the sediment can be removed from the runoff water. Hay bales, sedimentation basins, and silt fences have been used effectively to protect local streams and water bodies. Such installations are generally shown in the standard plans or elsewhere in the contract. Refer to Chapter 5, section on Surface Water for a detailed discussion of these installations.

If roadway construction is in lakes or open water, use of sheetpiling or properly designed silt curtains has been very effective in keeping construction runoff from contaminating the open water. Such measures are incorporated into the project by designers and ordinarily are not left to field forces to develop.

Methods for controlling soil erosion should not be used in ways that contribute to other problems. For instance, while the use of mulching is very effective in holding seed in place until it germinates, the type of mulch must be appropriate for the area; otherwise a heavy rainfall could

easily transport the mulch into local drainage channels, causing flooding and other damage.

Wind Erosion

When strong winds blow across unprotected land, they move the soil. Long-term wind erosion can be prevented by planting ground cover and establishing vegetation. The short-term local effects of wind erosion must be handled on a temporary basis by the contractor. One of the major contributors to wind erosion is construction traffic, which may lift large quantities of silt-sized particles high enough so only light breezes are needed to transport them great distances. Damage to crops and other vegetation, local residences, and vehicles may result.

To prevent excessive construction dust in construction traffic areas, it is often necessary to use temporary pavements or chemical palliatives such as salt, calcium chloride, asphalt emulsion, tar, and numerous other chemicals. All dust palliatives are potential environmental contaminants. For example, salt is known to damage local vegetation, and it is undesirable for it to go directly into a water supply. Therefore, all proposed palliatives must be thoroughly evaluated before they may be used on construction projects. Most agencies usually have a list of acceptable dust palliatives for the contractor's use.

INFLUENCE OF CONSTRUCTION ON THE LOCAL ENVIRONMENT

Construction activities impose conditions on local environments that can be perceived by the public as ranging from slightly objectionable to completely intolerable. Controlling off-site trucking routes and limiting the hours of their use can often minimize inconvenience to the public caused by noisy truck traffic and soil spillage onto roadways. Close cooperation among the project engineer, local law enforcement officials, and contractor's personnel can usually prevent serious problems of this nature. Restrictions placed on the contractor's operations must, of course, conform to the contract.

HAZARDOUS AND OBJECTIONABLE MATERIALS

Although it is not uncommon to encounter waste materials from industrial operations in rural areas, they are much more likely to occur in urban excavations. Waste materials can vary from garbage dumps and paper

mill wastes to mine and steel-making wastes (see Chapter 9, section on Waste Materials). Most of these wastes have concentrations of chemicals that can be harmful to the environment if not adequately handled. The excavation may even release gases that are harmful to humans, including the equipment operators and inspectors. Sometimes the gases have such noxious odors that they must be specially treated.

If there is any suspicion that the contractor has uncovered an area that might be environmentally obnoxious or hazardous, stop all operations and do not allow the contractor to transport the materials until a complete environmental assessment can be made. There are severe civil and criminal penalties if hazardous or contaminated materials are knowingly transported without proper permits, notification, and safeguards.

Occasionally the contractor may want to use waste products as part of the embankment construction (see Chapter 9, section on Waste Materials). Local industries are sometimes willing to pay contractors to haul away waste materials. Before accepting any of these materials on a project, have a complete evaluation made of the chemical constituents and of the stability of the materials when subjected to the local environment so that a long-term problem is not created. The contractor must also be alert to hazardous wastes coming from its own construction activities, for example, lead-based paint removed as part of the cleaning of bridges or asbestos from the demolition of old buildings. In these cases, applicable environmental laws must be strictly followed.

LONG-TERM ISSUES AND CONSIDERATIONS

Usually items that may have a long-term influence on the local environment are included as part of the contract documents. The engineer should be aware of the purposes of these long-term controls in the contract and make certain that the contractor's short-term activities do not negate the intent of any long-term environmental controls.

During construction, the engineer must be constantly aware of the need to prevent contaminants from reaching local water supplies. In order to grow vegetation on exposed soils, however, it is usually necessary to add fertilizers, lime, or other nutrients to the soil. These chemicals can cause concentrations that are very damaging to the local environment if they are improperly stored or misapplied. Certain contractor-produced wastes can cause permanent damage to the environment if not adequately controlled. The contractor's work yard needs to be constantly monitored for spills of fuel, oils, and chemicals, which should immediately be cleaned up. Crankcase oils, for example, should be collected and disposed of properly.

Unacceptable discharge from the project can produce permanent long-term damage to the environment if not adequately controlled. It may be necessary to separate runoff from the completed highway from local water supplies by salt berms, intercepted drainage, and other measures. However, when certain natural materials that are excavated and/or removed from their original location and used as construction materials are exposed to rainfall, harmful chemicals can be leached from them. These possibilities should be considered during the design phase and appropriately included in the plans and specifications. However, the engineer must be aware that any earthwork will be subjected to rainfall infiltration and possibly other materials, such as salts and oils, that might cause corrosion of structures under construction.

FROST ACTION IN EMBANKMENT DESIGN AND CONSTRUCTION

In areas where freezing temperatures occur, embankments and the facilities they support, such as pavements, rail lines, and so on, may be adversely affected by the freezing of the soil. This important factor has to be considered in the design of the project. The soils investigation should be done early in the location phase to properly prepare the specifications if problem materials exist. These problems should be noted in the contract documents so that the bidders are aware of unusual circumstances, requirements, disposal of unusable soils, and so forth.

Frost Effects

Frost heave is the upward expansion that occurs when certain soils freeze. Not only does the in situ moisture freeze, but more importantly, additional moisture flows to the freezing front. While the expansion of pore water upon turning to ice does contribute to heave, it is the additional water that moves to the freezing front that causes the dramatic heaving of road surfaces and railroad tracks (TRB 1974). Because heaving is seldom uniform over the area affected, road surfaces become very rough, resulting in cracked or damaged pavements. The distortion of road surfaces also leads to disruption of surface drainage.

Thaw settlement and instability occur with the advent of warmer temperatures in the spring. Thawing occurs from the surface downward, with the result that moisture is trapped between the surface and the still-frozen soil below. This results in greatly reduced bearing capacity. While most modern high-volume roads can withstand this condition, the typical pave-

ment thickness on low volume or older roads is insufficient, and seasonal load restrictions may have to be imposed. At the end of the thaw season, the surface may not settle back to its original level, so there may be residual pavement roughness (TRB 1974; Armstrong and Csathy 1963). Permanent rutting often results in water ponding, leading to the danger of hydroplaning.

Damage to and disutility of buried facilities is another possible hazard. Culverts may be heaved either temporarily or permanently, resulting in sharp bumps in the road, disrupted surface drainage, and eventual complete destruction of the culvert (Fredrickson 1963). Stormwater drains and outfalls for subgrade or pavement underdrainage systems may be rendered ineffective as well.

Mechanism of Frost Heaving

Three factors are necessary for detrimental frost heaving to occur (TRB 1974; Linell et al. 1963): (a) freezing temperatures for a sufficient duration of time, (b) a water supply sufficient to support the growth of ice lenses, and (c) a frost-susceptible soil, that is, one with a texture favorable to the upward movement of water to the freezing front.

Design to Control Frost Heaving

If any one of the above three factors can be eliminated during design, frost heave and instability during the thaw period can be essentially eliminated.

A soil subgrade, subbase, or base course material is considered to be frost susceptible if it contains more than 3 percent by weight of particles smaller than 0.020 mm (Linell et al. 1963). For convenience and practicality, and depending on local soils and climate, many frostbelt states specify an equivalent limit, typically ranging from about 8 to 15 percent of the maximum allowable percent passing the No. 200 sieve. Therefore, if any frost-susceptible soil within the depth of frost penetration can be removed and replaced with non-frost-susceptible soil, heave can be largely prevented. Alternately, it may be more effective to simply cover the frost-susceptible soil with a sufficient thickness of select non-frost-susceptible soil. This has the effect of raising the gradeline above natural ground, which often has side benefits in drainage and in areas of heavy snowfall.

The availability of moisture depends primarily on the permeability and the height of capillary rise of the subgrade or embankment soils. Some

agencies consider that if the water table is more than 10 ft below the surface, frost will present little problem. On the other hand, an effort is made to keep the gradeline at least 5 ft above the water table, depending on the specific soils at the site, and to use select granular material to construct the embankment (Erickson 1963). It will be noted that this results in an ambiguity where the gradeline is from 5 to 10 ft above the water table. Engineering judgment is required in this case.

Cuts are likely to be troublesome, especially when they are deep enough to approach the groundwater table. The cut will typically drain toward the cut-to-fill transition, and thus localized heaving problems can be created unless effective drainage is provided to keep the seepage out of the fill (TRB 1974; Armstrong and Csathy 1963).

It might seem that freezing temperatures would not be amenable to treatment, as they are inherent to the site. However, there has been considerable success in controlling frost heave by the use of foamed insulation boards placed on frost-susceptible subgrades. They are then covered by a foot or so of base course and pavement materials. Unfortunately, there have been cases where differential icing of the pavement surface occurred as a result of the interruption of the heat flow from the earth. As a result, many early installations of thermal insulation were removed (TRB 1974). At present there is a renewed interest in the use of insulation. Concerns for liability still remain, however.

Other Design Considerations

In general, frost penetration will be greater (although frost heave will be significantly less) for free-draining cohesionless soils, dry soils, and soils of greater dry density than for silts, clays, and wet soils. The project should be reviewed to determine the relative frost-susceptibility of the various soils that may become part of the embankment. Selective placement should be considered so as to place the least frost-susceptible soil nearest the top of the completed embankment (NCHRP 1974). If clean sands or gravels are conveniently available, they should be used to facilitate the drainage of the embankment.

Cold Weather Construction

Since basically all soils will heave when frozen, some more than others, uniformity of the soil in the embankment is an important objective of the contractor and construction inspectors. Having a uniform embankment will help eliminate differential heaving when freezing occurs. Methods to

accomplish this are discussed in Chapter 4, sections on Embankments and Compaction.

Care should be taken during construction to see that design features intended to counteract frost action are not altered by construction operations. For example, clean cohesionless materials used at the top of the embankment may sometimes be difficult to traffic by earthmoving equipment, and there may be pressure to add a little cohesive material to correct the "problem." This, of course, would be counterproductive to the frost-prevention purpose of the materials.

In general, construction during winter or freezing temperatures is avoided by most highway agencies. As noted in Chapter 4, section on Cold Weather Construction, it is very difficult to compact frozen soil to the densities which are typically specified, with the result that when spring comes, there is excessive settlement, especially differential settlement, of the embankment.

Where the work involves rock fill or clean, free-draining sands or gravels, however, and the work can be planned to avoid using frozen soil, it is possible to obtain satisfactory results (Haas 1988; Haas et al. 1988). The key is to prevent the soil from freezing before it is compacted and to suspend operations if it becomes too cold to do so.

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9

Special Soil Deposits and Embankment Materials

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A number of soil deposits, both natural and man-made, can pose difficult problems for embankment foundations. These materials may be difficult, if not impossible, to sample and test by ordinary means, and thus their engineering behavior is not well documented or well understood. Consequently, normal practice, design rules, and experience may not lead to satisfactory embankment performance. A number of these special deposits are good candidates for some type of soil improvement and foundation stabilization, as discussed in Chapter 6, so that highway embankments can be constructed on them safely and with tolerable settlements. When used as fill directly in the embankment, these materials may cause unusual problems for the contractor and therefore for the field engineers and inspectors. Such materials may be stabilized and improved in special ways, some of which are mentioned in this chapter. Generally, such deposits and materials are handled by special provisions in the project specifications.

Deposits and materials discussed in this chapter include landfills, dumps, wastes from industrial and mining operations, lightweight fill materials, shales, swelling clays, and collapsible soils.

WASTE MATERIALS

Sanitary Landfills and Dumps

Roads in urban areas frequently must be located on sanitary landfills, garbage dumps, and similar areas. Construction is certainly possible on sanitary landfills, as shown by Moore and McGrath (1970) and Chang and Hannon (1976), but the results are often less than satisfactory unless some special foundation treatment is carried out. As noted by Holtz (1989), the types of problems encountered at such sites depend on the nature of the landfill materials and their age, both of which may be highly variable. Embankments on some well-operated landfills will normally consolidate rapidly, and thus only a simple surcharge is required to adequately densify them. In other instances, embankments on loosely dumped municipal garbage and building wastes will experience very large total and differential settlements, and this may mean a poor riding surface and high maintenance. Landfills that are 15 to 30 yr old may have already decomposed sufficiently to be good candidates for foundation treatment, although they may contain wastes that cause them to be considered hazardous by current Environmental Protection Agency (EPA) standards.

Feasible methods of foundation treatment (see Chapter 6) include the following:

- Proof rolling with very heavy rollers,
- Surcharge,
- Excavation and replacement as a compacted fill,
- Embankment piles,
- Grouting,
- Vibrocompaction, and
- Dynamic compaction.

Many waste dumps are not controlled landfills as described above, but are sites such as swamps, tidal flats, river and stream banks, lakes, and so on, where garbage, used appliances, wrecked cars, used tires, and the like have been often illegally discarded. In addition to the nature of the waste materials, the type and condition of the natural soils in the area must be considered in evaluating the site for possible foundation treatment.

Landfill sites pose other problems during construction. Decomposition of municipal wastes generates methane and carbon dioxide, and the introduction of fresh air into a dump site could cause a fire by spontaneous combustion or even from smoldering material buried in the landfill. Difficulties have been experienced with noxious gases, and in such cases it has been necessary for the field personnel to use breathing apparatus,

apply deodorants to the site, and exercise special rodent and pest control measures after the area was opened.

Suspected toxic or hazardous waste dumps pose especially serious problems if they must be crossed by the highway. Special precautions must be taken to protect field crews, and it is prudent in these cases to call for specialized help.

Inorganic Industrial Wastes and Dredged Materials

Other wastes that are sometimes of concern to highway engineers include industrial by-products and wastes such as slags, bottom and fly ashes, and inorganic sludges. Dredged materials are sediments dredged from the bottoms of river channels, lakes, and harbors and deposited on land in diked containment areas. These waste materials are usually encountered in very localized areas, often near their source, although dredged materials and sludges may be transported some distance as slurries.

Loose deposits of predominantly granular materials such as slags and bottom ashes can be treated by methods appropriate for such materials (dynamic compaction, blasting, vibroreplacement). Provided proper environmental constraints are followed, they should make excellent embankment fills. Fly ashes are rarely foundation problems, and since they are mildly pozzolanic, they should be more than acceptable fill materials provided they are properly handled during transport, water addition and mixture, and compaction.

Sludge deposits and dredged materials, which may be silty or clayey or even somewhat organic, usually are a problem because of their high water content and compressibility. Holtz (1989) suggests acceptable foundation stabilization methods for these materials. Rarely do they make good fill materials.

Strip Mined Areas, Mine Wastes, Tailings, and Slurry Ponds

Both surface and underground mining operations usually leave rather unusual deposits and conditions that may cause locally difficult problems for embankment foundations. In addition, mineral processing operations also produce wastes in the form of tailings and slurries (slimes) that, if encountered, may be difficult to stabilize for construction.

Strip and underground mining operations often leave large areas of loosely dumped spoil materials. For embankment foundations, these deposits may be suitably treated (Holtz 1989), and in some cases may make excellent embankment fill.

Tailings from some mineral processing operations are another matter. They can be extremely difficult to stabilize for foundations, depending on their grain size and water content. Those factors, plus potentially hazardous conditions, for example, the presence of radioactivity, heavy metals, cyanide, or organics, make the use of tailings for highway fills very problematic. If such materials are suspected on your job, be sure that they meet all environmental requirements prior to approving their use as embankment fill.

LIGHTWEIGHT FILL MATERIALS

Both the stability and settlement of embankments on soft foundations can be improved by use of lightweight embankment fill (Moore 1966; Holtz 1989). Lightweight materials that have been used successfully in highway embankments include bark, sawdust, dried peat, fly ash, slag, cinders, cellular concrete, expanded clay or shale, expanded polystyrene, and oyster and clam shells. The advantages and disadvantages of the use of these materials are discussed by Holtz (1989).

Because the crushing strength of some lightweight materials is relatively low, care must be taken during construction to avoid damaging them, especially if conventional compaction equipment is used. Sometimes encapsulation is required for environmental reasons, and both synthetic liners (geomembranes) or compacted clay have been successfully used. In either case, great care must be taken during placement of the liner to avoid punctures, tears, and leaks. Strict adherence to the placement specifications is essential in these projects.

CONSTRUCTION OF EMBANKMENTS OF SHALE

The materials given the generic classification of "shale" are geologically widespread, and are frequently encountered in excavation and borrow situations. Two major problems have occurred when these materials have been used in highway embankments. Where the shales contain swelling clay minerals, the fills display the characteristic volume changes associated with swelling clays (see section on Compaction Problems with Swelling Clays). A somewhat more subtle problem situation occurs with the use of shales that are physically nondurable but are strong and rocklike when freshly excavated. Such materials have often been placed as rock fills, only to experience breakdown in service, producing excessive settlements and even slope failures.

This section concentrates on the technology required when building fills of hard but nondurable shales. These materials are commonly encountered throughout the midwestern United States, and thousands of examples of unsatisfactory performance have occurred where they were improperly placed.

Early classification of these materials is recommended, and the Franklin (1981) approach is appropriate. The primary test in this approach is the slake durability test, which combines two wet/dry cycles, with a rotational impact that dislodges slaked portions from the shale aggregates. The test is standardized as ASTM D 4644, Standard Test Method for Slake Durability of Shales and Similar Weak Rocks.

Once the second cycle slake durability index, I_d (2), is defined, it serves as a general guide for relative durability and also determines the second test required to accomplish the Franklin classification. If the I_d (2) is equal to or less than 80 percent, then a soil test such as the Atterberg limits and the plasticity index can be used to classify the material. On the other hand, if the I_d (2) is greater than 80 percent, the point load strength index, adjusted for an aggregate dimension of 50.0 mm, must be used to complete the classification. All these procedures are briefly described in Oakland and Lovell (1983), and in greater detail in Oakland and Lovell (1982).

If the shale is nondurable and yet strong and hard, it is advisable to conduct a compaction-degradation test on it (Hale et al. 1981). A nondurable material must be intensely degraded during excavation, placement, and compaction, and it must be finally densified to a specification appropriate to a similar soil. This is difficult to accomplish with some shales, but the compaction-degradation test allows the problem to be anticipated.

The testing procedure, also described in Oakland and Lovell (1982), produces a numerical value, termed the index of crushing, which is the percentage reduction in mean aggregate size, produced in the laboratory compaction process. If this number is relatively high, for example, greater than about 40, the shale will be easily degraded in the field. If it has a lesser value, the shale will strongly resist efforts to break it down, and special wetting and heavy rolling procedures may be required. The procedures and compaction specifications for compacted shales are best developed in a full-scale field test pad, and the results of such tests should be made available to the project engineer. Special wetting and compaction procedures, if required, will be detailed in the special provisions of the project specifications.

Strom et al. (1978) and Strom (1980) have written good references on the design and construction of shale embankments.

COMPACTION PROBLEMS WITH SWELLING CLAYS

Compaction problems with swelling clays require special attention. Swelling soils, which are frequently clays but are sometimes shales, marls, or other soils, cause an estimated $10 billion in damage in the United States every year (Krohn and Slosson 1980). Half of this damage occurs to the nation's highways, with most of the remainder occurring to other transportation facilities such as airport runways, railroads, canals, pipelines, sidewalks, and so forth. Swelling materials occur in all but six states. The problem of swelling soils has been studied with considerable intensity through the years. One of the major efforts was a $700,000 research project funded by the Federal Highway Administration and conducted by the U. S. Army Corps of Engineers, Waterways Experiment Station, (Snethen et al. 1975; Snethen 1979 and 1980). Other major research has been done for the U. S. Air Force (McKeen 1980), and a variety of state agencies (for example, Watt and Steinberg 1972; Steinberg 1985). Many of these studies have been published by TRB (1981; 1985).

Once the contractor is aware of the potential of a swelling soils problem, standard Atterberg limits laboratory tests should confirm whether indeed there is a problem. From there on, the best advice is to avoid overcompacting the material. Density testing is a significant help in this regard. Keeping the material at a moisture level dictated by the density curves will assist in reducing the likelihood that the swelling material will turn the finished project into a roller coaster track in a few years. Properly compacting materials identified as having swelling potential and avoiding overcompaction are initial steps only. When embankments are constructed with swelling materials, the results tend to be satisfactory over an extended period of time, certainly much more so than when dealing with swelling clays in an excavation area.

Because the problem of expansive soils is an international one, it is reassuring to know that several solutions have been tried and found to be successful. Lime treatment has been used successfully both in the United States and abroad (TRB 1987). The important thing to remember is that enough lime should be used and that it should be placed to a depth that will control the potential movement. (Potential vertical rise tests will give an indication of what these depths might be.) Electro-osmotic chemical stabilization and pressure injection of chemicals, primarily lime, have been used, but with mixed results.

The key to the successful mitigation of the effects of swelling soils seems to be in minimizing moisture variation underneath the structure, be it a pavement, building, runway, track, or whatever, to prevent the destruc-

tive movements from taking place. Moisture barriers have been tried in several locations. Examples include pressure-injected lime barriers, deep vertical fabric barriers, and horizontal geomembranes. These tests have been reported by TRB (1981; 1985) as well as state transportation agencies (Steinberg 1985). Ponding has also been used in several instances to solve earthwork construction difficulties with swelling materials. Watt and Steinberg (1972) drilled holes 20 ft deep, backfilled them with a pervious material, and then ponded water in them for 30 days. This procedure produced sections that have not had to be replaced because of subgrade problems.

Studies are continuing on swelling soils, and to minimize the damage these soils cause transportation structures and facilities, the engineer should be aware of the results of this research.

COLLAPSING AND SUBSIDING SOILS

Collapsing soils undergo a very large decrease in volume if their water content increases significantly, even without an increase in surface load. Examples include loessial soils, weakly cemented sands and silts, and certain residual soils. All these soils have a loose, open, "honeycomb" structure, in which the larger bulky grains are held together by capillary films, montmorillonite or other clay minerals, or soluble salts such as halite, gypsum, or carbonates. Loess is, of course, wind-deposited; other collapsible soils are found on flood plains and in alluvial fans as the remains of slope wash and mud flows, colluvial slopes, and some residual soil deposits. Many, but not all, collapsible soil deposits are associated with dry or semi-arid climates, but some dredged material deposits and hydraulic fills can also be collapsible.

Treatment methods for collapsible soils depend on the depth of treatment required. For modest depths, compaction with rollers, wetting or inundation, and overexcavation and recompaction, sometimes with lime or cement stabilization, are used (Bara 1978). Dynamic compaction (Lukas 1986) may also be feasible. For thicker deposits, ponding or flooding are ordinarily very effective, as is dynamic compaction. However, explosives, displacement piles, and vibroreplacement-vibrocompaction methods could possibly be used as well. Design information for the deeper stabilization methods is given by Clemence and Finbarr (1981) and summarized by Holtz (1989). Any of these procedures required would be detailed in the special provisions of the project specifications.

LOOSE SATURATED SANDS IN EARTHQUAKE COUNTRY; FLOW SLIDES

It is possible for deposits of loose, saturated granular materials to lose all strength when subjected to shock or vibrations from, for example, blasting, pile driving, or earthquakes. The phenomenon is called liquefaction, and it results because there is a tendency for loose sands to decrease in volume when strained or shocked. This tendency causes a positive increase in pore water pressure which results in a decrease in effective stress within the soil mass. Once the pore pressure becomes equal to the total stress, the effective stress becomes zero, and the soil mass loses all its strength (Holtz and Kovacs 1981). Because this loss in shear strength is sudden, the effect on highway embankments and other structures supported by such deposits is disastrous.

Flow slides are a type of liquefaction that occurs almost spontaneously in loose deposits of fine sands often found on the banks of large rivers. When these deposits are strained, say by erosion at the river's edge, excess pore pressures can develop which can lead to liquefaction and collapse of the deposit.

Because of the potential for catastrophic collapse of the foundation of an embankment on liquefiable sands, it is important that these deposits be identified and treated before construction. Virtually all the methods described by Holtz (1989) for granular materials are appropriate for densifying or stabilizing such deposits. Particularly attractive are dynamic compaction, blasting, vibrocompaction and replacement methods, relief wells and drains, and excavation and replacement. These procedures are quite specialized and would be given in the special provisions of the project specifications.

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ABBREVIATIONS

ASCE	American Society of Civil Engineers
FHWA	Federal Highway Administration

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Instrumentation for Embankments

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On many highway embankment construction projects, geotechnical instrumentation is an essential tool for monitoring the performance of the foundation and the embankment. This is especially the case if some type of special foundation treatment is employed (Chapter 6) or special soil deposits (Chapter 9) are encountered. The instrumentation and monitoring program is primarily used to alert geotechnical and construction engineers to soil behavior or construction problems different from those anticipated in design. For instance, adverse soil behavior may call for a reduction in the rate of embankment construction. Alternatively, encountering soil behavior much better than that assumed in design may allow increased construction loading rates, steeper embankment slopes, and elimination or reduction of recommended special foundation treatments. Instrumentation is useful for determining the rate of strength gain and degree of consolidation of the foundation soils. It can also indicate conditions of impending failure.

This chapter is intended to provide construction personnel with an overview of geotechnical instrumentation and an appreciation of its importance to the overall success of the embankment construction. Depending on the project, department practice, and contractual arrangements, the instrumentation may be installed by the contractor or a specialty subcontractor. In this case, field personnel will be responsible for inspecting the installation of the instruments. In all projects, field personnel are responsible for monitoring the contractor's operations so that the instru-

mentation, which is expensive and important, is properly protected during construction. On many projects, construction engineers and technicians are required to periodically read the instruments and to report the values to the appropriate project supervisor or geotechnical engineer. This is a very important responsibility, and it must be done diligently and carefully.

OVERVIEW OF GEOTECHNICAL INSTRUMENTATION

Information on Instrumentation

The notes for the FHWA training course on geotechnical instrumentation (Dunnicliff and Sellers 1980) are probably the best overall reference on the subject for highway construction. Dunnicliff (1982) gives a useful summary of the training course notes. Hanna (1985), Dunnicliff (1988), and Wilson and Mikkelsen (1978) have written good general references on geotechnical instrumentation.

When Is Instrumentation Used?

Instrumentation is used under the following circumstances:

- When a foundation failure could be expensive, life threatening, or damaging to adjacent property;
- When the design dictates waiting periods or controlled rates of loading;
- When new methods of foundation treatment or unusual embankment materials are being used;
- When the embankment is expected to settle greater than 2 ft; or
- When information gained by instrumenting the first sections of a large project that is being designed and constructed sequentially can be used to improve the design of subsequent sections.

Instrumentation Selection and Location

Selection of the specific types and numbers of instruments and their locations is done by the geotechnical engineer who designed the embankment and/or its foundation. A number of considerations influence these design decisions, but these considerations are not routinely communicated to project engineers or field personnel. This is unfortunate because

the more construction personnel know about the instruments and their purpose, the fewer problems there are likely to be with the installation and operation of the instruments, and the more reliable will be the measurements. Above all, good, frequent communication must be maintained between the geotechnical engineer and construction personnel. One way to ensure proper communication is for the geotechnical engineer to periodically visit the project site to discuss progress and review what to do at critical times during construction. The construction personnel should not hesitate to contact the geotechnical engineer if they have questions concerning any aspect of the instrumentation program.

Locations of the instruments will be indicated on the plans. In many instances instrumentation will be in the way of construction activities, or plans may call for instruments or readout devices to be placed in locations obviously hazardous to the instruments or equipment. The field engineer should consult with the soils engineer about relocating such items before they are installed. In most cases, soils engineers would rather have data from a less desirable location than no data at all, so changing the proposed location of instruments in the field is usually not a problem.

All instruments and readouts should be clearly marked or flagged when they are installed. Such marking cannot be overdone. The contractor's foremen, equipment operators, and laborers should be well aware of the location of the instrumentation and its importance to the project.

Instrument Types

Generally for embankment construction, measurements of pore pressures (using piezometers), vertical movements (using settlement platforms, Sondex tubes, and heave stakes), and horizontal movements (using inclinometers and survey stakes) are most commonly used. Other parameters such as earth pressure, soil strains, dynamic properties, and the like may occasionally be required.

There are numerous types of geotechnical instrumentation available, and each has its own advantages and limitations. Information on the specific instruments to be used on a project may be found in the references in the section on Information on Instrumentation or from a geotechnical engineer.

CONSTRUCTION MONITORING

It is often the responsibility of the construction engineer's staff to read the instruments. Soils engineers should make provisions for instructing and

familiarizing construction personnel with the instrumentation and explain the purpose of the instrumentation program, how the information will be used, how to read the instruments, how often they need to be read, and what to do with the readings. Presentation and interpretation of the measurements are the geotechnical engineer's responsibility; rarely are field personnel involved in this aspect of instrumentation.

Rules of Data Acquisition

The following rules should be adhered to in the acquisition of data:

1. Read all the instruments at approximately the same time of day on each day they are scheduled to be read.
2. Try to take readings when construction is not in progress. Read all of the instruments in the morning before construction starts, and/or at the end of the day after it has stopped.
3. Obtain complete and accurate information about the construction operations at the time of the readings. Examples are fill height, changes in visible water surfaces (rivers, excavations, dewatering, and the like), and any nearby construction operations. Note the weather conditions at the time of the readings.
4. Pay special attention to readings that are different from previous readings. Do not omit or ignore any readings, no matter how inaccurate they may seem. Substantial differences may indicate a reading error, instrument or readout malfunction, or the need for rapid remedial action. Double-check and record the value, and inform the geotechnical engineer immediately.

Frequency of Readings

Establishing the frequency of readings is the geotechnical engineer's responsibility. It will depend on the project, how rapidly settlement is taking place, or whether stability is critical. In general, readings will range from several times a day to one to three times a week during embankment construction, and perhaps one to four times a month during waiting periods or after the embankment is complete. Geotechnical specialists may initially specify more frequent readings than are really necessary and then adjust the schedule after interpreting the first few sets of data. If consecutive readings show no changes, the time between readings may be extended. If there are large differences, the frequency of readings may be increased. It should be emphasized that if impending failure is indicated

by the instruments, corrective action must be taken immediately, and instrumentation readings, data reduction, and interpretation done almost continually during such a time. Close coordination between field personnel and the geotechnical engineer is essential in such cases.

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