NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 147

TREATMENT OF PROBLEM FOUNDATIONS FOR HIGHWAY EMBANKMENTS

TRANSPORTATION RESEARCH BOARD
National Research Council

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TRANSPORTATION RESEARCH BOARD
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire highway community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff Transportation Research Board

This synthesis will be of interest to geotechnical engineers, highway designers, construction engineers, planners, and others interested in constructing or widening highway embankments on problem foundations. Information is presented on site investigation and testing and on the various construction alternatives that are available when a highway must cross a problem foundation site. In particular, the synthesis concentrates on those alternatives that are useful for constructing embankments across problem foundations.

Administrators, engineers, and researchers ae continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated, and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

Construction over problem soil areas requires extensive site investigations and comparative design analyses to evaluate alternatives. This report of the Transportation Research Board updates and expands Synthesis 29, which was published in 1973. It

discusses the treatments and construction procedures that can be used to construct highway embankments over areas of soil that would otherwise not support such embankments. It describes site investigations, design analyses, and the kinds of treatments currently being used, including where they are applicable and the advantages and disadvantages of each.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researcher in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

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

TREATMENT OF PROBLEM FOUNDATIONS FOR HIGHWAY EMBANKMENTS

SUMMARY

In many areas the earliest preliminary location planning should consider the possibility that some routes might involve problem foundation soils. The relatively long time required to evaluate the impact of such areas on the required overall design and construction time often makes it advantageous to include special foundation investigations as part of the preliminary planning studies. Right-of-way for some soft-foundation construction alternatives may exceed normal requirements. In addition, alternatives involving, for example, foundation stabilization by consolidation may require long periods of time under surcharge loading. Additional right-of-way and time for surcharge loading may be available only if early planning studies recognize these special needs and consider their impact before final route selection and public hearings. If this is not done, an objective appraisal of all appropriate construction alternatives may be precluded. These comments apply as well to upgrading of existing facilities; often extraordinary time is required for planning, design, and construction on problem soils.

Construction over problem soil areas requires extensive site investigations, environmental impact studies, and detailed comparative design analyses to evaluate possible construction alternatives. These investigations are expensive, and their costs and the costs for preparing comparative designs should be considered separately from the ordinary costs for the preparation of designs, plans, and specifications for normal subsoil conditions. The added engineering costs are, however, more than offset by the potential savings in construction costs; avoidance of environmental problems; improved performance; better-riding pavement; less maintenance and traffic interference; and, perhaps most important, a significantly reduced possibility of embankment failure or increased construction costs caused by contractor claims. To gain these benefits, agencies must have qualified geotechnical engineers on their staffs, even if plans and contract documents are prepared by outside consultants.

Applicable construction alternatives include (1) elevated structure, (2) berms and flatter slopes, (3) lightweight fill, (4) embankment supported by piles, (5) excavation or displacement of soft soils and replacement by suitable fill materials, (6) stabilization by consolidation with or without vertical drainage, (7) stabilization by chemicals and other additives, (8) physical densification of loose granular deposits and waste materials, (9) vibrocompaction methods and stone columns, (10) reinforcement, and (11) at sites of adequate stability, no soil improvement treatment whatsoever, despite the presence of soils that may undergo considerable settlement. In this last case, by relying

on especially detailed field investigations and meticulous design studies, relatively uniform settlements can be achieved. Each alternative can be evaluated based on factors such as local experience, cost of right-of-way, construction problems and costs, maintenance, ecological and environmental effects during and after construction, availability and cost of suitable fill materials, and disposal area availability. None of these methods should be considered to have sufficient merit to warrant a significant cost premium over other alternatives without detailed comparative cost analyses. The time available for construction and subsoil stabilization often affects selection of a particular improvement method. Other factors affecting this selection include previous favorable or unfavorable experience with some treatment alternatives, the ability to obtain high-quality subsurface information and soil properties, and the likelihood of having competent field inspection and monitoring of construction and performance.

It is possible to effectively treat and improve a number of special soils and problem materials that are occasionally encountered along highway routes so that they will make suitable embankment foundations at reasonable cost. Upgrading existing facilities is an important consideration today, and a number of ground improvement techniques can be very effective for widening embankments on poor foundation soils.

Economical and safe embankment foundations, especially if soil improvement is involved, are greatly dependent on the quality of the subsurface investigation, soils testing, and geotechnical analyses and design. In situ testing techniques can be cost-effective for obtaining soil properties required for design. If potential treatment costs are high, another useful technique to achieve maximum economy is a full-scale field test section. Such tests assure that the chosen technique is technically reliable, indicate possible design and construction improvements, and are useful for training construction inspection personnel.

Quality and amount of field inspection including geotechnical instrumentation are especially important. The postconstruction performance of most soil improvements techniques is directly dependent on the quality of construction supervision. With a number of methods, the quality of field control exercised affects the cost of the completed work, with the lowest total costs associated with the best quality control and performance monitoring.

CHAPTER ONE

INTRODUCTION

Advances in geotechnical engineering since Synthesis 29 (*1*) was published in 1975 have generated a need for an updated synthesis dealing with the practical applications of a number of new foundation treatment technologies. In addition, a changing highway program with an increasing need to rehabilitate or improve existing facilities has created problems somewhat different from those encountered with new construction. Under proper conditions, recent developments in foundation treatment methods may offer effective and economical solutions to these problems.

Highway construction in areas of problem foundation soils involves factors that affect (a) design, (b) construction procedures and scheduling, (c) construction and postconstruction costs, (d) use of the completed highway, and (e) public attitudes toward the competency of the engineering agency involved in the project. These and other factors are listed in Table 1.

Minimum construction cost is, of course, important, but it is not necessarily a dominating factor. The potential for embankment failures, poor-riding pavement, and high maintenance costs must also be taken into account. Interference to highway users on heavily traveled roads during postconstruction repairs involves major economic and safety considerations. These may warrant a premium for construction alternatives that reduce maintenance costs and highway user interference. This suggests that some input from maintenance departments during initial planning and design studies may be appropriate.

Today, it is essential to consider the environmental and aesthetic impact as well as the safety and public relations aspects of the project. This last consideration may be especially important when constructing embankments over soft foundations because of the long time period required if, for example, foundation stabilization is by consolidation under a surcharge fill. Under these circumstances, a news release may be desirable to explain that the delay in completing the roadway will minimize construction costs and improve postconstruction performance of the highway. A simple explanation of consolidation behavior and time required may be helpful. Other examples in which good public relations is important include projects using blasting or dynamic compaction, because of the vibrations these methods produce. Yarger (*2*) describes the public relations aspects of a dynamic compaction project in Montana. It appears that similar efforts could be used to advantage in other foundation treatment projects.

TABLE 1

FACTORS INVOLVED IN CONSTRUCTING EMBANKMENTS ON PROBLEM FOUNDATIONS

ITEM	REMARKS
Additional construction costs	Substantial; may be as much as several million dollars per mile.
Safety and public relations	Excessive postconstruction differential settlements may require taking part of roadway out of service for maintenance: Serious safety hazard for heavily traveled roads. Major inconvenience--public-relations problems.
Maintenance cost	May be large: More expensive construction may minimize postconstruction maintenance. Maintenance costs are sometimes regarded as deferred construction costs.
Environmental considerations	May determine type of highway construction and possible alternatives for foundation treatment.
Foundation stability during construction	Detailed subsurface investigations, laboratory and in situ tests, and design studies required.
Tolerable postconstruction total and differential settlements	Appropriate criteria not well formulated; subjective; depends on engineering and public attitudes.
Structure vs. embankment	An important decision affecting both construction and maintenance costs.
Construction time available	Some alternatives may be eliminated by need for early completion date.

QUESTIONNAIRES

In an attempt to find out about particular problem soils and popular methods of foundation treatment, a questionnaire (see Appendix A for an example) was sent to the appropriate geotechnical or materials engineer or geologist of each of the 50 states as well as the District of Columbia and Puerto Rico. Forty-two questionnaires were returned, and the answers provided were very helpful in the preparation of this synthesis. As might be expected, problem soils range from permafrost and muskeg to collapsing desert soils to garbage. Also, a wide variety of treatment methods are employed to deal with these problem soils. Throughout this synthesis, mention will be made as appropriate to many of these problem soils and treatment methods, including both successes and failures.

SCOPE

This synthesis is a revision and updating of NCHRP Synthesis 29, *Treatment of Soft Foundations for Highway Embankments* (*1*). It is intended to be used in conjunction with the following NCHRP syntheses:

2, *Bridge Approach Design and Construction Practices* (*3*) (currently being updated under Topic 18-03);
8, *Construction of Embankments* (*4*);
33, *Acquisition and Use of Geotechnical Information* (*5*);
42, *Design of Pile Foundations* (*6*);
89, *Geotechnical Instrumentation for Monitoring Field Performance* (*7*); and
107, *Shallow Foundations for Highway Structures* (*8*).

A primary objective of this synthesis is to describe the geotechnical alternatives for constructing highways over problem foundation soils. In addition to the treatment of soft clays and peats, which were the thrust of Synthesis 29, the scope is expanded to include treatment of other problem deposits such as landfills and waste dumps, mine wastes, liquefiable deposits, etc. Developments in treatment methods and geotechnical site investigation techniques since 1975 are also emphasized. Detailed design procedures are beyond the scope of this synthesis, but pertinent design details and selected references on design and construction of the various methods are given.

Although virtually all soil types that might serve as foundation materials are included in this synthesis, two important problem materials, expansive soils and shales, are not specifically treated herein. This is not to say that these materials are unimportant; they cause major pavement and embankment stability problems in many states and localities. However, as foundation materials, they rarely affect the stability of the embankment itself, although they may cause serious embankment deformations. For information on expansive soils in highway subgrades, see Snethen (*9, 10*). Strom et al. (*11*) and Strom (*12*) are good references on the design and construction of shale embankments.

LITERATURE AND REFERENCES

Since Synthesis 29 was written, considerable research has been carried out, new methods have been developed, old methods have been improved, and numerous useful case histories have been published. An extensive list of references is given on each method and problem soil discussed. Emphasis has been placed on Federal Highway Administration (FHWA) research and implementation studies, publications of the Transportation Research Board (TRB), and other geotechnical engineering literature. These references should be consulted for additional information regarding design procedures and calculations, construction details, research results, and specifications. Published case histories provide information that can be helpful to the design and construction engineer encountering similar problems.

CHAPTER TWO

DESIGN PROCESS PHILOSOPHY

PLANNING AND PRELIMINARY DESIGN

Adequate Transportation System

Providing an adequate transportation facility at the lowest overall life-cycle cost involves many elements besides initial construction. In selecting a procedure for the construction of a highway across a problem soil area, each of the following elements must be carefully considered:

- Construction cost
- Maintenance cost
- Performance and safety (pavement smoothness; hazards caused by maintenance operations; potential failures)
- Inconvenience (a tangible factor, especially for heavily traveled roadways or long detours)
- Environmental aspects
- Aesthetic aspects (appearance of completed work with respect to its surroundings)

Performance and safety considerations are especially important for heavily traveled roadways, and it may be necessary to provide more expensive initial construction in problem soil areas than would be required for roads carrying moderate to low traffic volumes. Inconvenience costs to the motoring public caused by closing a roadway or traffic lanes for maintenance can be large; in some cases, especially in urban areas, it may be almost impossible to close traffic lanes for maintenance purposes, even during off-peak or nighttime hours. Consequently, design criteria for an adequate transportation facility must be flexible and take into account long-term considerations.

Planning Studies

It is essential that the earliest planning studies recognize fully the impact of constructing highways in areas of problem soils. A thorough evaluation of possible alternatives involves many decisions that are generally affected by the construction time available; hence, it is particularly important that technical problems involved in constructing in these areas be recognized very early in the planning process.

The manner in which problem soil areas influence final decisions is not obvious because their effect varies greatly according to local conditions. Areas where problem foundation soils exist should be investigated early during the planning phase, because at that time it may be possible to avoid these areas. Public hearings held after completion of planning studies may make the alignment permanent, even though subsequent studies show that alignment changes could avoid serious difficulties in these problem areas. On the other hand, care should be exercised during the early investigations to avoid giving the impression to the public that one alignment is preferable to another.

Young et al. (*13*) discuss the general principles of highway route selection and the use of aerial photographs in planning studies. A discussion of the value of geotechnical input into the planning and route selection process is found in Rib (*14*), NCHRP Synthesis 33 (*5*), and Schuster and Krizek (*15*). A flow chart of the highway corridor planning process is shown in Figure 1; indicated are the special requirements and procedures that must be introduced when roadways cross problem foundation soils. Soil deposits and landforms that have the potential to cause embankment foundation problems are listed in Table 2. (Table 2 uses the unified classification of soils; equivalent AASHTO classifications can be found in References *16* and *17*.) In all cases, a thorough review must be made of previous experience with construction and maintenance of highways on similar geologic and subsoil conditions.

A classic study of the use of aerial photos and other terrain analysis methods for determination of the location of 90 miles (145 km) of Interstate highway in New York was reported by Hofmann and Fleckenstein (*18*). In addition to soils and geologic information, earthwork costs on similar classes of highways on similar terrains were studied, and probable total earthwork construction costs were developed. This information was an important consideration in the selection of the most economical route for the proposed highway. During the past 20 years, engineers at the Swedish Geotechnical Institute have developed similar procedures using aerial photographs to assist highway planners during the very early stages of corridor selection. Through the techniques developed, planners are able with a high degree of certainty to determine the approximate depth and strength of soft clays and to locate the boundaries between different soil types, potential landslide areas, and granular materials for roadway construction. Thus it is possible to include pertinent geotechnical information during the very early stages of highway planning. (For additional information, contact the Swedish Geotechnical Institute, S-581 01 Linköping, Sweden.)

The use of computers and optimization techniques can materially assist in the initial location and planning studies. Information on geology, topography, soils, foundation conditions, and land use is entered into GCARS, the Generalized Computer-Aided Route Selection system developed at Purdue University (*19, 20*). This system optimizes the route selection process starting with a region and progressing down to a few specific trial alignments (*21, 22*). Although it has not been done, in principle, information important to foundation treatment could easily be taken into account by GCARS.

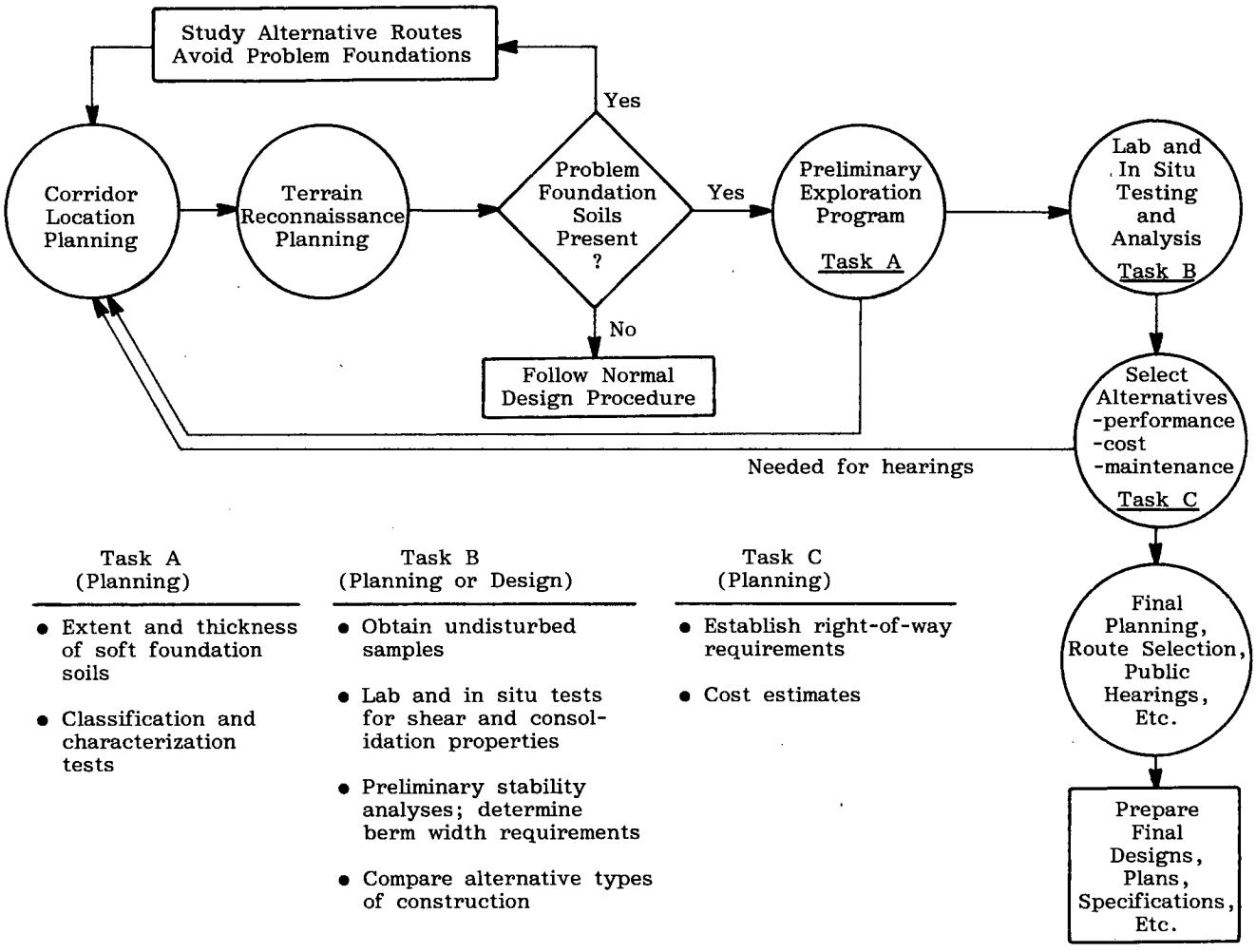


FIGURE 1 Requirements for input of geotechnical information into the corridor planning phase when problem soils are present.

In rural areas, early consideration of the impact of problem foundation soils on highway construction often permits relocation of the alignment. In urban areas, high real estate costs and public relations aspects may limit construction to problem areas, such as dumps, swamps, and tidal flats, that have not previously been developed. In some areas, the highway must not upset the existing environment, and this may become a controlling factor. Alternatively, where a swamp or soft-foundation area has already been environmentally spoiled by pollution, unauthorized dumping of wastes, etc., the construction of an embankment may be a way to obliterate an eyesore and to actually enhance the utilization of the area and the public image of the project.

A major factor in constructing highways in areas of problem foundation soils is the early recognition of the time required and cost of construction in these areas. For example, if the treatment method appropriate for a particular set of geotechnical circumstances requires consolidation of the soft subsoils, several months to a year or more may be required to complete the process. This time may not be available unless the time factor is recognized early during the planning phase. It may be possible to schedule work on various sections of a road to permit use of alternatives such as foundation consolidation for some sections. Failure to consider the time required for some foundation treatments may preclude consideration of their use during subsequent highway design, even if the particular treatment would result in substantial cost savings. Unfortunately, construction and maintenance costs often increase as the time available for construction decreases.

In fact, some of the least costly but more time-consuming techniques have become less attractive in recent years because of shorter project programs, tighter schedules, and the need to maintain traffic. Thus, it is becoming all the more crucial that appropriate geotechnical information be available for use early in the planning stage.

Preliminary Design Studies

Preliminary design studies are often carried out as a part of the planning process. In those cases in which previous studies have shown that it is not practicable to avoid problem foundation soils, the influence of subsoil conditions should be considered early in preliminary design. In this way, changes in the gradeline and/or alignment to avoid or minimize problems can be more readily accommodated. Often, for example, minor changes in

TABLE 2

DEPOSITS PRESENTING POTENTIAL EMBANKMENT FOUNDATION PROBLEMS

UNIT	LANDFORM	SOIL TYPES	UNIFIED CLASSIFICATION[a]
Alluvial deposits	Flood plain	Highly variable; natural levees to back swamp deposits.	All (GW, GP rare)
	Terrace	Highly variable.	All (GW, GP rare)
Glacio-lacustrine	Delta	Typical structure of gravelly topset, sandy foreset, and fine-grained bottomset beds. Generally becomes finer with depth and distance from source.	All, depending on location in land form
	Lake plain shore deposits	Thin veneer of sandy or silty soil over silt-clay soils.	SM; ML; ML-CL; ML-MH
	Lake plain bottom sediments	Varved silt-clay soils with occasional fine sand-silt laminae.	CH; CL; MH-CH; CL-CH; ML-CL
Glacio-marine	Marine plain bottom sediments	Massive silt-clay soils (laminations usually absent).	CH; MH-CH; CL-CH
Organic deposits	Swamp, bog, muskeg, etc.	Plant remains in various stages of decomposition with some mineral soil; may contain marl.	PT; OL; OH
	Tidal marsh	Highly variable but usually fine-grained. May have vegetative mat over organic silt or organic clay.	PT; OL; OH; MH; CH; MH-CH
Kart topography	Sinkhole, cavern, channel, etc.	Clays with occasional gravel, boulders.	CL; CH
Desert deposits	Alluvial fan; bajada	Highly variable; may be collapsible.	All, depending on location in landform
	Playa	Inorganic clays and silts; may be stratified.	CL; CH; ML; MH
	Other alluvial landforms	Generally granular; may be loose or collapsible.	All G and S
Landfills, dumps	Variable	Highly variable; organic and inorganic.	?
Industrial wastes	Variable	Usually loose, soft, unconsolidated marginally stable.	?
Mine wastes	Tailings dam	Sands, silts, and clays; usually inorganic; may be stratified.	SP; SM; SC; ML; MH; CL; CH
	Slurry pond	Silts to very fine clays.	ML; MH; CL; CH
	Strip-mined area	Highly variable; depends on mining operation and nature of overburden; boulders to clays; may be organic.	All

[a]For equivalent AASHTO classification, see References 16 and 17.

alignment can result in a significantly reduced thickness of soft-foundation soil by traversing areas where the deposit is not as deep. Although economic aspects of highway construction over poor foundations should not dominate the final selection of gradeline or roadway alignment, neither should these factors be finalized without due consideration of the possible cost and time benefits to be gained when the impact of problem foundations on highway construction and maintenance is properly taken into account.

NCHRP Synthesis 33 (5) has a good general discussion of the impact of geotechnical information on preliminary design.

Final Foundation Design

The time available for final design seems to be becoming shorter and shorter, and occasionally it is only sufficient to enable the preparation of contract documents. When this is expected, the general features of the design must be determined in essentially a final manner during the preliminary design phase and before the final foundation design phase is initiated. It should be recognized that although a detailed geotechnical investigation and design may save money, it will take time to complete. These delays may cause increased construction costs

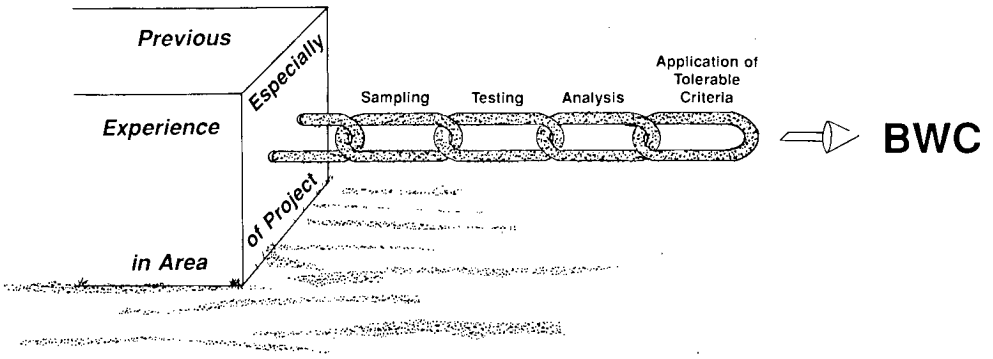


FIGURE 2 The geotechnical "system" (after *23*). BWC = Build With Confidence.

because of inflation and fluctuations in interest rates. On the other hand, a proper geotechnical design adequate for the construction will probably reduce contractor claims caused by "changed conditions."

PHILOSOPHY OF DESIGN

The Geotechnical Design System

The design system ordinarily followed in geotechnical engineering practice can be represented by the links of a chain, as shown in Figure 2 (*23*). Each link represents a stage in design: sampling (site investigation), testing (laboratory and/or in situ), analysis (design calculations), and the application of tolerable criteria (minimum factor of safety, maximum allowable settlement, etc.). This last item is based on experience with the performance of similar structures. Of course, the chain must be anchored or fixed to previous field experience in the area. If each of these "links" is followed according to usual practice in geotechnical engineering, then we are able to "B.W.C.," that is, "build with confidence." The analogy of the links of a chain is quite appropriate: The system is only as strong as its weakest link. If any one link is inadequate, failure can occur. Virtually all designs in geotechnical engineering practice, including those involving soil improvement and foundation treatment, follow this system.

Critical vs. Noncritical Design and Construction

Carroll (*24*) applied the concept of critical and noncritical design and construction to filtration and drainage applications using geotextiles. The concept can be adapted to other engineering situations such as embankments on problem foundation soils. Example criteria are presented in Table 3. The level of design required for features with critical conditions is more detailed and involved than that required for less critical or routine designs. Catastrophic failure of most highway embankments would be considered critical because of the potential for loss of life, the involvement of appurtenant structures, and the cost of repair. Even without a catastrophic failure, poor embankment performance can adversely affect traffic safety and volume. Highway engineers responsible for embankments at problem soil sites have a serious responsibility in this regard.

Steps in Ordinary Foundation Design

In order to develop the most economical foundation for a particular structure, the geotechnical engineer goes through the following steps (*25*):

1. Establish the scope of the problem
2. Investigate the conditions at the site
3. Formulate a trial design

TABLE 3

EXAMPLES OF CRITICAL AND NONCRITICAL EMBANKMENT DESIGN AND CONSTRUCTION

	Critical	Noncritical
1. Stability	large, unexpected, catastrophic movements	slow, creep movements
	structures involved	no structures involved
	no evidence of impending failure	evidence of instability
2. Settlements	large total and differential	small total and differential
	occur over relatively short distances	occur over large distances
	rapid direction of traffic	slow transverse to direction of traffic
3. Repairs	repair cost much greater than original construction cost	repair cost less than original construction cost

4. Establish a model of the subsurface to be analyzed
5. Determine the loads and soil parameters
6. Perform the analyses
7. Compare the results
8. Modify the design
9. Observe the construction

These steps must be followed, whether the foundation is for a bridge or for an embankment. The analyses performed are for stability (called "bearing capacity" in foundation engineering) and settlement. The first requirement is that the foundation must be safe against possible instability; that is, the foundation soils must have adequate bearing capacity to support the embankment. Thus, a bearing capacity analysis is done first and the factor of safety (Chapter 6) must be adequate. If the bearing capacity is satisfactory, then settlement analyses are carried out in which both the immediate and long-term settlements are estimated. In both instances, if the bearing capacity is inadequate and/or the settlements are too large, the site is a good candidate for some sort of soil improvement.

Settlements Considered Tolerable

There is considerable information in the geotechnical literature concerning the tolerable settlements of just about any structure except highway embankments. The work of Skempton and MacDonald (*26*) is, of course, classic, and updatings by Sowers (*27*) and Wahls (*28*) have been very useful to both foundation and structural engineers. Unfortunately, there is almost no similar information available about highway embankments. Synthesis 29 (*1*) has noted that postconstruction settlements during the economic life of a roadway of as much as 1 to 2 ft (0.3 to 0.6 m) are generally considered tolerable provided they (a) are reasonably uniform, (b) do not occur adjacent to a pile-supported structure, and (c) occur slowly over a long period of time. If the last condition occurs, any detrimental settlement can be corrected when the pavement is resurfaced, which is often done at intervals of 10 to 15 years.

An additional requirement is that sufficient subsurface information is available to assure the designer that acute differential settlements will not occur. For example, where a high fill crosses a valley, the maximum settlement generally occurs near the center of the valley. But the greatest differential settlement will most likely be near natural or man-made discontinuities such as the transition between cut and fill, unless the valley is very narrow and settlements near the center are large. Differential settlements are almost always a problem with low embankments because of the variability of soil properties and thickness of the compressible layers. When some doubt exists about the uniformity of postconstruction settlements, a more detailed site investigation program (Chapter 5) is required. Also, flexible pavement is usually selected, at least in the areas of greatest expected differential settlement. This is also done in some states when predicted settlements exceed 6 in. (150 mm).

Bridge approach embankments are a major concern, and reference should be made to NCHRP Synthesis 2 (*3*) and the forthcoming update. Tolerable settlements of approach embankments depend on the type of structure, location, foundation soil conditions, operational criteria, etc. Information and references on abutment movements can be found in References *8*, *29*, and *30*.

WHAT TO DO?

What choices does the geotechnical engineer have if either the allowable bearing capacity of the conventional embankment is too low and/or the predicted settlements are too large? Depending on the site conditions, traffic considerations, local experience, and other factors, there are basically three choices available: (a) take the "no build" alternative, (b) put the roadway on a structure, and (c) use some type of foundation soil improvement so that the embankment will be stable and settlements tolerable.

The first choice rarely is a real possibility. The second choice is discussed in the next section; basically, it is a decision as to what type of bridge to build. Although they do have some disadvantages, soil "bridges" (i.e., embankments) are much less expensive than steel or concrete bridges. And if the decision is to go with an embankment, then the results of the stability and settlement analyses will often indicate that some type of improvement of the foundation soil should be seriously considered.

Elevated Structure or Embankment?

Some questions involved in highway construction on problem foundations are listed in Table 4. The first and most important decision is the choice between elevated structure and embankment construction, with or without foundation soil treatment. For some highways, this question involves a choice between methods having cost differentials that may be several million dollars per mile. Structure costs including foundations generally range from about \$40 to \$100 per ft^2 of traffic lane (\$400 to \$1000 per m^2), but they vary widely depending on local conditions. For example, consider that an embankment 27 ft (8.5 m) high constructed of lightweight fill weighing 20 lb/ft^3 at \$100 per yd^3 costs about \$100/ft^2, which is about what bridges cost in many localities.

In areas where problem foundation soils have been particularly troublesome, the question of using an elevated structure versus an embankment is an important one, especially in urban areas and for heavily traveled roads. Where foundations are thick, weak, and highly compressible, this decision involves, for some engineers, the feasibility of constructing an embankment without excessive postconstruction settlements. This question (see Table 4) is particularly troublesome where designers have not had satisfactory experiences with the soil improvement techniques that they have tried, and where they have not attempted some of the recently developed techniques. Under these circumstances, an evaluation of experiences reported by others is difficult, especially because of differences in details among specialists.

In evaluating the merits of elevated structure versus an embankment with some type of foundation stabilization, it is satisfactory to assume that each can provide an equivalent roadway when current state-of-the-art technology is employed. Conversely, inferior design or construction can result in a rough pavement surface regardless of the method of construction employed. Designers with limited experience in constructing em-

TABLE 4

QUESTIONS INVOLVED IN HIGHWAY CONSTRUCTION ON PROBLEM FOUNDATIONS

QUESTION	REMARKS
Elevated structure or embankment?	Will the embankment be stable?
	What is the probability and cost of failure?
	Can an embankment provide a satisfactory riding surface?
	Can added cost of elevated structure be justified?
	Construction time required may be a factor.
	What are the relative maintenance costs?
	What is the economic/design life of the structure?
Can, or should, postconstruction embankment settlements be accepted?	Will settlements be uniform or irregular?
	Should design remove all primary settlements and reduce secondary compression settlements?
	How much time is available for construction?

bankments on soft foundations may question the applicability of some of the recently developed soil improvement procedures. Although such designers would pay a premium to obtain an elevated structure, experience indicates that such a premium is rarely justified. This does not mean that an elevated structure in certain areas is not economically attractive or desirable (e.g., because of timing, high grade line, or other factors), but that alternatives should be evaluated on their merits without one method of construction being favored by an inherent cost premium regardless of the results of comparative evaluations.

Finally, the different maintenance requirements for embankments versus elevated structures should always be kept in mind. Embankments are essentially maintenance free, whereas elevated structures require periodic maintenance and may require major repairs or even complete replacement every 25 or 30 years.

Foundation Stabilization for Embankments

Even at sites where designers consider an embankment to be a practical means for supporting a highway, major questions still remain (see Table 4) regarding stability and postconstruction settlements that can or should be tolerated. Some alternatives provide for minimal construction cost, but at the expense of postconstruction maintenance. On the other hand, designs can be developed that minimize postconstruction maintenance resulting from problem foundations. A basic decision must be made as to which approach should be used for particular local conditions. The answer depends heavily on the acceptance by designers and project managers of the feasibility of designing and constructing embankments that will tolerably settle but not experience excessive differential settlements nor require excessive future maintenance.

The attitude of a particular highway agency toward postconstruction settlements influences greatly the design criteria and the method of construction selected. Some agencies consider that construction of an embankment over soft or problem foundations should not result in postconstruction maintenance more severe than in usual soil areas. This is possible, but it involves added construction costs. The position of these agencies is that design for anything less merely shifts construction costs to maintenance and subjects the using public to inconvenience costs and unnecessary safety hazards.

Alternatively, other highway agencies have the opinion that postconstruction settlements are not necessarily detrimental, provided they are uniform and the agency is willing to accept certain risks. With these agencies, the basic philosophy is that detailed programs of site investigation, soils testing, and geotechnical analyses are warranted, and it is feasible to achieve a minimum construction cost along with reasonable postconstruction maintenance and risk. This line of reasoning requires a willingness to spend a considerable added amount for site investigation, testing, and design, and a commitment to the concept that the results will be satisfactory if these procedures are carried out.

In this regard, too, there seems to be a trend with some agencies to admit that some construction and performance anomalies will exist, especially at problem soils sites. The approach then is to use geotechnical advice during construction and special provisions to take care of geotechnical anomalies encountered during construction.

Specific procedures for improving foundation conditions will be described in detail in Chapter 3.

Special Studies Required

Design concepts and attitudes for constructing highways over areas of problem foundation soils depend heavily on the experiences of individual highway departments and their consultants. Best results are achieved when a separate study is made of the

construction alternatives before final design of a highway is begun. Adequate lead time should be provided in the planning and design time schedules to be able to plan and conduct the necessary geotechnical activities such as site investigation, laboratory and in situ testing, and design studies. This lead time may be critical when foundation problems are of major importance. Construction of highways over problem soils usually involves greatly increased construction and maintenance costs. Also, substantial costs are incurred for the additional subsurface investigations and testing and for an adequate engineering evaluation of possible alternatives. The evaluation of various alternatives requires specialized knowledge and experience in geotechnical and foundation engineering.

Problem foundation areas vary greatly, and depending on agency experience and knowledge, often a number of alternative designs and variations must be prepared to accommodate differences in subsoil properties. Selection of the most suitable alternative technique generally involves an effort much greater than that normally required to design for average subsoil conditions, because the process is iterative in nature. If everyone approves of the first result, this part of the process is finished; if not, then other alternatives must be investigated and the process repeated until a satisfactory answer is obtained. This approach necessarily requires time, resources, and experience.

When the highway design is to be made by consulting engineering firms operating under contract, special geotechnical design and evaluation studies should be provided and paid for separately to assure that all appropriate alternatives are investigated. Even if some specialized geotechnical investigations and designs must be done by outside consultants, it is recommended that agencies continue to do some of their own geotechnical design work, especially the more difficult and challenging projects. Such in-house capability is important to be able to evaluate specialized geotechnical designs, and to manage contracts and control construction, especially if foundation treatment methods are utilized.

CHAPTER THREE

FOUNDATION TREATMENT METHODS

INTRODUCTION

Basically problem foundation soils can be improved by (*31*):

1. reducing the load;
2. replacing the problem materials by 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 and/or its foundation.

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 by 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 5. All have been used singly and in combination in the United States, although some methods are much more popular than others and some

TABLE 5

FOUNDATION TREATMENT ALTERNATIVES[a]

METHOD	VARIATIONS OF METHOD
Berms; flatter slopes	
Reduced-stress method	Lightweight fill (see Table 6).
Pile-supported roadway	Elevated structure supported by piles driven into suitable bearing stratum.
	Swedish method of supporting embankment on piles driven into suitable bearing material. Piles have individual pile caps covering only a portion of the fill.
Removal of problem materials and replacement by suitable fill	Complete excavation of problem materials and replacement by suitable fill.
	Partial excavation (the upper part) of soft material and replacement by suitable fill. No treatment of soft material not removed.
	Displacement of soft material by embankment weight, assisted by controlled excavation.
	Displacement of soft material by blasting, augmented by controlled placement of fill.
Stabilization of soft-foundation materials by consolidation	Consolidation by surcharge only.
	Consolidation by surcharge combined with vertical drains to accelerate consolidation.
	Consolidation by surcharge combined with pressure relief wells or vertical drains along toe of fill.
Consolidation with paving delayed (stage construction)	Before paving, permit consolidation to occur under normal embankment loading without surcharge; accept postconstruction settlements.
Chemical alteration and stabilization	Lime and cement columns; grouting and injections; electro-osmosis; thermal; freezing; organic.
Physical alteration and stabilization; densification	Dynamic compaction (heavy tamping); blasting; vibrocompaction and vibroreplacement; sand compaction piles, stone columns; water.
Reinforcement	Geotextiles and geogrids; fascines; Wager short-sheet piles; anchors; root piles.

[a]Note that some combinations of methods are also feasible

have only been used on an experimental basis or for structures other than highway embankments.

To help assess which method of ground improvement may be the most appropriate, the following factors should be considered (*32*):

1. The operating criteria for the embankment; e.g., stability requirements, allowable total and rate of settlement, level of maintenance, etc. This will establish the level of improvement required in terms of properties such as strength, modulus, compressibility, etc.
2. The area, depth, and total volume of soil to be treated.
3. Soil type and its initial properties.
4. Availability of materials, e.g., sand, gravel, water, admixtures, etc.
5. Availability of equipment and required skills.
6. Environmental factors such as waste disposal, erosion, water pollution, and effects on adjacent facilities and structures.
7. Local experience and preference.
8. Time available.
9. Cost.

The cost difference between the various methods of constructing on soft or problem soils can amount to several million dollars per mile. Where very high fills (> 100 ft or 30 m) are required, an elevated structure may be more economical, especially if the foundation is soft. But this may not be the case for embankments of 50 ft (15 m). Where soft-foundation areas are extensive, say miles or several tens of miles in some locations, the decision as to an appropriate construction procedure becomes a major factor worthy of detailed comparative cost analyses of several alternative designs. For projects of this size, field test sections are often desirable to establish the behavior of alternatives to elevated structures. Other reasons for their use are discussed in Chapter 5. Variations and combinations of the basic methods listed in Table 5 can be considered applicable, but not necessarily the most economical, for virtually any thickness or type of problem soil.

Good general information on the methods in Table 5 for treating soft and problem soil areas can be found in References *31–40*. These references also include detailed discussions of many of the specific treatment methods described in this chapter. Design procedures for the various methods are described in Chapter 6.

BERMS; FLATTER SLOPES

Probably the simplest way to increase the stability or bearing capacity of an embankment is to use berms (Figure 3a) or flatten the side slopes (Figure 3b) of the embankment (*31, 34, 36, 41*). As can be seen in Figure 3a, berms provide a counterweight against rotational or translational type failures, and in effect they add to the resisting forces.

Because of right-of-way costs, the cost of stabilizing berms may be large enough to affect the selection of the final type of

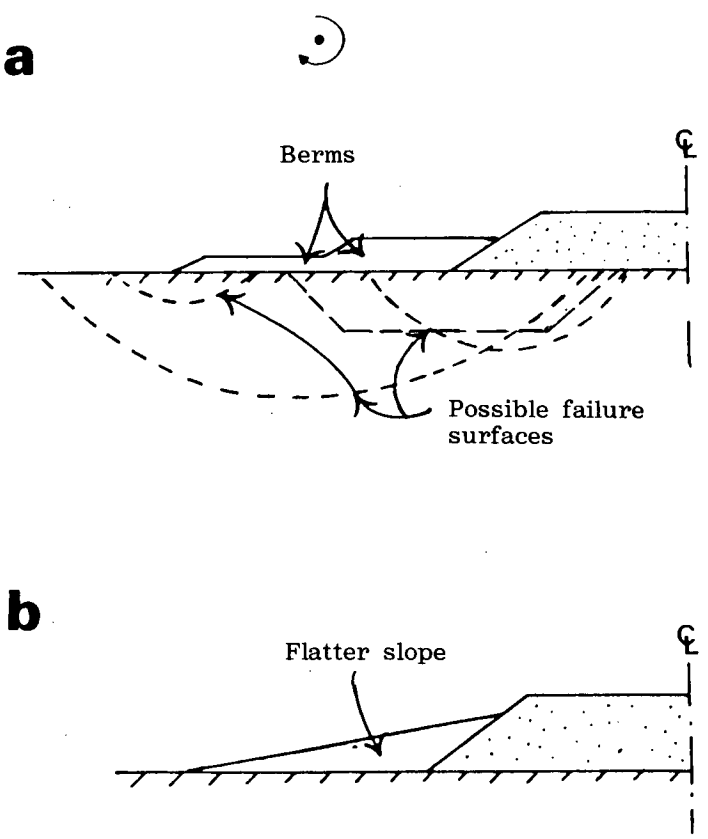


FIGURE 3 Stabilization of an embankment with (a) berms or (b) flatter slopes (after *31*).

construction; thus it is desirable to reduce the size of berms to a minimum. Occasionally, when the excavation and replacement technique is used for constructing a portion of the facility that must be opened to traffic relatively early, other sections that will not be required until a later date can be stabilized by a less costly technique, for example, consolidation. In such a case, the excavated materials can be used in the berms. In fact, berms can be constructed of just about any material available on site (except highly organic soils and peats), because their principal contribution is dead weight. They are a good place to dispose of excess excavated materials when balancing cut and fill, as well as a place to dispose of wetter-than-optimum materials, tree stumps, waste concrete, etc. If any of these types of materials are utilized in constructing very flat slopes, they should be placed outside an imaginary 2h:1v slope starting from the embankment crest.

The main disadvantage of berms is their size. Berm widths affect right-of-way requirements and length of drainage structures; hence, possible berm and slope requirements must be known when performing planning studies. Conversely, available right-of-way may preclude use of design alternatives requiring large berms and very flat slopes. The required widths of berms, especially if the soft deposit is very deep, can be on the same order as the width of the roadway itself. Berms are probably only economical in rural areas where land costs are very low and there is likely to be only minimal effect on utilities and adjacent property.

Berms also may increase total settlements, especially of the outer edge of the embankment. Because of the greater width of the loaded area, the dissipation of stress with depth is less. This may be a blessing in disguise, because greater settlement at the outer edge probably means less differential settlement between the outer edge and the centerline.

According to the questionnaire sent to U.S. highway agencies, berms have been used to some extent by almost all respondents. Design considerations for berms and flatter slopes are discussed in Chapter 6.

REDUCED STRESS (LIGHTWEIGHT FILL)

Because both settlement and stability of embankments on soft foundations depend on the weight of embankment material, settlement can be reduced and stability increased by use of lightweight embankment fill (*31, 33, 34, 36, 41*). Various types of lightweight materials that have been used in or considered for highway embankments are listed in Table 6 (*41, 42*). Because their compacted densities are significantly less than the unit weights of soils ordinarily used in embankment construction, the effective weight of the embankment can be reduced significantly by the use of these materials.

In addition to their low unit weights, many of the materials in Table 6 have a high angle of shearing resistance and a reasonable compressive modulus because of their interlock capabilities, especially after compaction (*43, 44*).

Figure 4 shows some typical cross sections of embankments constructed with lightweight materials in Sweden. If expanded clay or shale is used, a minimum depth of soil cover on the slopes of 20 in. (0.5 m) is required; minimum thickness of the road base is 2 ft (0.6 m).

A successful landslide correction using chipped bark and sawdust in a sidehill embankment was reported by Nelson and Allen (*43*). They used a 1 ft (0.3 m) gravel base under the pavement section; in addition, an asphalt seal was placed on the exposed slope to retard deterioration and pollution. There have been numerous other projects that used sawdust and bark as lightweight fill in the Pacific Northwest. Large quantities of sawdust were used in the approach embankments to the Dumbarton Bridge in San Francisco Bay. Edil (*45*) reports the use of sawdust, wood chips, and expanded shale in the lower portion of surcharge fills on peat. The lightweight part of the fill is left in place after the surcharge is removed, and because of large settlements, it usually remains below the water table and does not decompose.

Lightweight fill materials are often expensive, especially if they are manufactured (expanded shales and clays, polystyrene, lightweight concrete, etc.). Typical costs range from \$50 up to \$100/yd^3, including transportation. Even if they are a waste material (sawdust, bark, shells, cinders, slag, flyash, bottom ash, etc.) and almost free at the source, they must be transported to the site. In many instances, transportation costs alone have made the use of a lightweight material uneconomical, but each case should be examined on its merits.

In addition to their cost, lightweight materials have some other problems that designers must be aware of. They may absorb water; thus they must be encapsulated. They may break down when compacted; this of course increases their unit weight. Some lightweight materials are toxic and corrosive, and some are susceptible to breakdown caused by frost action. Compressed and baled peat, sawdust, bark, and similar materials must be maintained in a submerged or sealed condition to avoid deterioration. In some areas, these materials are prohibited for environmental reasons; even below the water table, they may be considered too acidic to be used adjacent to natural waterways. If properly encapsulated, however, their use may be acceptable.

Expanded polystyrene (EPS) is extremely lightweight (average 3 pcf or 48 kg/m^3) and can be made sufficiently strong to be able to support ordinary highway pavement and traffic loads with tolerable settlements. But it is also an excellent insulator, and there has been some hesitancy among highway departments in the northern United States to use it within 4 ft (1.2 m) of the pavement surface because of potential differential icing problems. Monahan (*46*) suggests that this disadvantage can be overcome, and he believes the material has great potential, especially in embankment applications. A test in northern Japan indicated that EPS boards effectively reduced frost penetration when used to widen narrow low-volume roads (*47*). Other problems with EPS include reports of burrowing animals in the material and increases in unit weight because of water absorption.

On the other hand, experience with EPS in Scandinavia has been very positive. Flaate (*48*) reports that EPS has been successfully used on more than 100 highway projects in Norway since 1972. Applications include embankments on peat and other soft soils, behind bridge abutments, and reconstruction of sidehill embankments on unstable slopes. In Sweden, more than 20 road embankments have been constructed with EPS (*41*). de Boer (*49*) reports using EPS behind seven bridge and overpass abutments in British Columbia. To protect against possible gasoline spills, the top and sideslopes were covered with a 10 mil polyethylene geomembrane. For additional information on EPS, see the *Proceedings* of the Conference on Plastic Foam in Road Embankments held in Oslo, Norway, in 1985 (*50*).

TABLE 6

LIGHTWEIGHT EMBANKMENT FILL MATERIALS (ADAPTED FROM *41, 42,* AND OTHER SOURCES)

Material	Approximate unit weight		Comments
	(kN/m^3)	lb/ft^3	
Bark (Pine and Fir)	8-10	35-64	Waste material used relatively rarely as it is difficult to compact. The risk of leached water from the bark polluting groundwater can be reduced or eliminated by using material initially stored in water and then allowed to air dry for some months. The compacted/loose volume ratio is on the order of 50 percent. Long-term settlement of bark fill may amount to 10 percent of compacted thickness.
Sawdust (Pine and Fir)	8-10	50-64	Waste material that is normally used below permanent groundwater level but has occasionally been employed for embankments that have had the side slopes sealed by asphalt or geomembrane.
Peat:			Proved particularly useful in Ireland for repairing existing roads by replacing gravel fills with baled peat.
Airdried: milled	3-5	19-32	
Baled horticultural	2	13	
compressed bales	8-10	51-64	
Fuel ash, slag, cinders, etc.	10-14	64-100	Waste materials such as pulverized fuel ash (PFA) are generally placed at least 0.3 m above maximum flood level. Such Materials may have cementing properties producing a significant increase in safety factor with time. In some cases (e.g., furnace slag), the materials absorb water with time, resulting in an increase in density.
Scrap cellular concrete	10	64	Significant volume decrease results when the material is compacted. Excessive compaction reduces the material to a powder.
Low-density cellular concrete	6	38	This is an experimental lightweight fill material manufactured from portland cement, water, and a foaming agent with the trade name Elastizell. The material is cast in situ.
Expanded clay or shale (lightweight aggregate)	3-10	20-64	The physical properties of this material, such as density, resistance, and compressibility, are generally very good for use as a lightweight fill, although some variations may be produced by the different manufacturing processes. The material is relatively expensive but can prove economical in comparison with other techniques for constructing high-standard roads. The minimum thickness of road pavement above the expanded clay is generally on the order of 0.6 m.
Expanded polystyrene	0.2-1	1.3-6	This is a superlight material used in Norway, Sweden, the United States, and Canada up to the present, but where its performance has proved very satisfactory and its usage is increasing. In Norway the material is used in blocks. The thickness of the cover varies between 0.5 and 1 m, depending on traffic-loading conditions. Incorporated within the pavement is a reinforced concrete slab cast directly on the polystyrene to reduce deformation and provide protection against oil, etc. The material is very expensive, but the very low density may make it economical in special circumstances.
Shells (oyster, clam, etc.)	11	70	Commercially mined or dredged shells available mainly on Gulf and Atlantic coasts. Sizes 0.5 to 3 in. (12 to 75 mm). When loosely dumped, shells have a low density and high bearing capacity because of interlock (35).

Because the crushing strength of some lightweight materials is relatively low, care must be taken during construction to avoid damaging the material, especially if conventional compaction equipment is used. Also, these materials may not be suitable for use as part of the pavement (as base or subbase) materials. Some expanded shales reportedly have poor resistance to freezing whereas others appear to be satisfactory in this respect. In any event, expanded clays and shales seem to be among the more popular materials selected, probably because their engineering behavior is more certain. Scrap lightweight concrete also has been used, but it seems to be easily crushed during transport and placement, and the crushed material is frost susceptible (*31*).

Clam and oyster shells have successfully been used as lightweight fill for embankments on very soft soils in Louisiana (*36*). The shells develop a high bearing capacity because of interlock, even when they are loosely dumped, which is necessary because of their low crushing strength. Differential settlements have reportedly not been a problem.

Lightweight fills have also been used on pile-supported roadways, described below, in order to reduce the loads on the piles.

Lightweight fill has been used to some extent by 40 percent of U.S. highway agencies responding to the questionnaire. Design considerations for embankments with lightweight materials are discussed in Chapter 6.

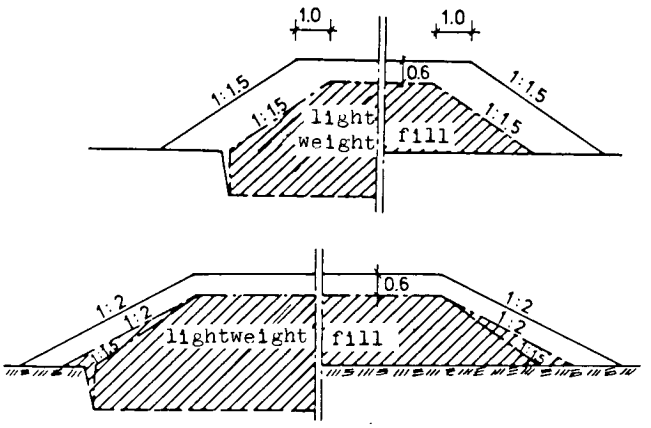


FIGURE 4 Cross sections of road embankments built of lightweight expanded clay (*41*). Dimensions in metres.

PILE-SUPPORTED ROADWAYS AND EMBANKMENTS

Conventional Elevated Structures

As discussed in Chapter 2, conventional trestle or bridge construction is sometimes used to cross areas of problem soils, particularly where the soils are extremely thick and slow draining so that settlements would be large and occur slowly. The use of a conventional type of elevated structure is advantageous in that the design is often much simpler than the design of an embankment with foundation treatment. A few typical sections often suffice for the design of an elevated structure, whereas an embankment on a soft soil must be designed for several different subsoil conditions, variations in thicknesses of soft soils, different properties, varying embankment heights. The pile foundations for the structure will also require design effort and costs, but these are usually provided for when the design fee is a percentage of estimated construction costs. This may not be the case for an embankment on a soft foundation.

Maintenance costs for an elevated structure could be low or relatively high, depending largely on the exposure of the structure, on the climate, and on numerous local factors. In any event, the structure may have to be entirely replaced in 25 to 30 years.

Pile-Supported Embankments

Embankments on relief piles have been used for more than 50 years in Sweden to reduce the settlements and increase the stability of embankments on soft foundations (*31, 51, 52*). They are also used rather extensively in Norway, Finland, Singapore, and Thailand, especially to even out the settlements of approach embankments to bridges. Four states (Georgia, Kansas, New York, and Virginia) report using the system on both new construction and for embankment widening, and as will be discussed later, this latter use appears to be particularly promising. Use of embankment piles avoids problems arising from directly loading soft-foundation soils by transferring the embankment loads to firmer strata at depth.

The system employs a large number of individual piles to carry the embankment load either by skin friction (Figure 5a) or by precast concrete pile caps (Figure 5b), above which the embankment fill is placed. If the shear strength of the foundation is particularly low (< 10 kPa or 200 psf or 1.5 psi) or the dry crust very thin, some type of stabilization or reinforcement is needed at the tops of the piles to keep the fill from punching into the soft clay surface, even between the pile caps. Traditionally, brushwood fascines were used for this purpose, but recently geotextiles have been satisfactorily substituted. Also, with very soft soils, there is a risk of lateral displacement of the piles during driving of adjacent piles; depending on the pile type and spacing, this could also occur with stiffer soils.

The piles are normally timber piles, but precast concrete piles have occasionally been used. The fill arches between individual pile caps so that piles carry the embankment and superimposed roadway and traffic loads. Separate precast concrete pile caps (0.8×0.8 m to 1.5×1.5 m square) generally cover about 25 percent of the base area of the embankment, depending on the spacing of the piling, the height of the embankment, and the strength of the foundation. The pile cap may be secured to the pile by a simple drift pin, or, more commonly, a tapered recess is cast in the pile cap so that it can fit directly onto the upper portion of the pile.

Because the sites where pile-supported embankments are used usually have a high water table, deterioration of timber piling is not considered a serious problem. If possible, the cutoff at the top should be below the permanent groundwater table. In clays and silts, the cutoff can be 1 ft or 0.3 m above because capillarity will keep the piles wet so they will not rot.

When embankment piles are used to support bridge approach

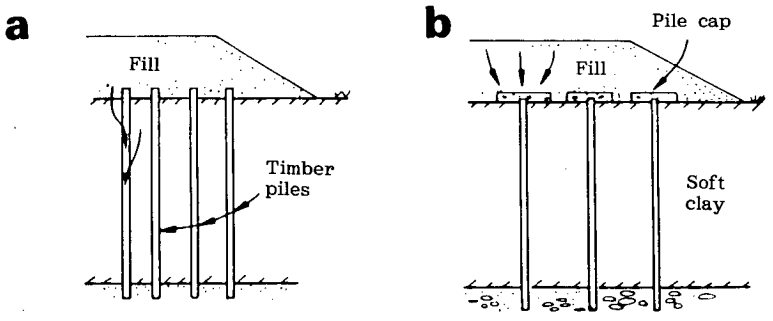


FIGURE 5 Embankment piles (a) without pile caps and (b) with pile caps (*52*).

embankments, vertical piles installed some distance from the abutment are driven with relatively less penetration into firm bearing soil, so that they can settle somewhat. As the bridge abutment is approached, the penetration requirements increase, so that the roadway settles relatively little adjacent to the bridge abutment (which is, of course, pile supported at such sites).

Beneath narrow embankments, at the edges of wide embankments, and adjacent to bridges, the piles are battered (typically at 1:10 up to 1:4) to take care of horizontal stresses imposed by the embankment; elsewhere, the piles are simply driven vertically. Driving piles in very soft clays, especially on sloping ground or adjacent to bridge abutments, has its problems. High excess pore water pressures can be generated, which increases the danger of a slide. In recent years, it has become common to utilize prefabricated strip drains to help reduce these excess pore pressures and speed up the reconsolidation of the clays around the piles (*53*).

The batter piles mentioned above are not without problems either. If the underlying firm bottom is highly irregular bedrock, seating of batter piles is problematic. Holtz and Massarsch (*54*) describe such a project in which all piles were driven vertically, and a geotextile reinforcement was used to take up the horizontal stresses imposed on the subsoil at the base of the embankment.

Depending on the relative costs of timber piles, precast concrete plates, and surcharge fill, the use of a pile-supported embankment is usually more expensive than preloading with or without vertical drains (discussed below), but it should be significantly less costly than an elevated structure. As mentioned, the method has only been used a few times in the United States, but it should be applicable to those sites where otherwise an elevated structure or even more expensive foundation treatments might be selected.

Sites where an existing embankment must be widened also present difficult problems, especially if the soils are soft and deep. This problem will be discussed in Chapter 4. Embankment piles could be advantageous in such a situation. Design considerations for embankment piles will be discussed in Chapter 6.

EXCAVATION/REMOVAL AND REPLACEMENT METHODS

A simple way to improve a soft or problem soil site is to remove the problem soils and replace them with higher quality materials. This treatment is most common for organic deposits and very soft clays, but it has also been used for extremely loose deposits of granular materials and collapsing soils, as well as for landfills, dumps, mine wastes, etc. This method is very popular among highway engineers, to judge by the questionnaire responses; almost all respondents report having used the method, both for stability problems and to reduce excessive settlements. Excavation and replacement methods are described by Broms (*31*), Sinacori et al. (*33*), Moore (*34*), Arman (*36*), and Hartlén (*41*), and they can be considered under the topics of (a) complete removal, (b) partial excavation, (c) underwater fill placement, (d) removal by displacement and partial excavation, and (e) displacement by blasting.

Complete Removal

The complete removal of soft- or problem foundation soils and replacement by suitable sand, sand and gravel, or blasted rock fill is a positive construction procedure, if properly carried out. Factors involved in using this method are listed in Table 7. For highway construction, the method has been applied pri-

TABLE 7

EXCAVATION OF SOFT-FOUNDATION SOILS

FACTOR	REMARKS
Excavation methods	Dozers and scrapers.
	Dragline or clamshell.
	Hydraulic suction dredging.
	Dipper dredge.
Disposal of excavated materials	A major consideration: Availability of disposal areas (mainly on site).
	Availability of transport.
	Cost of disposal may control feasibility.
	Required permits are time consuming to obtain. Must be obtained during planning or design.
Environmental considerations	Environmental considerations must be fully evaluated during preliminary design and environmental impact statement prepared.
	Disposal of excavated materials may affect disposal area adversely or beneficially; consider odors, leachate.
	Aesthetic enhancement and other benefits of controlled-disposal operations may be significant; investigate fully in preliminary design.

marily to predominantly organic swamp and marsh deposits underlain by firmer materials at a depth generally less than 15 to 20 ft (5 or 6 m). It has also been used successfully to much greater depths for dams built partially under water and for creating land for industrial development. By using special dredges, the method has been applied at a number of places where the maximum required dredging depth exceeded 100 ft (30 m) and has been used in water depths as great as 217 ft (66 m). Hence, consideration of the method for highway construction can be based largely on its cost relative to other alternatives. Shallow excavations can be made in the dry, especially in loose sands, wastes, and fibrous organic materials, but there may be slope instability and dewatering problems; excavation in the wet is normally economical and practicable. Where underwater excavation is used, the water level in the excavated area should be maintained at its natural elevation, no matter what excavation methods are used, to prevent collapse of the excavation.

Environmental aspects (Table 7) may be the most important consideration in the selection of the excavation and replacement method. Proper disposal of the excavated materials is difficult and expensive unless they can be disposed of on site, and prevention of environmental contamination during dredging operations, for example, can be a real problem. Environmental considerations of this method are discussed by Arman (*36*) and Broms (*31*).

The excavation of undesirable foundation soils and replacement with suitable fill materials is usually thought to be a relatively simple operation. Practically speaking, however, this is not the case, and stringent inspection and control must be provided to assure satisfactory and economical results; this is especially true for underwater excavations. Unless careful inspection is provided, soft materials may not be completely removed, with detrimental results, or alternatively may be removed to excessive depths, which increases both excavation and backfill costs. Regardless of the method of excavation, the process of excavating soft soils underwater almost invariably results in quantities of soft materials going into suspension and settling out along the bottom. Unexcavated soft material or accumulations of soft material ahead of the advancing backfill may become entrapped by the fill. This can be detected and prevented by careful and complete construction control, inspection, and testing procedures (*55*), as listed in Table 8. Without good quality control, entrapped pockets of soft material can become so thick that large sections of backfill must be excavated and the bottom thoroughly cleaned; alternatively, entrapped materials might be stabilized by consolidation through use of a temporary surcharge fill. On rare occasions, vertical drains have been installed in these pockets of entrapped materials to accelerate consolidation.

It is not generally possible to excavate underwater in one continuous cut to final grade. Almost invariably it will be necessary to make several passes and to do some cleanup excavation as the fill is placed. Details are discussed by Johnson et al. (*55*).

The extent of the excavation to remove the soft materials depends on their thickness and requires stability analyses of the excavated slopes. Design aspects of this method are discussed in Chapter 6; problems specific to bridge abutments are in NCHRP Synthesis 2 (*3*). Because the volume of material excavated must be replaced by an equal volume of backfill, for this method to be practical, suitable backfill materials must be available at reasonable cost. In some cases, the excavated materials can be used as the backfill after drying or other treatment, or in the case of loose granular materials, simply replaced and recompacted in lifts. Backfill operations underwater are not so simple, and comments on this subject will be made later in this section.

TABLE 8

CONSTRUCTION CONTROL AND INSPECTION PROCEDURES FOR EXCAVATIONS AND BACKFILLING OPERATIONS (AFTER *55*)

METHOD	CHARACTERISTICS
Borings and sampling	Grab sampling Clamshell Other
	Exploration borings and undisturbed sampling
	Wire-line sampling (under water)
In situ testing	Vane shear
	Penetration tests Dutch cone Standard Penetration Test Piezocone
	Pressuremeter
	Dilatometer
	Borehole shear
	Plate bearing tests
	Geophysical borehole logging methods (resistivity, density, etc.)
	Nuclear density meters
Laboratory testing	Classification and identification
	Compaction
	Permeability
	Shear strength
	Consolidation
Instrumentation (see Chapter Seven)	Piezometers
	Settlement platforms (surface and at various depths); profile gage
	Slope indicators

Partial Excavation

At some sites, the undesirable materials are underlain at modest depths by a somewhat stronger and less compressible material. If the surface materials are removed and replaced and the second layer can provide an adequate foundation, then it is often economical to excavate the upper layer and replace it by suitable fill, with no treatment of remaining soft materials. Where this has been done, the upper material has generally been fibrous peats or other organic soils capable of causing large settlements, whereas the lower materials are soft but less compressible clays, silts, or marls (*33*). The unexcavated and un-

treated soft material may cause significant nonuniform postconstruction settlements, and thus partial excavation may not be desirable for high-quality roads except at sites where the compressible layer is relatively homogeneous and approximately the same thickness, and the settlements that do occur are tolerable. Such sites obviously require a thorough site investigation and testing program.

In addition to settlement problems, the partial excavation method frequently involves stability problems. The underlying unexcavated soft materials may be too weak to support the embankment without stage construction, berms, lightweight fill, or very flat slopes. Furthermore, the excavation process itself may leave accumulations of soft material on the bottom or it may disturb the underlying soft soil. Embankment stability analyses (Chapter 6) with failure surfaces extending to the unexcavated soft material should be performed.

Underwater Fill Placement

Clean sand or sand and gravel with less than 8 to 12 percent finer than a 75μm (No. 200) sieve is well suited for underwater placement because these materials are not sensitive to placement water content. Materials containing more fines can be a problem because of segregation and density control, but they also can be upgraded, as described by Sinacori et al. (*33*). Densification of backfill materials underwater can be accomplished, and Johnson et al. (*55*), Arman (*36*), and Broms (*31*) describe some possibilities, although none are very cheap. Recent developments are reviewed by Koning (*56*) and Dembicki (*57*). One of the strong advantages of using blasted rock for backfill underwater is that for modest thicknesses, it is not very compressible, even when loosely dumped. To protect the underlying clays during dumping operations, a layer of sand and gravel or a geotextile could be used as a separator layer. If a geotextile is used, it would have to have a very high "survivability" (*58*) to avoid being damaged during backfilling.

A variety of methods may be used for placing fill materials underwater, and they are listed in Table 9. Construction inspection and control procedures suitable for excavation and fill placement were given in Table 8. Effective construction control for excavation or dredging and fill placement has an identifiable economic value because it affects directly the amount of material excavated and hence the volume of backfill that must be placed. Differences in quality control as they relate to the volume of required excavation and hence volume of fill are discussed for highway construction by Johnson et al. (*55*). The quality of construction control provided affects the overall cost of the job far more than do the differences in cost of improved inspection. In addition, poor construction control may result in a rough-riding pavement because of postconstruction settlements from the consolidation of soft entrapped or unexcavated materials that would have been avoided by better construction supervision.

TABLE 9

UNDERWATER FILL PLACEMENT METHODS (*55*)

METHOD	CHARACTERISTICS
Bottom-dump scows	Fill assumes flat slopes unless retained.
	Limited to minimum depths of about 15 ft (4.5 m) because of scow and tug drafts.
	Rapid; quick discharge entraps air and minimizes segregation.
Deck scows	Usable in shallow water.
	Unloading is slow, by dozer, clamshell, hydraulic jets, conveyors.
	Steep side slopes of fill can be achieved.
Hydraulic	Coarse materials drop out first; may cause shear failures in soft foundations.
	Fines may collect in low areas and have to be removed.
	Inspection of material being placed may be difficult.
Dumping at land edge of fill and pushing material into water by bulldozer	Fines in material placed below water tend to advance and accumulate in front of advancing fill.
	Work arrangement should result in central portion being ahead of side portions to displace sideways any soft bottom materials.
	In shallow water, bulldozer blades can shove materials downward to assist displacement of soft materials that accumulate at toe of fill. (Not suitable for displacing unexcavated soft materials.)

Removal by Displacement and Partial Excavation

As an alternative to excavation, it may be possible to displace soft materials by deliberately overstressing and therefore failing them by the weight of embankment combined with a temporary surcharge and/or partial excavation. This method is illustrated in Figures 6 and 7, and it is described by Sinacori et al. (*33*), Moore (*34*), Arman (*36*), Broms (*31*), and Hartlén (*41*). Weber (*59*) describes a reasonably successful project in which 60 ft (18 m) of soft San Francisco Bay mud were stabilized by displacement.

Removal by displacement is an old technique; it was not uncommon for fill to be continually dumped on a soft marshy area until the roadway eventually stabilized. Usually a large mud wave was created to the sides and ahead of the fill. As noted in Synthesis 8 (*4*), the method is not used so often today because of the uncertainty of complete removal of the undesirable soft materials. The questionnaire indicated that almost half the states have used displacement methods for both stability and settlement problems.

Broms (*31*) shows some procedures used in Sweden to control the direction of the displaced soil mud wave (Figure 8). Because the displacement method relies on the progressive failure of the foundation soils, the work must be carried out continuously. Thus it is important during even short breaks in the operation that personnel and equipment be kept some distance back of the leading edge. Any upheaved marsh material that accumulates at the leading edge of the fill should be removed to avoid

FIGURE 6 Marsh displacement (45 ft or 14 m deep) and embankment construction to surcharge grade in Michigan (*3*).

entrapping pockets of the displaced soil within the embankment. Although in some cases excellent removal of the soft soil down to firm bottom may be achieved, elsewhere pockets of soft soil may remain, which results in differential settlements and poor embankment performance. This is similar to the problem of incomplete excavation discussed above, and techniques discussed in that section apply here also.

Another problem is that the displacement of soft subsoils by the weight of the embankment may result in the intrusion of fill into the soft soils outside the limits of the roadway, which would add to the cost of the construction. However, there is good evidence that this does not occur to any appreciable extent if the mudwave is properly controlled (*34, 60*). The mudwave and possible surface organic mat should be removed from in

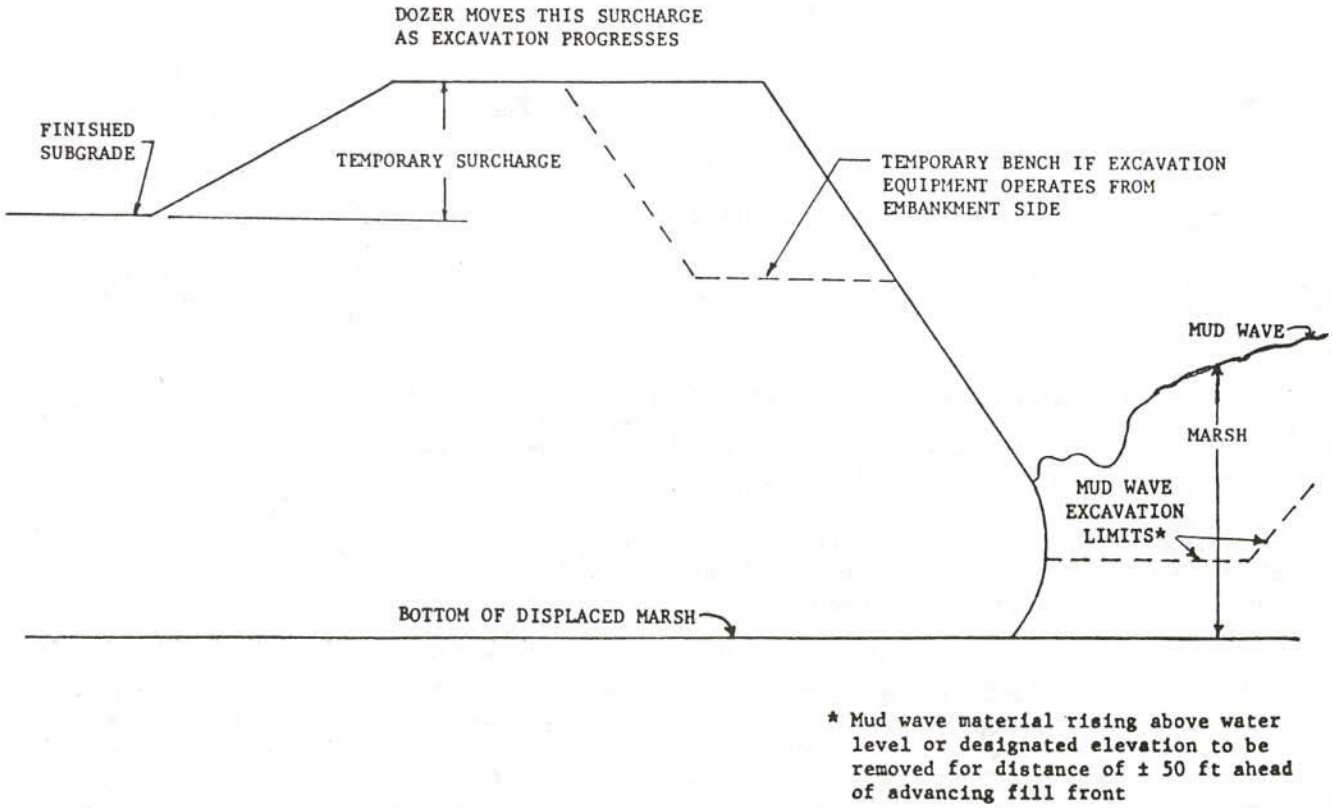


FIGURE 7 Longitudinal section of marsh removal by displacement and embankment construction with surcharge (*3*).

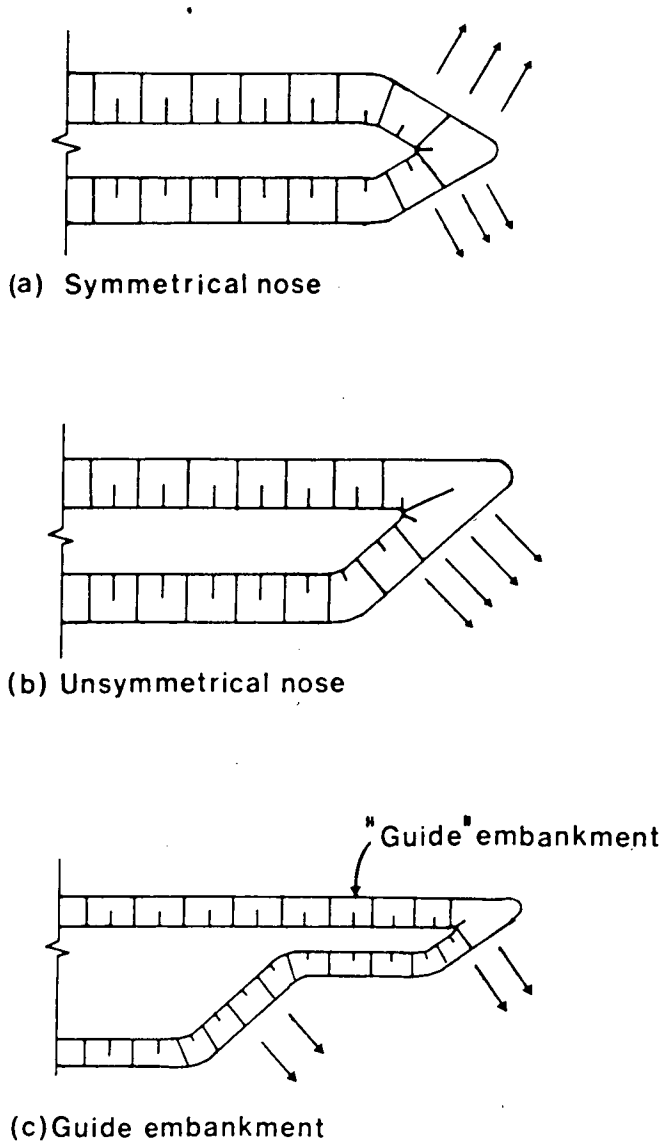


FIGURE 8 Directing the displacement of soft soils (*31*).

front of the fill for a minimum distance of 50 ft (15 m); then all displacements will go in this direction and there will be very little sideways displacement. Moore (*60*) recommends excavating any of the mudwave appearing above the water surface.

Recent research at the Swedish Geotechnical Institute (*41*) has shown that (a) a large embankment height results in a more pronounced failure of the subsoil, (b) because of smaller subsoil displacements, a narrow embankment penetrates more easily, (c) in very wide embankments, the displacement is primarily ahead of the leading edge, rather than to the sides. This results in a need for excavation and blasting (next section). The Swedish research also found that the method is *not* recommended when (a) the overall stability of the area is low, (b) the firm bottom is steeply inclined ($\geq 45°$) perpendicular to the centerline (a slide could occur sideways), (c) the strength increases substantially with depth (this prevents penetration and displacement and results in long-term settlements), and (d) clays of high strength and high sensitivity (quick) are present that are subject to sudden, rapid sliding.

The displacement method should be designed just like any other alternative; recommended procedures are given in Chapter 6. Its success may depend on the sensitivity of the soft soils to remolding, and perhaps that is why the method has been used with such success in Sweden. Just as with any other treatment technique, selection of this method should follow a detailed evaluation of it and other alternatives. Available construction control and inspection and the consequences of delays and of postconstruction settlements are very important considerations.

Displacement of Soft Materials by Blasting

The displacement of soft materials by blasting has been attempted on numerous occasions, and as mentioned above, it has been used in connection with removal by displacement. Thus, its use must be augmented by controlled placement of foundation and embankment fill. The technique is described by Sinacori et al. (*33*), Casagrande (*61*), Arman (*36*), Broms (*31*), and Hartlén (*41*); the paper by Casagrande provides the most extensive discussion of procedures for displacement blasting.

The blasting technique requires expert and constant field supervision to assure that adjustments to blasting procedures are made as conditions warrant. Unless this is done, soft material may not be completely and properly removed. This method is considered to be sufficiently sensitive and difficult to use so that it should *not* normally be considered as an appropriate alternative. Only three states report having used blasting, with one (Wisconsin) remarking, "not in the last 25 years!"

Embankment Widening

Today there is much more emphasis on upgrading of existing facilities rather than construction in completely new locations. Some of the problems associated with embankment widening will be described in Chapter 4. All four methods just described might be suitable under the right circumstances for widening existing embankments, provided the construction procedures are feasible. Moore (*34*) describes one such solution to the problem of widening embankments crossing swamps and marshes (Figure 9). Note that in order to avoid instability of the existing embankment, the length of the open excavation must be carefully controlled by keeping the backfilling operations close to the excavation. Also, the water level in the excavation must be maintained at its original elevation. There are also other soil improvement procedures that would be suitable for stabilizing widened sections to existing embankments, and these will be discussed as appropriate in this synthesis.

FOUNDATION STABILIZATION BY CONSOLIDATION

Components of Settlement

When a soil deposit is loaded by an embankment, both vertical deformations (called settlements) and horizontal deformations will occur in the foundation soil. Because these deformations may adversely affect the performance of the embankment and structures it supports (pavements, bridge abutments, etc.), predictions of these deformations is a primary obligation of the

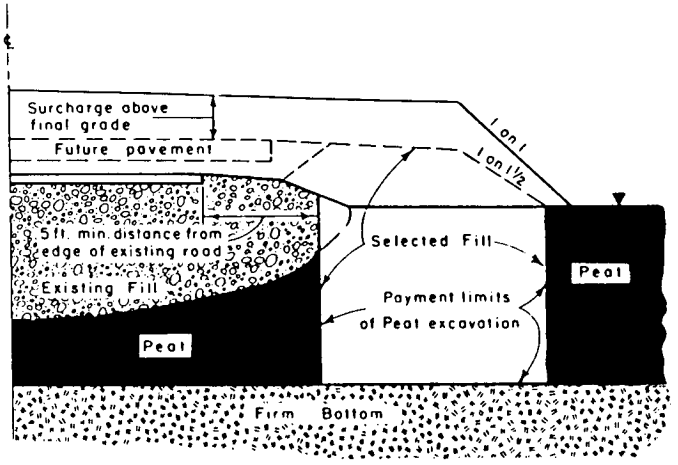


FIGURE 9 Typical section for widening an embankment on peat (*34*).

foundation engineer. The total settlement of a loaded foundation soil is usually assumed to have three components: (a) immediate or elastic settlement, (b) consolidation (time-dependent) settlement, and (c) secondary compression, which is also time dependent. For convenience, it is assumed that these components occur in chronological sequence, in the order given. The relative importance of each of these components for settlement analyses depends on the soil type, as shown in Table 10. For granular materials, only the immediate settlement is of concern. For clay soils, consolidation settlement is the major component, but immediate and secondary must also be checked. For organic soils, especially the fibrous peats, most of the settlement is secondary compression. Because of their high permeability, consolidation settlement occurs so rapidly as to almost be "immediate," and both components are usually combined for analysis purposes.

In sands and other materials with a high permeability, immediate settlements usually occur during construction and are negligible. Where necessary, they can be predicted using empirical procedures with soil properties estimated from in situ tests. For foundations on clay soils, immediate settlements are predicted using elastic theory. They ordinarily may be neglected as they occur during fill placement, especially if the upper soil layers are partially saturated. Good references on estimating immediate settlements of shallow foundations include Terzaghi and Peck (*62*), Perloff and Baron (*25*), U.S. Navy (*63*), Cheney and Chassie (*64*), and Wahls (*8*).

TABLE 10

RELATIVE IMPORTANCE OF IMMEDIATE, CONSOLIDATION, AND SECONDARY SETTLEMENT FOR DIFFERENT SOIL TYPES

Soil Type	Immediate Settlement	Consolidation Settlement	Secondary Compression
Sands	yes	no	no
Clays	possibly	yes	possibly
Organic soils	possibly yes	possibly no	yes

The other two components of settlement will be introduced below; settlement analyses for design are treated in Chapter 6.

Consolidation Concepts

When a saturated soil is loaded, pressures in excess of the existing static groundwater pressure are created. As this excess pore water pressure dissipates, the volume decreases and surface settlements occur. This process is called consolidation, and the rate of settlement depends on the permeability of the soil. Because clays have a low permeability, they take a long time to compress, but as they do, they become denser and stronger, and the rate at which they settle gradually decreases. That basically is the principle of stabilization by consolidation—foundation soils can be improved by allowing them to consolidate, gain strength, and increase their stability.

By preloading soft-foundation soils, the required reduction in volume or settlement that occurs under the final embankment load can be achieved before the roadway is paved, and hence postconstruction settlements are significantly reduced or eliminated.

The consolidation process described above is sometimes termed primary consolidation to differentiate it from secondary settlements, discussed below. It is reasonably well described by the Terzaghi one-dimensional theory of consolidation and subsequent advancements. Details are given in a number of textbooks (*17, 25, 62*) and manuals (*64–66*).

The magnitude of consolidation settlements for embankments varies from a few inches or centimetres in moderately soft soils with low embankment heights to as much as 10 or 15 ft (3 to 4.5 m), or even more where compressible soils are thick and have high water contents and where embankment loadings are high.

Secondary Compression

Secondary compression is a continuation of the volume change that began during primary consolidation, only it occurs at a much slower rate. Also it is different from primary in that it takes place at constant effective stress, that is, after all the excess pore pressure generated by the applied load has dissipated. It is thought to be caused by a plastic readjustment of the bonds between the soil particles, but the phenomenon is not well understood physically.

Secondary compression is most commonly reported in terms of the secondary compression index C_α, which is defined as the change in the void ratio Δe vs. $\Delta \log t$; or

$$C_\alpha = \frac{\Delta e}{\Delta \log t} \tag{1}$$

The amount of secondary settlement can be correlated with the natural water content of the soil, because the water content often reflects soil type (*67*). It also depends on soil plasticity and organic content and, interestingly, on the compression index C_c, because the ratio of C_c/C_α is approximately a constant for most soils (*68, 69*). For additional information on secondary compression, consult Raymond and Wahls (*68*), Holtz and Kovacs (*17*), and Jamiolkowski et al. (*70*).

For inorganic clays, the magnitude of secondary settlement is ordinarily only a small fraction of the primary settlement, and it occurs at a much slower rate. It should be noted that even though they are relatively small, secondary settlements can significantly influence the performance of, for example, bridge approach embankments. For organic materials, especially peats, the secondary component can be very large, even several times larger than the primary component. It is important to predict the magnitude and rate of secondary compression in order to effectively judge the relative merits of preloading versus other foundation treatment alternatives. Such predictions are especially important for cases of embankment widening, because when the new section is constructed, the settlement process begins all over again. How to make predictions of secondary compression will be reviewed in Chapter 6.

Preloading/Surcharge Fills

If the embankment is constructed long before the roadway is paved (i.e., if soft-foundation soils are preloaded), postconstruction settlements are reduced. Note that preloading as used here means that the load—the embankment—is applied some time before the final grading and pavement construction occur. Preloading in this sense does *not* mean placing a temporary load that is removed before application of the final load such as is sometimes done with piles or structures. Also, if the elevation of fill placed during the preloading period is above that required for the final embankment elevation (i.e., if a temporary surcharge fill is placed), more settlement will occur during a given time period than would ordinarily occur with only the required height of embankment fill. This is the basic concept of using surcharge fills, as indicated in Figure 10. Although this figure illustrates only the use of a surcharge load to obtain the ultimate primary consolidation during the surcharge loading period, the same concepts (*71*) are applicable to reduce postconstruction secondary compression settlements. These points will be discussed in Chapter 6.

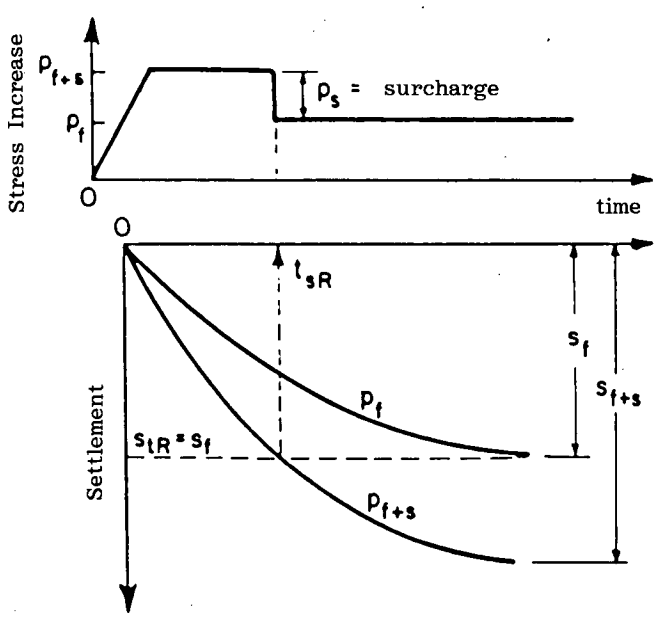


FIGURE 10 Principle of preloading by surcharge to compensate for primary consolidation settlement (after *73* and *75*).

The amount of settlement that can be achieved during any given surcharge loading period is increased by increasing the thickness of surcharge load. This benefit is partially offset by the increased cost of handling and placing the fill and subsequently removing the unneeded portion. Also berms or flatter slopes may be required because the danger of foundation instability is increased when a higher surcharge is used. This is especially true when the thickness of the soft soil is large and consolidation occurs slowly. Although the many combinations of soft-soil thickness and type make generalizations almost meaningless, thicknesses of soft-foundation soils of 10 to possibly 15 ft (3 to 4.5 m), and sometimes more if the deposit is stratified or varved, can often be stabilized by consolidation under surcharge fills only.

According to the questionnaire responses, preloading/surcharging is a very popular foundation treatment method; some 83 percent of the respondents report using it.

Good references on preloading and surcharging include Sinacori et al. (*33*), Moore (*34*), Johnson (*71*), Krizek and Krugmann (*72*), Arman (*36*), Broms (*31*), Mitchell (*32*), Jamiolkowski et al. (*73*), Hansbo (*74*), and Choa (*75*).

Controlled Rate of Loading (Stage Construction)

It is also possible to stabilize soft-foundation soils by controlling the rate of construction of the embankment, to allow for consolidation and strength gain to occur gradually with time (*33, 34, 36, 76, 77*). The main requirement is the time available, which in clays may be substantial without some type of vertical drainage system also being installed. Because of the possibility of instability, this procedure also requires a detailed subsurface investigation and geotechnical analyses (Chapter 5 and Reference *77*), and an instrumentation and monitoring program (Chapter 7).

Where extremely wide berms are required to maintain stability during the surcharge period, the use of stage construction may always be advantageous, if time permits. In this procedure, the embankment is constructed to an intermediate elevation and a waiting period allowed during which the subsoils consolidate and gain strength. Upon resumption of embankment fill placement, the increased strength of the subsoils permits construction of a higher fill, a significant reduction in the volume of the berms, and therefore a substantial reduction in cost. Waiting periods for consolidation under the first stage fill may vary from one month to a year or more.

In view of the apparent trend toward tighter scheduling and program times, there is less time available for surcharging, and this method without vertical drainage is becoming less feasible, although nearly 30 percent of the states report using the method for embankment stability problems.

Design of surcharge fills is discussed in Chapter 6.

Surcharge Loading by Water, Vacuum, and Groundwater Lowering

Although the most common type of surcharge load is earthfill, other means of applying additional surface stress are possible

and may have certain advantages in some situations. A big cost factor with earthfill surcharges is the cost of removing the surcharge. Water in ponds lined with geomembranes (*78*) could be used; although not as heavy as soil, it is almost free and disposal is simpler. The vacuum method, developed by Kjellman (*79*) and also described by Holtz and Wager (*80*), involves pumping the air from beneath a geomembrane that is sealed around the edges. Figure 11a shows the methods used in conjunction with vertical drains; Figures 11b and c show how the method could be sealed around the edges. Surcharges of 1200 to 1600 psf (60 to 80 kPa), which is roughly equivalent to 12 to 16 ft (3.5 to 5 m) of fill, are possible. Important advantages of the method are that stage loading is not needed and possible stability problems are eliminated because no large shearing stresses are imposed on the base of the embankment as they are with a granular surcharge fill. And at the end of the loading period, the surcharge can be almost instantly removed. Wellpoints have also been used to lower the groundwater level and thereby increase the effective stress to stabilize a runway constructed on silt and organic clay (*81*).

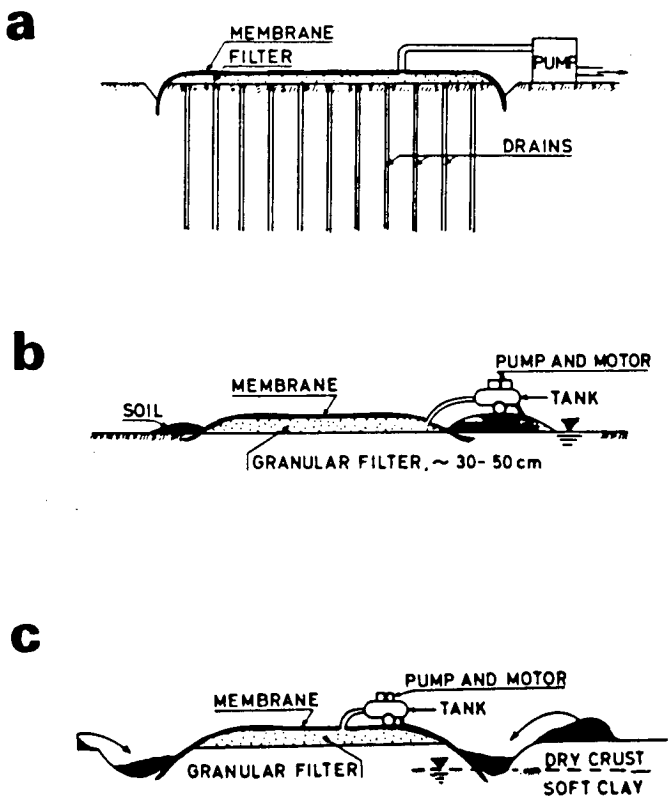


FIGURE 11 The Kjellman vacuum method (a) with vertical drains, (b) used where the water table is high and there is no dry crust, and (c) used where there is a thin dry crust and high water table (*80*).

Vertical Drains

In thick deposits of soft clay, the time required for consolidation under a surcharge may be unacceptably long or foundation instability may be a serious concern. In such cases, it is often economically advantageous to install some type of vertical drains in the foundation soil to decrease the length of drainage path for water that is squeezed from the voids in the soil. Figure 12 shows a typical installation. Because the rate of consolidation is inversely proportional to the square of the length of drainage path, the benefits of reducing the flow distance are to reduce the (a) time required for surcharge loading, (b) thickness of surcharge fill, and (c) size of berms, if any. These benefits are partially offset by the cost of the drains and the drainage blanket that is required to conduct water outside the loaded area, as illustrated in Figure 12. Furthermore, collector drains in the drainage blanket, which further increase the cost, may also be required beneath very wide fills.

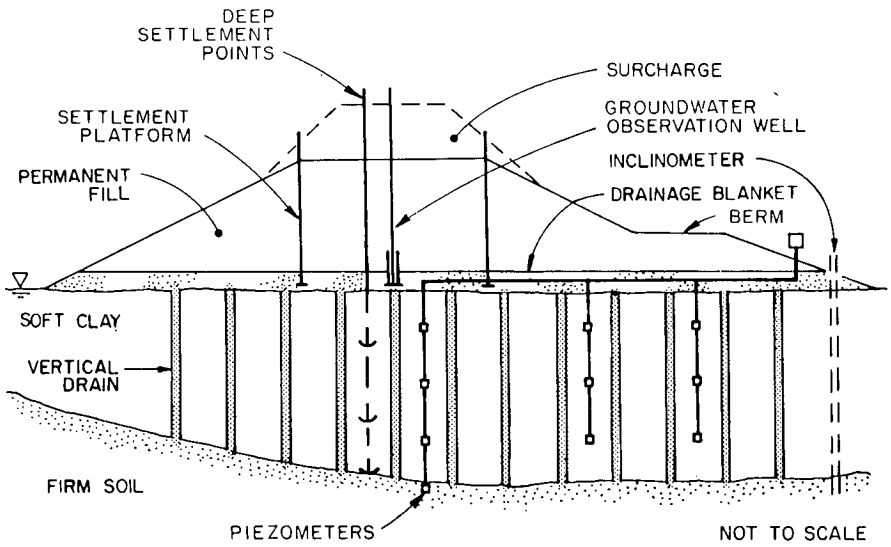


FIGURE 12 Schematic of a typical vertical drain installation for a highway embankment (*86*).

Traditionally, with one notable exception, vertical drains were made of sand. The historical exception is the Kjellman (*82*) paper "wick" drain, which is the prototype for virtually all modern prefabricated vertical "wick" or "strip" drains. With this exception, sand drains were widely used both in this country and abroad since their invention during the 1930s until about 15 years ago. Since then, a large number of prefabricated vertical drains, at present more than 50 different types, have appeared on the market. Competition has appreciably decreased their cost, and installation procedures have been improved to permit rapid, simple installations to depths approaching 200 ft (60 m) at rates up to 3 ft/sec (1 m/s). Today it is fair to say that prefabricated vertical drains have made all types of sand drains virtually obsolete.

Preloading with vertical drainage is moderately popular among the states as a foundation treatment method; 43 percent have used sand drains, with about 55 percent reporting some experience, mostly good to excellent, with prefabricated vertical drains.

The most comprehensive report on sand drains is Johnson (*83*); it is still worth consulting because much of it is also applicable to prefabricated drains. Other good sources of information about drain types, design, selection, and installation include Broms (*31*), Hansbo (*84*), Mitchell (*32*), Jamiolkowski et al. (*73*), Choa (*75*), Transportation Research Circular 309 (*85*), Rixner et al. (*86, 87*), and Holtz et al. (*88*).

Most prefabricated vertical drains are modeled after the original Kjellman "wick"; that is, they have a rectangular cross section typically about 4 in. × 0.15 in. (100 × 4 mm) and are made in long, thin strips or bands. Most drains consist of a plastic core with grooves, studs, or channels surrounded by a filter, which is most commonly made of a nonwoven geotextile, but sometimes a heavy kraft paper filter is used. Figure 13 shows some typical prefabricated drains. The core carries the excess water to the ground surface, and the filter keeps soil particles from entering the core. The core also supports the filter during installation and helps to resist in situ soil pressures.

The most common installation procedure is by a closed-end mandrel driven by an attachment to a crane or end loader (Figure 14). Depending on the site conditions and construction sequence, drilling through the embankment or upper dense layers may be required. Because the prefabricated drains are much smaller than the old sand drains, which were 6 in. (150 mm) to 18 in. (450 mm) in diameter, disturbance caused by installation is undoubtedly less than for closed-end mandrel sand drains. How much less disturbance than with other types of sand drain installation methods (e.g., jetted, augered) is uncertain, especially when the differences in drain diameter are considered. Installation patterns are similar to those for sand drains (square, triangular); typical spacings range from 4 ft (1.2 m) to 12 ft (3.6 m) but may be greater. Design considers the soil properties, drain characteristics, disturbance, and the required consolidation time to arrive at the design drain spacing; design is discussed in Chapter 6.

Stabilization of a soft foundation under a widened embankment would be possible with preloading and some type of vertical drainage. There are problems with the surcharge fill, as noted in Chapter 4, but stabilization by consolidation is only likely to be feasible for such projects when done together with vertical drainage. In transition sections where the thickness of the compressible layer decreases and/or the height of the embankment decreases, the spacing and/or lengths of the drains can be varied to achieve a more uniform settlement of the fill.

According to Rixner et al. (*86*), installed costs of prefabricated drains are between $0.75 and $1.00 per linear foot ($2.50 to $3.25 per metre), although some recent projects have been bid with prices less than $0.50 per foot. These costs do not consider the cost of a drainage blanket, working mat, mobilization, predrilling, etc. For comparison, sand drains recently have cost around $4 or $5 per linear foot ($13 to $16 per metre).

Vertical drains will be most efficient in those soil deposits in which primary consolidation settlement is important. Because vertical drains are not useful for, nor intended to reduce, secondary compression settlements, they are ordinarily ineffective in organic materials, particularly peats, because of their high permeability. However, peats are frequently underlain by soft soils into which drains are sometimes beneficially installed, as borne out by field tests in Poland (*89*). When properly designed, vertical drains may be especially beneficial in soils with a highly developed macrofabric, because the vertical drains permit intermediate pervious layers to function as horizontal drains. However, as noted by Holtz et al. (*88*), to achieve an economical drain design at such sites requires a very high quality subsurface investigation and laboratory testing program (Chapter 5).

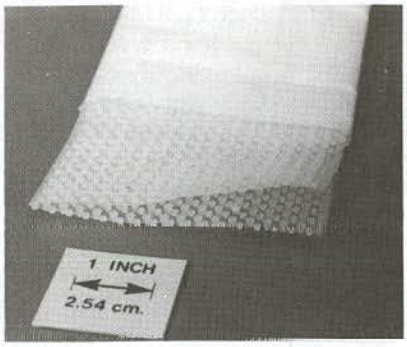
Preloading and Pressure Relief Wells

Some soil deposits are very strongly stratified with continuous silt and sand layers, which are capable of serving as drainage layers, thereby accelerating consolidation under the surcharge fill. This effect can eliminate the need for vertical drains. However, where the base width of an embankment is large, the volume of water squeezed from the soft soil into the horizontal drainage layers must flow laterally a long distance. This can result in large head losses and high excess pore water pressures in the intermediate drainage layers, which reduce their efficiency. This effect has been observed even in varved soils, where the pore water pressures beneath the central portion of a wide embankment were almost as high as they would be if intermediate drainage layers were not present. This is dangerous because instability may result either at the toe of the embankment or beyond, as indicated in Figure 15. Where this occurs, vertical drains or even wellpoints can be installed near the toe of the embankment to decrease the pore water pressure in the intermediate drainage layers. In some cases more than a single line of drains at the toe of the embankment is required. This can be determined by appropriate analysis and/or field trials.

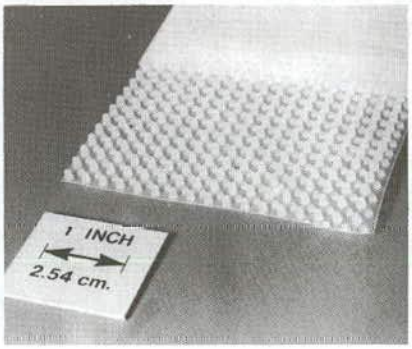
A similar situation exists beneath fills where soft-foundation soils are underlain by sands. In this case also, the pore water pressure in the sand can become high unless relieved by drains or small pressure relief wells installed into the underlying sand stratum. The influence of width of embankment relative to the thickness of compressible soil is illustrated in Figure 16.

Other Drainage Systems

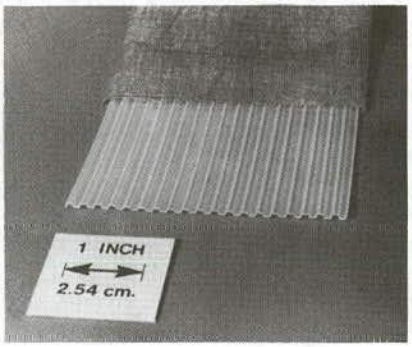
Minnesota reports success in increasing embankment stability by simply lowering the groundwater level by means of trench or french drains containing a perforated pipe and pervious backfill. Idaho also has had average to good success with stabilization



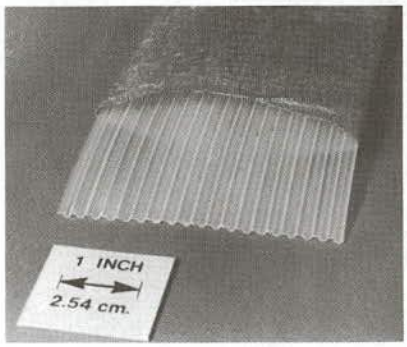
a. Alidrain



b. Alidrain S



c. Amerdrain 307



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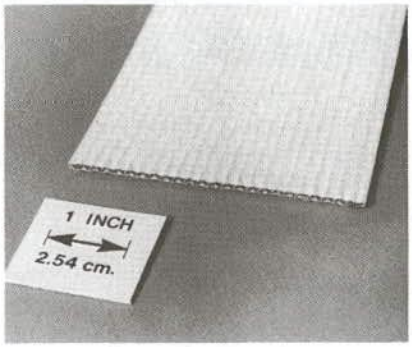
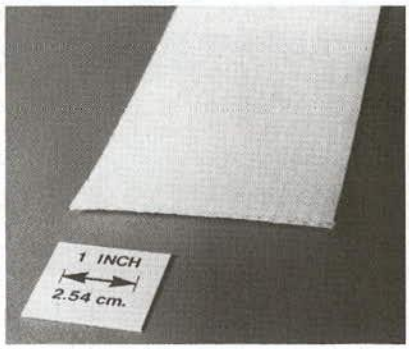
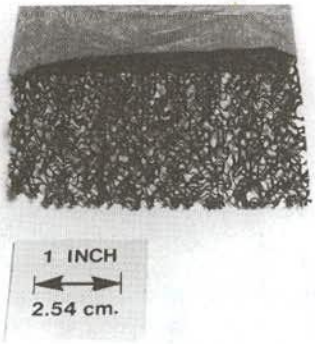


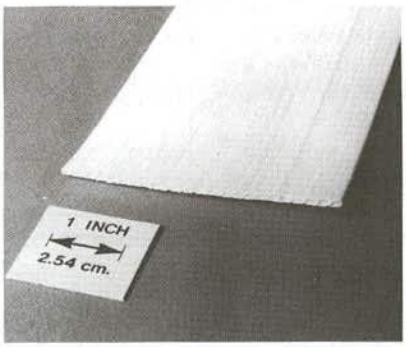
e. Bando



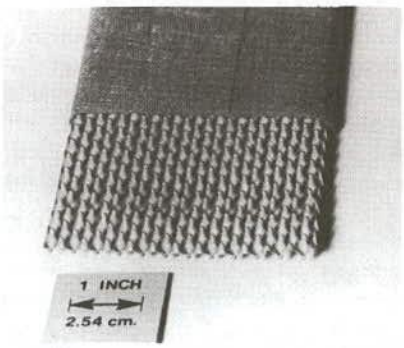
f. Castle Drain Board



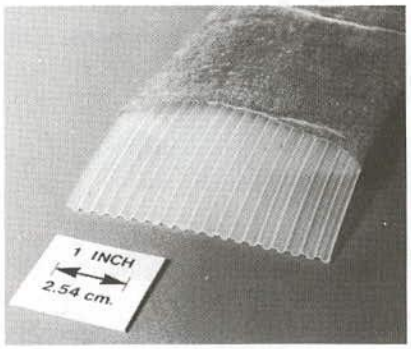
g. Colbond CX-1000



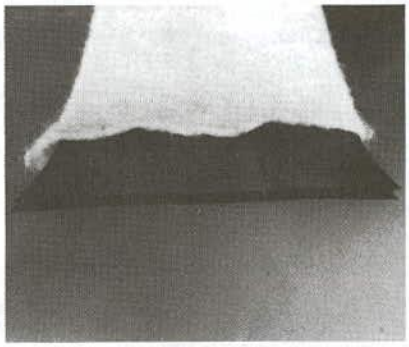
h. Desol



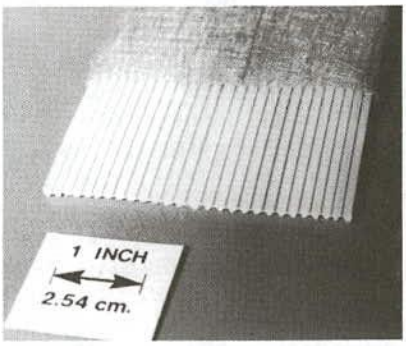
i. Hitek Flodrain



j. Mebradrain MD 7007



k. Sol Compact



l. Vinylex

FIGURE 13 Typical prefabricated band-shaped vertical drains (*86*).

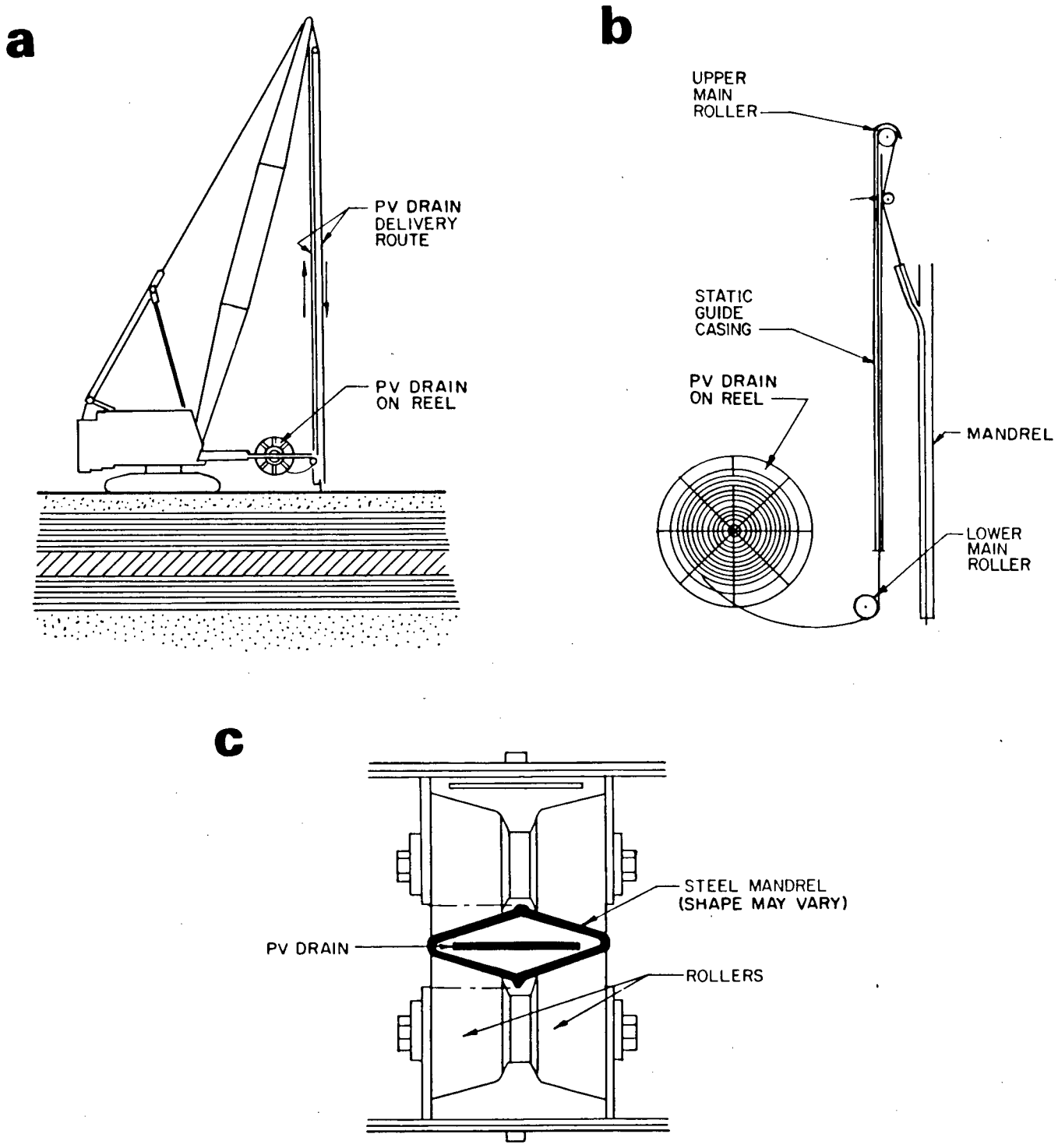


FIGURE 14 Installation of prefabricated vertical (PV) drains: (a) crane-mounted installation rig, (b) drain delivery arrangement, and (c) cross section of mandrel and drain (*86*).

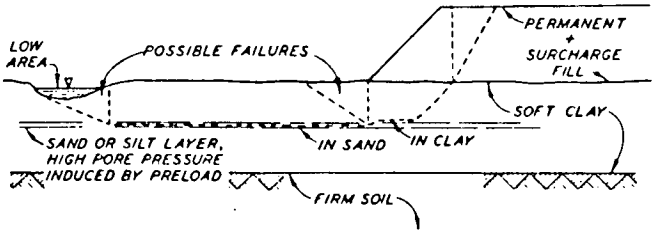


FIGURE 15 Effect of preload-induced pore water pressure on stability (*71*).

by drainage alone when a high water table is encountered in upland areas.

CHEMICAL AND THERMAL ALTERATION/ STABILIZATION

Judging by response to the questionnaire, chemical and thermal stabilization are not very popular for treating problem foundation soils. Only three or four states report using lime columns, grouting, or chemical stabilization; none report using electroosmosis or thermal stabilization. The reasons for this lack of use are probably related to the cost of the methods as well as a general unfamiliarity with the techniques. Although some of the methods appear to be unsuitable for conventional highway embankments, there may be locally important conditions, such as a bridge approach embankment, where some of these techniques might prove feasible.

Lime Columns, Cement Columns, Etc.

Soil stabilization by admixtures such as lime and portland cement is well known and until recently only applicable to compacted soils. But during the past 10 to 15 years, equipment and procedures have been developed that can apply and mix the stabilizers in situ to make lime and cement columns, which have been successfully used to stabilize highway embankments on soft inorganic clays. Lime columns were developed in Sweden and they have been used mostly in Scandinavia; cement columns that use similar equipment are used mostly in Japan. Total annual production in Sweden, Norway, and Finland is more than 260,000 m (850,000 ft), with about 40 percent of this total used for road foundations. As production was less than one-tenth this amount only 10 years ago, it appears that the method may have considerable promise for soft-ground stabilization in the United States. Further, recent research at the Swedish Geotechnical Institute has shown that using gypsum and fly ash together with quicklime also improves organic soils and silts as well as inorganic clays (*90, 91*).

Good references on lime columns include Broms and Boman (*92–94*), Broms (*31, 95*), Mitchell (*31*), Broms and Anttikoski (*39*), and Sutton and McAlexander (*96*). Åhnberg and Holm (*97*) summarize 10 years of research and practical experience with the method, but unfortunately it only has a summary in English.

Although a number of uses for lime columns have been suggested, the most feasible applications are for improving the stability of natural slopes and excavations and for reducing the settlements of shallow foundations, especially for highway embankments and small structures. The columns as used in Sweden are 0.5 to 0.6 m (20 to 24 in.) in diameter with a maximum length of 15 m (50 ft). For deposits of soft clays less than 15 m, the entire depth can be stabilized; for deeper deposits, the stabilized zone acts like a deep "dry crust," which serves to spread the loads and diminish but not totally eliminate settlements. Still, as will be seen, the overall improvement is excellent, and very appropriate for structures such as highway embankments that can tolerate some settlements.

Figure 17 shows schematically how the columns are made. The mixing tool (Figure 18) is quickly augered down to the desired length of the columns. Then the direction of rotation is reversed and the tool is slowly rotated out of the ground as unslaked lime (CaO) or other additive is forced out into the soil at a controlled rate. The rotary table and kelly are on a mast, which is mounted on a special off-road vehicle as shown in Figure 19. The vehicle also tows a container with the lime supply, typically sufficient for 10 or 15 columns.

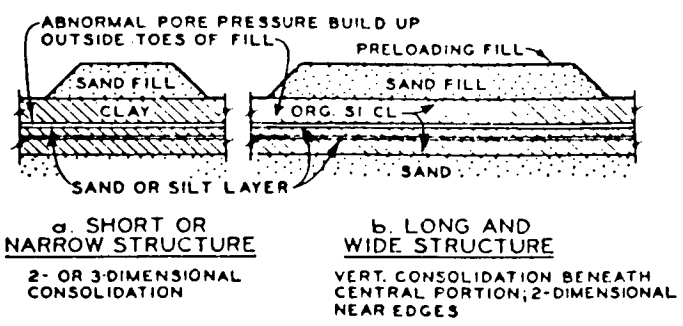


FIGURE 16 Influence of embankment size relative to thickness of compressible soil (*71*).

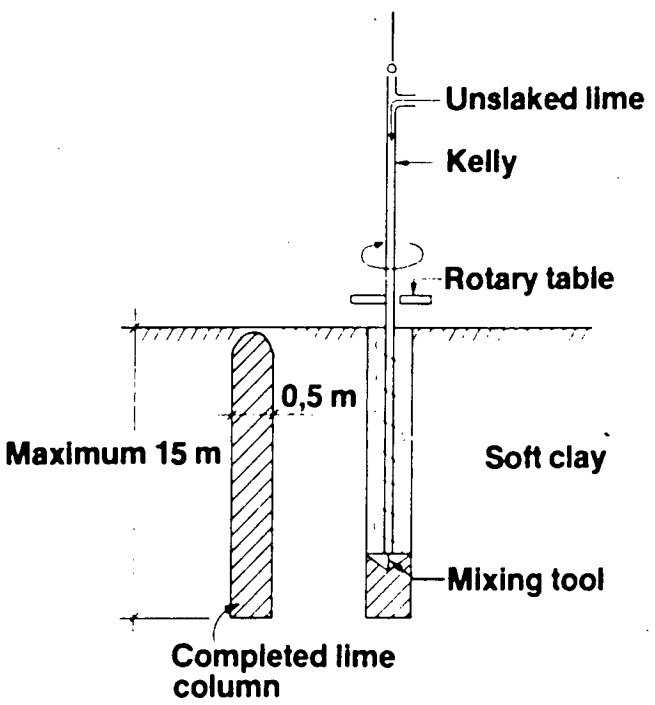


FIGURE 17 Manufacture of lime columns (*92*).

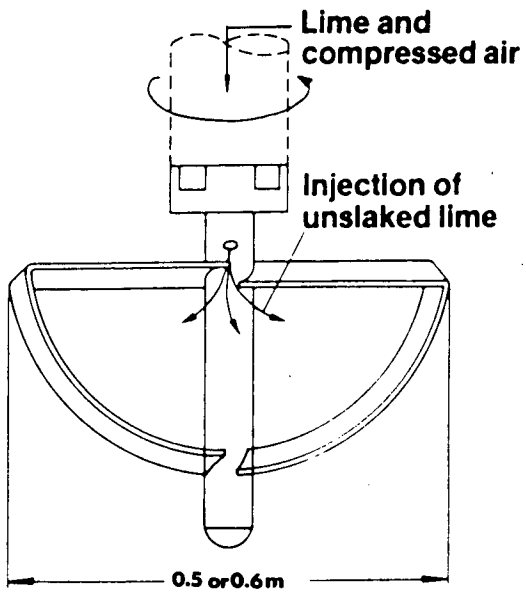


FIGURE 18 Swedish mixing tool for lime columns (*92*).

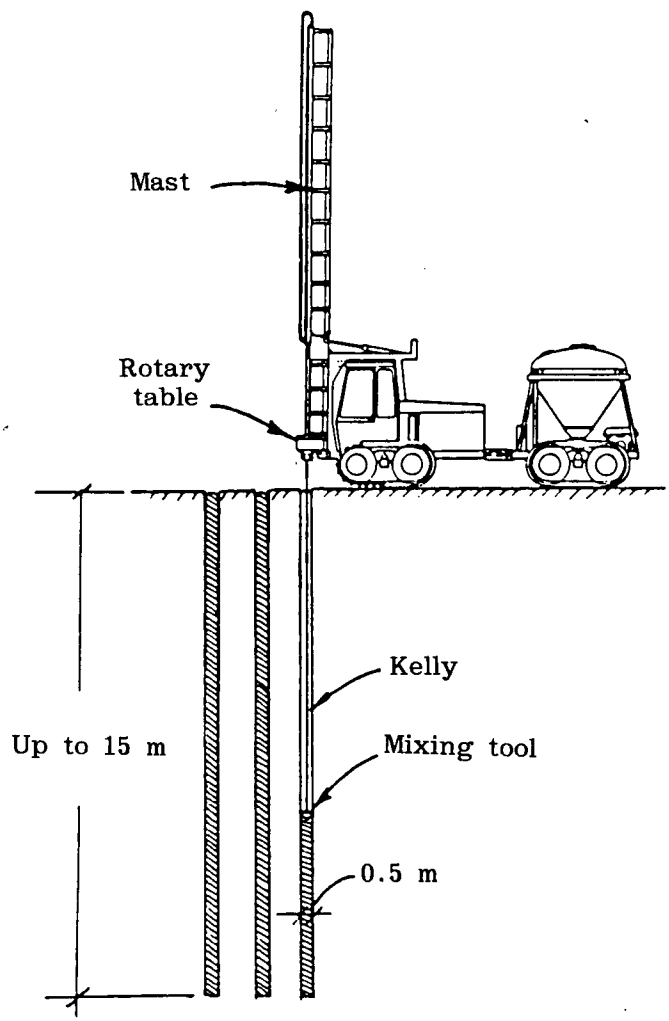


FIGURE 19 Lime column machine (*95*).

Production rate depends on soil type, additive, depth of treatment, etc. Generally, it takes about 10 to 15 minutes per column with lime (somewhat longer for portland cement, as more mixing time is required). On the average, one machine can produce 300 to 400 m of columns per 8 hour shift. Figure 20 indicates how lime columns are used to reduce settlements under road embankments. Normal spacings are 0.8 to 1.8 m (2.5 to 6 ft) center-to-center.

According to Åhnberg and Holm (*97*), the lime column method has been used for more than 20 highway embankment projects in Sweden to help reduce excessive total and differential settlements. They have performed very well, even better than originally anticipated. Not only are total settlements significantly reduced (in some cases by a factor of two; see Reference *98*), but differential settlements, both transverse and longitudinal, are reduced as well (*99*). The method seems especially attractive for bridge approach embankments. In one interesting case, columns of different lengths and spacings were successfully used to even out the settlements of an embankment on soft clays approaching a roadway section that had been improved by excavation and replacement with blasted rockfill (*99*).

Stabilization with unslaked lime is apparently quite acceptable for inorganic clays, but its effectiveness decreases as the organic content increases. Also, silts are difficult to stabilize with lime only. Portland cement appears to be somewhat better in this regard, as do gypsum and some fly ashes. Mixtures of these products with lime also lead to significant improvement for both inorganic as well as organic soils, and with gypsum-lime mixtures, the rate of strength gain is about twice as fast as with lime alone, although the ultimate strengths were about the same (*90, 91*).

Lime columns seem to be a possible solution to the problem of widening and grade raising of existing embankments on soft foundations, although as far as is known, the method has not been used for this purpose.

Design of embankment foundations stabilized with lime columns will be discussed in Chapter 6.

Grouting and Injections (Chemical, Slurry, Jet, Compaction)

Grouting and injection into soils involves (a) permeation, (b) displacement, densification, or compaction, and (c) encapsulation (*32*). Grout types include particulate materials (portland cement and soil-cement mixtures), chemicals (primarily silicates), and slurries (lime, bentonite, etc.).

As noted by Arman (*36*), the use of grouting to stabilize soft soils has been limited to corrective measures for postconstruction problems because of its relatively high cost and the absence of precise methods to predict and evaluate the results of the procedure. It has been a method used when nothing else would work. However, recent developments, particularly in compacting and jet grouting, are promising, and there may be cases where grouting would be a feasible solution to a specific embankment foundation stabilization problem. Deposits where grouting might prove to be feasible include locally important deposits of loose or collapsible soils, waste landfills and dumps, loosely backfilled strip-mined areas, and karstic and cavernous limestone areas, to name a few.

Besides Arman (*36*) and Mitchell (*32*), useful information on

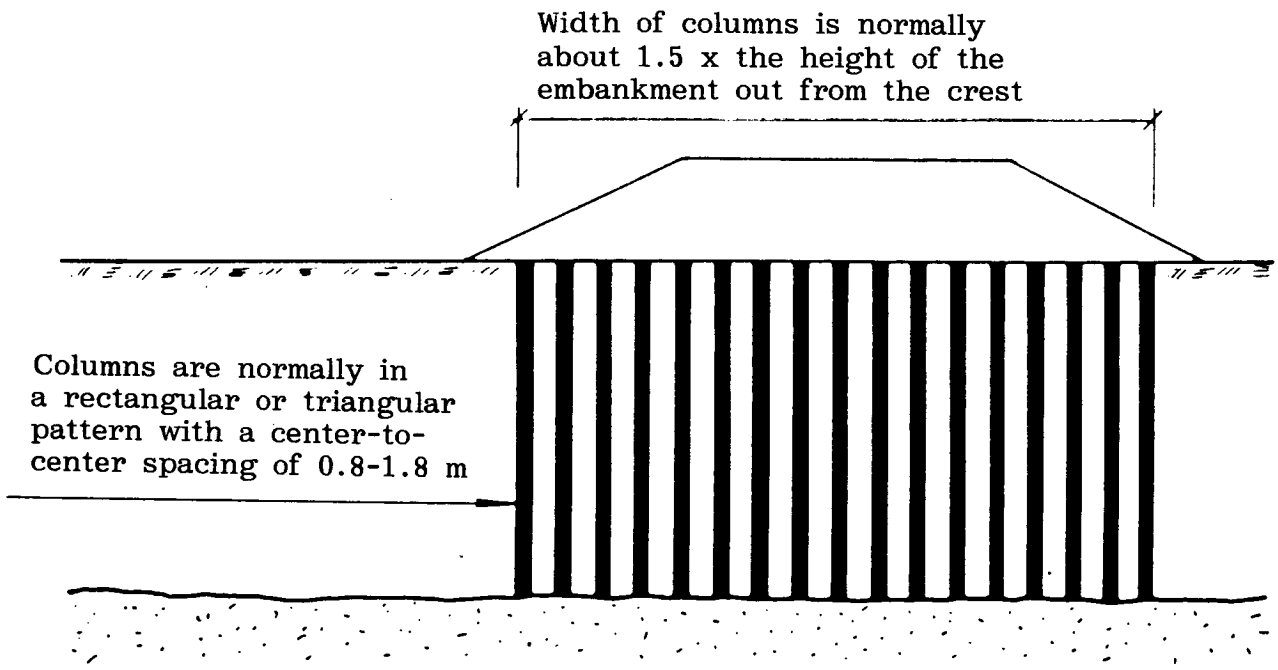


FIGURE 20 Lime columns under a highway embankment used for settlement reduction (after *97*).

all aspects of grouting can be found in References *39, 96,* and *100–108.*

Electro-Osmosis and Electro-Kinetic Stabilization

Electro-osmosis can be used to dewater and consolidate soft clays and silts. When a direct electrical current is applied to a soil mass between two electrodes, water flows from the anode toward the cathode. If water is removed from the region around the cathode, the water content then decreases in the region of the anode, and consolidation and strength gain occur.

Electro-osmosis for stabilization of problem sites has primarily been applied to unstable slopes and structural foundations (*109–111*). However, Fetzer (*112*) used the technique to stabilize the foundation of a dam in Ohio on lacustrine clay interspersed with layers of silt and silty sand (see also Reference *113*). Chappell and Burton (*114*) used it to stabilize the foundation of an 8 m (26 ft) high embankment to be used for a cofferdam on very soft silty clay soils in Singapore. Instability of the embankment occurred during construction, and because of the low permeability of the soil, drainage by gravity was not possible in the short time available for construction. Other stabilization procedures, such as sheet piling and toe berms, were considered more expensive than electro-osmosis. After a field trial to obtain design information, the entire 60 m (200 ft) long section was successfully stabilized.

The method is probably too expensive for most highway embankment foundations, but References *31, 32, 36,* and *105* contain additional information.

Electro-kinetic stabilization is a hybrid between electro-osmosis and chemical grouting. It seems that the infusion of certain stabilization chemicals into especially silty and clayey soils is made more efficient by the application of an electrical potential difference to the soil mass. The procedure seems to be more effective in silty soils that are otherwise difficult to grout ordinarily. Information on this technique can be found in Broms (*31*) and Mitchell (*32*).

Thermal and Freezing Methods

Soil stabilization can occur by both heating and freezing. With heating, it is necessary to increase the temperature to above 100°C (212°F) to promote drying, consolidation, and an increase in strength. Higher temperatures, e.g., 600 to 1000°C (1100 to 1800°F), can produce even more stable soils, and very high temperatures can actually fuse the mineral grains, as in pottery and brick making. Most published applications of deep thermal stabilization have been in the USSR and Eastern Europe, where, apparently, energy is relatively cheap. For information, see Broms (*31*), Mitchell (*32*), and Broms and Anttikoski (*39*).

Frozen soils are quite strong and relatively incompressible in comparison to their thawed state, so artificially freezing the ground is also a potential stabilization method. Probably because of cost, it has primarily been used for temporary support of excavations and tunnels. One of the most complete references on ground freezing is Jessberger (*115*). For additional information, see References *31, 32,* and *39.*

Organic Stabilization

The use of living organisms to stabilize soils sounds quite esoteric, but the roots of trees and shrubs are very successful in stabilizing slopes (*116–118*). It seems unlikely such procedures will ever be applicable for embankments, but one never knows. Perhaps plants could be induced to send their roots out horizontally under an unstable embankment. The method would be especially attractive if rapid, deep growth could be motivated.

Enzyme reduction and geoworms are two other organic possibilities (*119*). The first concept is to introduce certain bacteria into the soil to fix sulfur, toxins, etc.; applications to waste dumps and landfills are a possibility. Geoworms are similar to silkworms; a modification of their genetic makeup (DNA, etc.) enables them to "worm" their way through loose and unstable soils while leaving an organic thread intertwined in the soil. The improved soil mass is similar to "texsol," the French system of mixing synthetic fibers in with sand (*120, 121*). A few problems, such as how to feed them and how to get them to go in the desired directions, still remain to be solved before geoworms will be a feasible soil stabilization alternative.

PHYSICAL ALTERATION/STABILIZATION—DENSIFICATION

Although granular soils generally make good foundations, loose deposits can settle too much when loaded, or if they are saturated, they can liquefy because of earthquakes or other vibrations. Loessial soils and some desert soils are collapsible, especially when they become wet. Finally, municipal and mine wastes often are loose and highly compressible. Most of the techniques described in this section are applicable to loose granular materials, collapsible soils, and some waste fills and dumps. The exceptions are some vibro-replacement methods, including stone columns, which can also be used in cohesive soils.

Dynamic Compaction

Densification of a deposit by repeatedly dropping a heavy weight onto its surface is called dynamic compaction. Typical weights are between 6 and 30 tons (5 and 27 Mg) and drop heights range between 30 and 75 ft (9 to 23 m). Spacing between the drop points depends on the soil type, thickness of the deposit, and the location of the groundwater table, but ranges between 7 and 30 ft (2 to 15 m). Figure 21 is an aerial view of the grid pattern produced by dynamic compaction of a landfill for an Interstate highway embankment in Indiana. Conventional crawler cranes can be used for weights up to 20 tons (18 Mg) and drop heights up to 100 ft (30 m). Beyond that, conventional equipment must be modified or special equipment must be used. Dynamic compaction is very effective in compacting loose partially saturated sands, municipal wastes, strip-mined areas, building rubble, loosely dumped rockfill, and the like. With saturated sands, liquefaction occurs upon impact, and as the excess pore pressures dissipate, densification occurs. Densification of finer-grained and less permeable materials is somewhat questionable, especially if they are saturated, because of the time required for the excess pore pressures to dissipate. It is possible that the use of prefabricated drains together with dynamic compaction could be used at such sites. Dynamic compaction of highly organic and peat deposits is not recommended.

Depth of improvement D in metres by dynamic compaction can be estimated by the equation

$$D = n\,(W\,H)^{1/2} \qquad (2)$$

where W = mass of weight in metric tons,
H = height of fall in metres, and
n = an empirical coefficient.

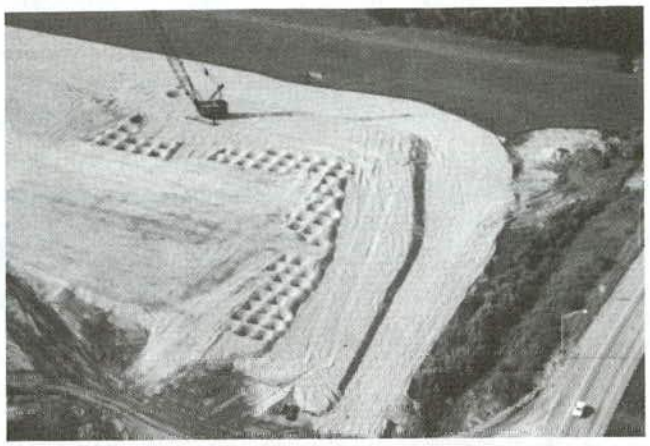


FIGURE 21 Aerial photograph of Interstate highway construction in Indiana in which landfill is being densified by dynamic compaction. (Photograph courtesy of J.P. Welsh, GKN Hayward Baker Inc.)

The coefficient n has an average value about 0.5 and a range of 0.3 to 0.8. It is necessary to account for factors other than the mass of the falling weight and the height of drop that influence the depth of improvement. The degree of improvement that can be expected is large; some soil properties will be improved at least 100 percent, and in some cases up to 400 percent increases have been observed.

A major research project on dynamic compaction sponsored by FHWA has just been completed, and the project report by Lukas (*122*) is very comprehensive and highly recommended. The report describes the deposits where dynamic compaction is suitable, the kind of improvement attainable, suggestions for project monitoring, and a description of off-site effects such as vibrations and lateral displacements. Other useful information is included on equipment required, contracting, costs, and specifications. Additional information on dynamic compaction can be found in References *31, 32, 74, 105*, and *123–125*.

According to the questionnaire responses, 22 percent of the states have used dynamic compaction with good success, for both new construction as well as for upgrading facilities.

Design of dynamic compaction projects will be discussed in Chapter 6. Special problems associated with its use for densification of wastes and collapsible soils will be discussed in Chapter 4.

At some sites and foundation conditions, it is possible that dynamic compaction would be appropriate for improving the foundations of an embankment next to an existing facility, such as might be required for facilities upgrading.

Blasting

Detonation of buried explosives can very effectively densify deposits of clean, saturated, uncemented sands. The process is similar to that of dynamic compaction, in that liquefaction of the deposit results in subsequent densification. With greater depths, because the effective stresses are greater, larger charges are required to achieve the same result. Mitchell (*32*) reports that equivalent relative densities of 75 to 80 percent are possible,

but that the results are likely to be somewhat erratic, depending on how homogeneous the deposit is initially. Dense layers and lenses are likely to be loosened somewhat, but the overall improvement will probably be satisfactory. An almost immediate surface settlement of between 2 to 10 percent of the layer thickness will occur after blasting. There may be some problems with evaluating the effectiveness of the blasting by means of penetration tests (e.g., SPT, Dutch cone) because these tests often show little improvement immediately after blasting; however, after several weeks, they do indicate that the density has in fact increased (*126*). This phenomenon of time effects in sands is not well understood. Solymar (*127*), Solymar et al. (*128*), and Solymar and Mitchell (*129*) describe an interesting project in which a very deep, loose sand deposit was densified by blasting and different vibrocompaction techniques.

Besides in Mitchell (*32*), useful information on densification by blasting can be found in Janes and Anderson (*123*), Hansbo (*74*), and Smoltczyk (*124*). Design information will be given in Chapter 6.

Except for rather special circumstances, it does not appear that blasting would be appropriate for embankment widening or grade raising because of potential difficulties with the existing embankment foundation.

Vibrocompaction Methods—Granular Soils

Another way to densify loose deposits of clean granular materials is to insert some type of vibrating tube, probe, or blade at different locations into the deposit. Sometimes water jets or air pressure are used to aid in insertion, and usually the probe is repeatedly inserted and withdrawn at the same location. In the vibroflotation and sand compaction pile systems, water and additional sand are added during withdrawal of the probe so that a column of denser granular material remains after the process is completed (Figure 22). Other systems include the Foster Terra-Probe, Vibro Wing, and the Franki V-Probe.

Although vibrocompaction methods have mostly been used to improve building sites, some of the projects have been very large, which suggests that these procedures in principle could also be used for highway embankments. Useful references for these methods include *32, 74, 105, 123-125, 128, 130,* and *131.*

According to the questionnaire responses, only about 10 percent of the respondents have used vibrocompaction methods. Design will be covered in Chapter 6. Vibrocompaction methods would appear to be suitable for upgrading facilities at appropriate sites.

Vibroreplacement Methods—Cohesive Soils

For cohesive soils, vibration alone will not improve the soil, so some other material such as sand (sand compaction piles), sand and gravel (sand-gravel piles), or gravel (stone columns) is introduced into the soil and densified, usually by vibration. Sometimes, stone columns are grouted, or, in a few cases, instead of clean gravel or stone, concrete is used instead. Interestingly, hand-dug stone columns were used to support the Taj Mahal in India (*132*).

Sand and stone columns are made in a manner very similar to the vibrocompaction methods described above (Figure 23) (*133*). A vibro probe 12 to 18 in. (0.3 to 0.45 m) in diameter is pushed or jetted into the soft soil; clean gravel or crushed stone is dumped into the hole in 1 to 4 ft (0.3 to 1.2 m) lifts and vibrated repeatedly to densify the stone and to force it into the surrounding soil. If the soils are too soft for the hole to stand open, the probe is left in the hole and flowing water is used to stabilize the hole and wash out fines, as stone is dumped into the annulus between the hole and probe. Some probes are available with tubes that deposit the stone at the bottom of the hole while the probe is in place ("dry bottom feed"; *134*). As a consequence of all these operations, the diameter of the completed columns is somewhat larger than the probe diameter; a typical range is between 2 and 4 ft (0.6 to 1.2 m).

Usually between 15 and 35 percent of the volume of the soft soils is replaced by stone. The stone columns are installed in a triangular or rectangular grid pattern at typical center-to-center spacings of 5 to 12 ft (1.5 to 3.5 m). Common lengths are between 20 to 40 ft (6 to 12 m), although some columns as long as 60 ft (18 m) have been constructed (*134*). Design loads on the

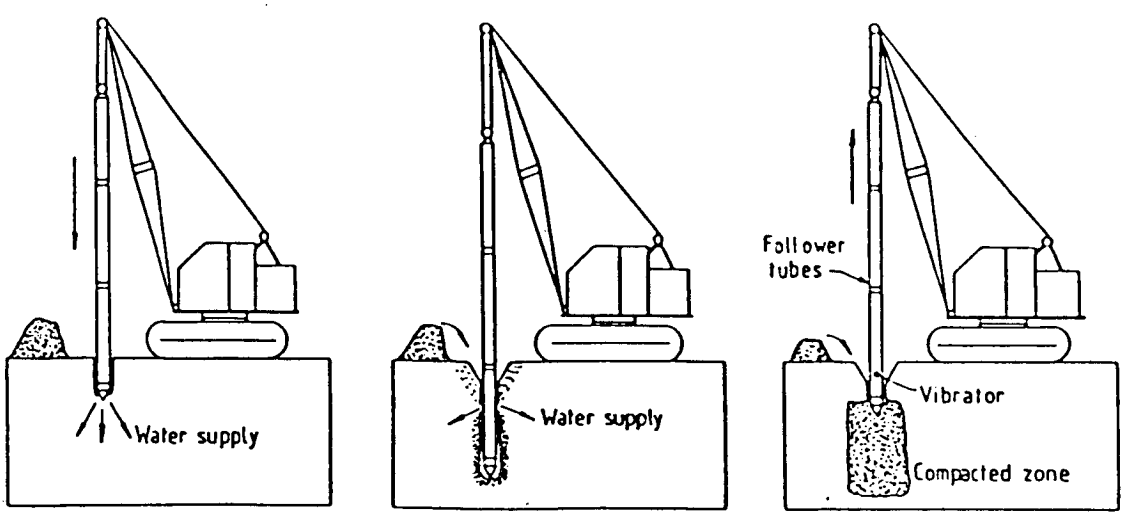


FIGURE 22 Vibrocompaction equipment and process (*125*; used by permission of ASCE).

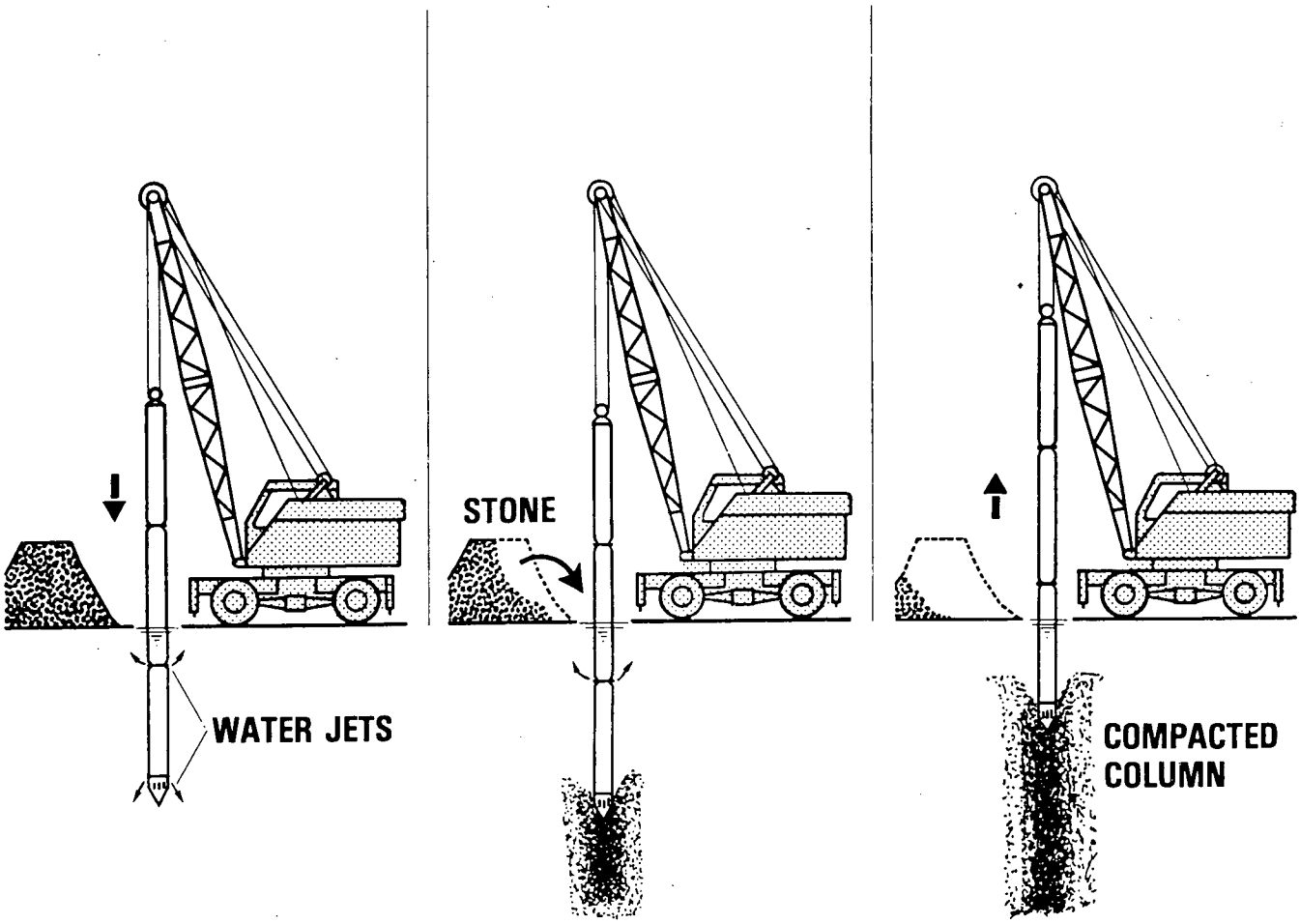


FIGURE 23 Stone columns constructed by vibroreplacement (*133*).

columns range from 20 to 50 tons (180 to 440 kN). They have been used to increase the bearing capacity and reduce the settlements of highway embankments.

Vibroreplacement methods are appropriate for a rather wide range of soils including silty sands down to clays. Stratified sands or sands with some clay that cannot be effectively treated by vibrocompaction methods are good candidates for stone columns. They are not recommended for highly sensitive clays nor for peats or other organic materials. The methods work best in soils with shear strengths between 600 to 1200 psf (30 to 60 kPa), although they have been used in clays with strengths as low as 300 psf (15 kPa).

Depending on design requirements and subsurface conditions, stone columns would appear to be quite suitable for upgrading and widening existing embankments, provided they could be economically installed.

The FHWA has sponsored considerable research on stone columns, and the reports by DiMaggio (*133*) and Barksdale and Bachus (*135*) are recommended. Additional useful information can be found in References *31, 32, 39, 105, 124, 131*, and *136–138*. Design for stone columns will be discussed in Chapter 6.

Nearly 25 percent of the states report having had some experience with stone columns, mostly for new construction. Most respondents rated their experience as satisfactory to highly successful, although a couple reported highly unsatisfactory results. One failure is worth mentioning (*139*). An earth-reinforcement bridge abutment constructed on a stone column foundation failed during construction, partly because of (a) too rapid construction, (b) contamination of the columns, which lengthened the consolidation time, (c) weakening of the foundation soils by vibration and water jetting, and (d) an overestimation of the safety factor for short-term stability. A concentration factor (see *135*) of 2 was assumed in design; back analysis after the failure indicated this factor was only about 1. Thus the load of the earth-reinforcement wall was not successfully transferred to the stone columns from the low-strength clays and silts in the foundation. Although the anticipated cost savings were not realized, the lessons learned from the failure have already led to improved design procedures.

Inundation

Collapsing and subsiding soils are rather common in some localities, and they can pose serious problems for the foundation engineer. Examples include loessial soils as well as the remains of mudflows, slope wash, alluvial fans, etc. in arid regions. These soils have in common a loose, open, "honeycomb" structure

that collapses when wetted (thus the term "hydrocompactible"), loaded, or even vibrated. These soil deposits and their treatment will be discussed in some detail in Chapter 4.

According to Bara (*140*), the most economical treatment method for collapsible soils is inundation. This can be done in two ways (*105*): (a) by pumping from wellpoints, so that seepage forces are directed downward, applying a consolidating force, and (b) by prewetting, flooding, or ponding with water over the area to be densified. Depending on the nature of the cementation or bonding between soil grains, inundation will result in a compression of up to 8 or 10 percent of the thickness of the collapsible soil layer. This increases the density of the layer and thereby reduces its compressibility under surface loads such as embankments.

REINFORCEMENT

The concept that in some situations soils require reinforcing is not particularly new. Tree roots, for example, can be quite effective as slope reinforcement. Some 3000 or more years ago, very large temples ("ziggurats") in Mesopotamia were constructed of soils reinforced with woven reed mats, and the Great Wall of China (200 B.C.) contains sections constructed of mixtures of clay and gravel reinforced with tree branches. Reinforcement of earthen revetments and fortifications has been done in Europe since Roman times, perhaps earlier (*141*).

Embankments on soft foundations sometimes require reinforcing. Examples include levees and roads constructed directly on brush fascines, logs or timbers (corduroy), and bamboo fascines (*31, 142*). A modern counterpart of these systems is the Columbus vehicle and fascine mat developed in Sweden about 25 years ago (*143*).

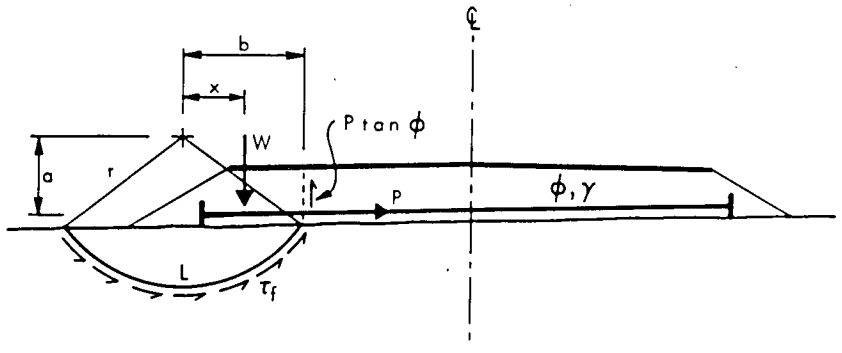
Casagrande, in his lectures at Harvard University, mentioned reinforcing embankment dams with steel rods and plates. The idea resulted from an understanding of the earth pressures developed within embankments constructed on compressible foundations (see *144*), but it was rejected as being too costly for dams. Tie rods were used by Terzaghi (*145*) to stabilize an ore pile confined between two large retaining walls in Cleveland, but as far as is known, they were never used for highway embankments until a practical and sometimes economical variation of this principle was developed by Wager at the Swedish Geotechnical Institute. In the Wager system, two rows of short sheet piles or steel channel sections under each crest are connected by steel tie rods to increase the stability of embankments constructed on soft foundations (*146*). Full-scale tests were conducted in the 1960s, which proved the system worked and verified some design rules (Figure 24). These rules are the basis for one of the design methods used today for geotextile-reinforced embankments. The Wager system has been used more than 30 times in Sweden and Denmark for both highway and railroad embankments. The system is expensive, and consequently it has only been used when no other treatment alternative is feasible for one reason or another.

Instability of an embankment on a soft foundation results in part from the weight of the overlying embankment and in part from a tendency of the fill to spread laterally because of internal earth pressures in the embankments. Both factors impose additional shear stresses on the foundation soils, and if these stresses are greater than the shear strength of the soil, failure will occur. The foundation must, of course, have sufficient overall bearing capacity to carry the vertical load of the embankment. To be effective, any type of reinforcement placed at the interface between the embankment and the foundation must reduce the transference of horizontal shear stresses from the embankment to the foundation. This is the reinforcement principle of fascines, corduroy, and the Wager short-sheet-pile method.

In addition to the materials already mentioned, embankments have also been reinforced with plastic and steel nets and meshes, steel landing mats, used automobile tire casings (*147*), and geotextiles and geogrids. Holtz (*142, 148*) has summarized a number of these developments for both embankments and retaining walls.

Geotextiles and Geogrids

In only a very few years, geotextiles and geogrids have joined the list of possible solutions to a number of important problems in transportation and geotechnical engineering. In many cases, the use of a geotextile or geogrid can significantly increase the



$$FS = \frac{\tau_f L r + P a + P (\tan \phi)\, b}{W x} \geq 1.5$$

FIGURE 24 Design procedure for the Wager short-sheet-pile reinforcing method (*146*).

safety factor, improve performance, and reduce costs in comparison to conventional solutions. For embankments on extremely soft foundations, geotextile reinforcement has actually enabled the construction to be successfully completed at sites where all the other techniques discussed in this chapter were either impossible or prohibitively expensive (*149*).

There are few developments in geotechnical engineering in recent years that have had such a strong impact on practice as geotextiles and related materials. In terms of growth rate and influence, they probably rank with the programmable calculator and microcomputer. The growth in the number of materials available is phenomenal. In 1970, there were only five or six geotextiles available; today about 250 different materials are sold in the United States as geotextiles, and worldwide, the number probably approaches 400. The size of the geotextile market is also indicative of their influence. In 1986, it is estimated that about 200 million ft^2 (19 million m^2) geotextiles and geogrids were sold in North America alone. Worldwide consumption is probably twice that amount. Because the total cost of the construction involving geotextiles and geogrids is at least four or five times the cost of the material itself, the impact of the materials on civil engineering construction is very large indeed.

The various concepts for using geotextiles and geogrids to reinforce embankments on soft foundations are shown in Figure 25. The most common approach is the system in Figure 25a; Figure 25b is a variation of the Wager method (*146*) but using a geotextile instead of steel tie rods. As far as is known, this system has not been used. The concept in Figure 25c is supposed to provide anchorage for the reinforcement; this is a controversial matter at present, and really is only applicable to very low embankments or roadways. For the construction of very wide embankments on very soft subsoils, fabric-reinforced toe roads for hauling the fill have been constructed in a manner similar to Figure 25c.

The mechanism of geotextile or geogrid reinforcement was discussed in the introduction to this section—it must be able to counteract the shear stresses imposed on the foundation by the embankment. To do this effectively, the reinforcement (a) must have a sufficiently high tensile modulus and ultimate tensile strength and (b) it must be able to develop sufficient frictional resistance with the subsoil and/or embankment materials. Furthermore, these properties must be reasonably constant throughout the design life of the project, or, if they decrease because of creep or biochemical changes, the rate of this decrease must be less than the rate of increase in shear strength of the foundation caused by consolidation. Additional requirements are that the reinforcement must have enough tensile strength and modulus, puncture and tearing resistance, and burst strength to be able to survive the construction operations (fill placement operations, construction equipment, etc.).

Geotextiles and geogrids have considerable potential to increase the stability of embankments that are widened and/or have their grades raised (*150*). According to Edil (*45*), geotextiles can assist in a number of ways, including reinforcement, with the common practice of preloading of peats in Wisconsin.

Good references on the use of geotextiles and geogrids for embankment reinforcement include *31, 58*, and *151–156*. References *155* and *156* also contain a rather extensive list of books, conference proceedings, and special reports that provide general information about geotextiles and geogrids, case histories, and design information. Mitchell and Villet (*157*), which concentrates on reinforced retaining walls and slopes, has some useful information about geotextiles and their durability.

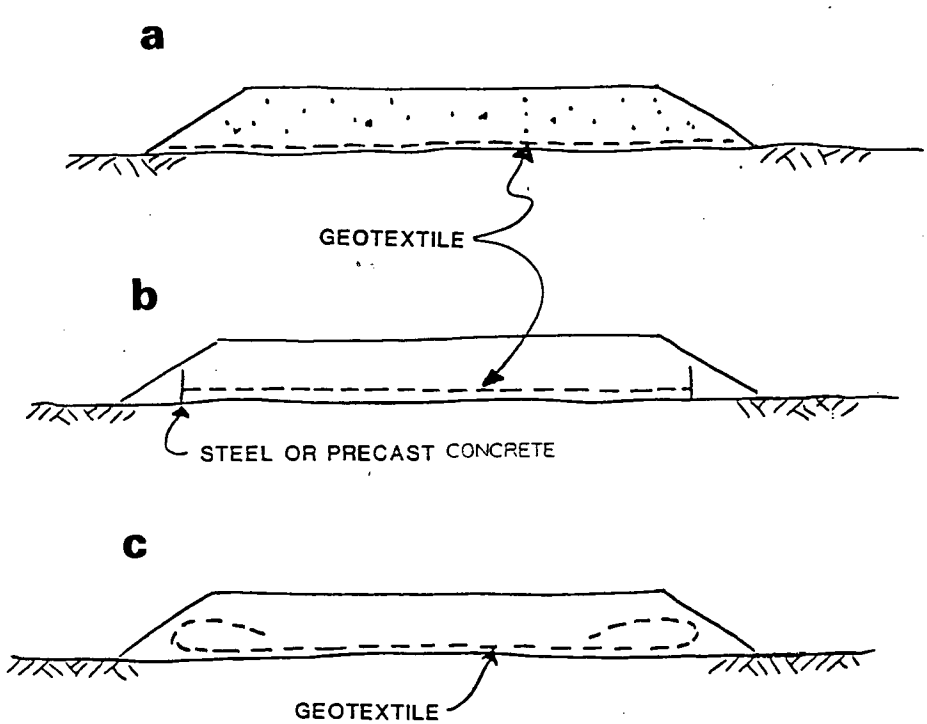


FIGURE 25 Concepts for using geotextiles to reinforce embankments on soft foundations (*58*).

Design procedures for reinforced embankments will be treated in Chapter 6.

Ground Anchors, Soil Nailing, Micro or Rootpiles, Etc.

Other types of reinforcement that might be suitable under certain circumstances for embankment foundations include ground anchors or tiebacks, soil nailing, micro or root piles, and related systems. Most of these systems seem more appropriate for reinforcement of structure foundations, slopes, and retaining walls than embankment foundations, but it is possible that these systems might prove feasible for the stabilization of, for example, sidehill embankments. Consequently, a few pertinent references will be given for completeness:

Ground anchors and tiebacks: *138, 158,* and *159–164.*
Soil nailing: *32, 138, 157,* and *165–167.*
Micro or root piles: *32, 136, 138,* and *167.*

The FHWA also has a couple of research projects under way on nailing and earth reinforcing. When these projects are completed, their reports should be a valuable source of practical information on these subjects.

Sheet Piles, Slurry Diaphragm Walls, Etc.

These methods are mentioned in this section because they have on rare occasion been used for embankment stabilization. See textbooks such as Terzaghi and Peck (*62*) and U. S. Navy (*63, 105*) for a description of these systems.

CHAPTER FOUR

SPECIAL CONSIDERATIONS

CULVERTS, BRIDGES, AND OTHER STRUCTURES

Culverts in embankment fills present special design problems, as do bridges.

Sometimes, when the subsoils are very soft and large settlements are expected, culverts are supported on piles to maintain their flow capacity. As the adjacent embankments settle, the culvert will tend to project through the pavement, resulting in a "bump" over the culvert. This is a particular problem at sites where improvement of the foundation was not attempted before the installation of the culvert—i.e., where settlements are accepted on the premise that they will be essentially uniform in nature. If no foundation soil improvements are attempted and if settlements are expected to be substantial, it is essential that the culvert not be supported on piles. Even so, nonuniform settlements may occur anyway, depending on the size of the culvert, height of the embankment and amount of cover over the culvert, amount of overexcavation, type of backfill, and the consolidation characteristics of the foundation soils.

Settlement effects on culverts can be minimized by locating the culverts near the edges of swamps—for example, where the thickness of soft soils may be less. Of course, culverts should either be oversized or designed with a camber so that the grade line after settlement still satisfies the hydraulic requirements.

One possible but relatively expensive solution to the culvert settlement problem is to simply use two culverts at the site, one stacked upon the other; as the embankment settles and one culvert gradually disappears below stream grade, the one above begins to take over. This procedure has been successful under a geotextile-reinforced embankment on a peat bog in Wisconsin (B.R. Christopher, personal communication, 1984).

Bridge abutments, which are very often pile-supported, present special problems similar to those discussed for culverts. These problems are discussed in detail in Synthesis 2 (*3*) and in its update to be published in 1989. Additional information on abutment movements can be found in References *8, 29*, and *30*. Techniques described in Chapter 3 that may be appropriate for stabilization of approach embankments include lightweight fills, embankment piles, excavation and replacement, all the consolidation techniques, lime and cement columns, stone columns, and dynamic compaction. In order to reduce the bump at the end of the bridge, designers should seriously consider the use of transition zones in which the amount of improvement is varied as the abutment is approached and/or the thickness of the compressible layer varies.

UPGRADING EXISTING EMBANKMENTS

There is a trend toward less construction on completely new sites and more emphasis on upgrading existing facilities. Upgrading projects can be quite complex with interchange and overpass embankments, widening of existing roadways, or perhaps a completely separate but adjacent roadway. On the other hand, upgrading may involve relatively simple things such as widening shoulders, flattening slopes, changing grades to improve sight distance or drainage, and adding extra traffic or turning lanes. Even these simple things can present difficult problems, especially if the soils are soft and deep.

If the present road has been in place for many years, the foundation soils under the existing embankment are probably well into secondary compression, and relatively little additional settlement is likely to occur. On the other hand, adjacent soils under the new construction are probably nearly normally consolidated, and when they are loaded, will undergo relatively large settlements. Thus, large differential settlements are likely between the old and new construction, which may result in a longitudinal crack or fault in the surface. Furthermore, there is always the possibility of a slide between the old and new construction, especially if the grade is raised substantially. The problem is how to attach, key in, or tie the new construction to the old without a slide or detrimental differential settlement.

Almost all soil improvement techniques described in Chapter 3 have potential for addressing this problem. Depending on the specific design requirements and site conditions, the following methods should be particularly attractive for this purpose: lightweight fill; embankment piles; excavation and replacement; consolidation with vertical drainage; lime or cement columns; vibrocompaction and vibroreplacement methods, especially stone columns; and reinforcement, particularly with geotextiles and geogrids.

Some methods have serious construction problems that probably would make them unfeasible. Blasting immediately comes to mind. Excavation and replacement, although often a very positive method, as discussed in Chapter 3, will be difficult to construct adjacent to an existing roadway, especially if the excavation is rather deep. One can imagine problems with stability and access for excavation equipment and trucks bringing backfill, among others. In this case, it is necessary to keep the backfilling operations closely following the excavation (*34*). Surcharging and stage construction, probably impractical without vertical drainage, would be difficult if the required stress level

is large. Problems include safety considerations while the surcharge is in place and how to bring in the fill and then remove it without disrupting traffic. On the other hand, this might be a good place to use the vacuum surcharge method, provided a suitable way can be found to seal it at the existing embankment. At some sites, dynamic compaction may also have some potential benefit for widening roadways.

SPECIAL DEPOSITS

Introduction

A number of deposits, both natural and man-made, can pose difficult problems for the designer of embankment foundations. They may be difficult if not impossible to sample and test by ordinary means. Because of this, their engineering behavior is not well documented nor well understood. Consequently, usual practice, design rules, and experience may not lead to satisfactory performance. A number of these special deposits are good candidates for some type of soil improvement and stabilization, so that highway embankments can be constructed on them safely and with acceptable settlements.

Deposits discussed in this section include landfills and dumps, wastes from all types of mining operations, collapsible soils, and deposits subject to liquefaction.

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 (*168*) and Chang and Hannon (*169*), but the results are often less than satisfactory unless some special foundation treatment is carried out. The type of problems encountered at such sites depends on the nature of the landfill materials and their age, both of which may be highly variable. In some well-operated landfills, the area is relatively clean, contains lots of natural soils, and was subjected to some compaction when placed. These fills will normally consolidate rapidly, and thus only a surcharge load applied for a reasonably short time is required to adequately densify them. In other instances, where municipal garbage and building wastes are loosely dumped, embankments on such landfills will experience very large total and differential settlements over a long time period, because of consolidation and gradual decomposition of the wastes. In these recent or young landfills, long-term creep settlements can take place even after ground treatment, and this may mean a poor riding surface and high maintenance. Older landfills (15 to 30 years old) may already have decomposed a great deal and thus are good candidates for foundation treatment.

According to Lukas (*122*), the rate of settlement of a landfill is affected by (a) state of compaction of the refuse, (b) water content and water table location, (c) composition of the refuse, and (d) environmental factors such as pH, temperature, and presence of toxins, all of which affect the rate of decomposition. He also gives some information on testing of landfills and summarizes research attempts to develop predictive models for settlement rates. Procedures for estimating rates of settlement under self weight and external loads are given in U.S. Navy (*105*). However, because of the variability of the material plus unknown environmental factors and time effects, postconstruction settlements of sanitary landfills under embankments are difficult to predict. Further, they are likely to be large and nonuniform. Therefore full-scale field test sections (Chapter 5) are probably necessary to design with much confidence.

Landfill sites pose other problems during site exploration and later during construction. As noted by Charles and Burland (*170*), decomposition of municipal wastes generates methane and carbon dioxide, and the former presents an explosion hazard if it is able to accumulate in any quantity. Also, the introduction of fresh air into a site, even from a borehole, could cause a fire by spontaneous combustion or even from smoldering material buried in the landfill. Thus, boreholes are best stabilized during drilling with water and drilling mud. Also, it is good practice to grout exploratory boreholes at the completion of the site investigation program. In some instances, difficulties have been experienced with noxious gases, and it has been necessary to use breathing apparatus, deodorants, and exercise special rodent- and pest-control measures after the area was opened during construction.

Suspected toxic or hazardous waste dumps pose especially serious problems if they must be crossed by the highway. Such sites become the owner's responsibility and liability should they be inadvertently acquired as part of a right-of-way purchase. Special precautions must be taken during the site investigation and testing program to protect field crews during sampling and laboratory personnel while testing the samples. It is prudent in these cases to call for help from firms specialized in this type of work. Even if a hazardous site is only temporarily open, problems can be caused by the infiltration of rainwater runoff into the leachate collection system. Hazardous sites should be avoided whenever possible.

A number of useful papers on properties, construction, environmental factors, and stabilization of landfills are References *171* and *172*.

Possible feasible methods for the treatment of landfills for embankment foundations include (a) proofrolling with very heavy rollers, (b) surcharge, (c) excavation and replacement as a compacted fill, (d) embankment piles, (e) grouting, (f) vibrocompaction, and (g) dynamic compaction. Both Moore and McGrath (*168*) and Chang and Hannon (*169*) used 50 ton (45 Mg) rollers, but surcharge was also effective in reducing settlements. Provided a suitable disposal means can be found, excavation and replacement is feasible and may be economic, depending on the nature of the refuse, depth, location of the water table, etc. (*170*). Although feasible, as far as is known, embankment piles have not been used to stabilize landfills. One example of a lime-fly ash grouting of a landfill was reported by R.H. Prysock (personal communication, 1987), and there are probably others. Of the other methods, probably dynamic compaction is the most practical, and the suggestions of Lukas (*122*) for effective utilization of this technique at landfill sites are recommended.

On the other hand, California has had some rather unpleasant experiences with construction on closed landfills. Municipalities were reluctant to permit an adequate exploration of such sites to determine the geotechnical characteristics of the materials. Caltrans attempted simply to build embankments on these unexplored areas and use surcharges to achieve early settlement. Unfortunately, serious consequences resulted. Transverse tension cracks of 2 in. (50 mm) or more in width occurred across

the surcharge surface and for the full depth of the surcharge, and subsequent internal erosion resulted in large channels near the base of the fill. These channels were difficult to backfill, and there is concern that they might again occur after the surcharge is removed and the project completed (R.H. Prysock, personal communication, 1986).

Many waste dumps are not controlled landfills as described above, but are sites such as swamps, tidal flats, river and stream banks, lakes, etc. where random garbage, trash, used appliances, wrecked cars, used tires, etc. have been 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. The methods discussed above may be suitable in these cases also, provided they are also suitable for the natural materials at the site.

Inorganic Industrial Wastes and Dredged Materials

Other wastes that are sometime of concern to highway designers include industrial byproducts and wastes that are essentially inorganic; at least the organic content is such that long-term decomposition does not significantly affect their geotechnical performance. The steel industry produces various types of slags and ashes, chemical industries sometimes generate inorganic sludges, and fossil-fuel electrical-generating plants produce various residues such as scrubber sludges, bottom ash, and fly ash. 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. If these deposits are located during the initial planning and site investigation stages, the normal geotechnical design process can propose the usual foundation design alternatives, including soil improvement if necessary.

Loose deposits of predominately granular materials such as slags and bottom ashes can be treated by methods appropriate for such materials (dynamic compaction, blasting, vibroreplacement). Fly ashes are usually deposited in large heaps and are rarely foundation problems. However, if fly ashes are encountered, they may be difficult to stabilize in situ because they are so fine grained. Because they are mildly pozzolanic, mixing or flooding with water should provide considerable stabilization. According to Lukas (*122*), dynamic compaction has only a fair suitability for stabilizing these materials.

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; also, often the deposit contains alternating layers of finer and coarser materials of a highly variable water content because of periodic deposition and surface drying. Preload and surcharge methods with vertical drainage would appear to be quite effective in consolidating these materials, *if* it is possible to actually construct the fill. In some dredged spoil areas, the sediments are so soft and deep that reinforcement by very high strength geotextiles may be the only feasible means of constructing an embankment. If the dry crust can support excavation and compaction equipment, then excavation and replacement with chemical and fly ash stabilization may be effective, and methods such as surcharging are readily possible. In some cases, berms, lightweight fill, embankment piles, stone columns, and geotextile reinforcement could be feasible and should be considered when making preliminary comparative cost estimates.

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

Mining operations, whether surface or underground, 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 and construct on.

Strip-mining operations often leave large areas of loosely dumped materials called mine spoil, which range in size from boulders down to clay. If coal is the mineral mined, often there is some organic material left in the waste. Usually the spoil and overburden materials are cast back into the area just excavated. Depending on the nature of the overburden, how these materials are excavated and redeposited, and the mine reclamation operations, if any, the backfilled areas may be very loose and highly variable. Usually excessive settlements are the main problem with embankments constructed on strip-mined areas, but in some local cases, stability could also be low. Stabilization by excavation and replacement is always a possibility, as is preloading and surcharging; there is usually plenty of low-cost surcharge fill available at such sites. Of the other techniques, dynamic compaction is probably the best choice, unless the material degrades or slakes during tamping (*122*). Vibroreplacement and vibrocompaction methods might also be feasible in certain spoil areas.

Wastes from underground mines are also loosely dumped, but rarely in a form or locations for embankment foundations. Should this occur, however, because such waste heaps are predominately granular in nature, the usual means for stabilizing loose granular materials (dynamic compaction, vibrocompaction, etc.) should be suitable.

Some mineral-processing operations, such as crushing, grinding, floatation, etc., produce tailings, which are usually transported to the waste depository ("tailings dam") as a high-water-content slurry. Tailings ordinarily range in sizes from coarse sand down to very fine silts and clays. Depending on how the material is deposited in the dam, it can be clean, uniform sand in some areas, grading to silts and very fine clayey sediments ("slimes") in other areas. Often the materials are quite stratified, with thin (0.5 to 2 in.; 12 to 50 mm) lenses and laminations of alternating sandy and finer materials. Because tailings are deposited by hydraulic filling, they are in a very loose state of compaction. Stabilization of the clean sands is not particularly difficult; the methods previously described could be feasible, depending on local conditions. Stratified materials may be effectively treated by stone columns or perhaps vertical drains. Stabilization of the slimes could be more difficult; consolidation under surcharge with vertical drainage is always possible, depending on their strength. Some type of reinforcement may be the only feasible means for actually supporting the embankment on very weak slimes. If vertical drains are used, one must be sure the filter is not affected by the minerals or acids in the slimes.

Another factor that should not be overlooked is the safety of

field personnel. Tailings from uranium mining in New Mexico, Utah, and Wyoming often contain some radioactivity, and personnel working with such materials should follow all necessary safety precautions.

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 (*140, 173*). 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 a soluble salt such as halite, gypsum, or carbonates. Loess is of course wind-deposited; other collapsible soils are found in alluvial flood plains and 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. Some dredged material deposits can be collapsible, too.

Dudley (*174*) gives an extensive review of the identification and treatment of collapsible soils; Bara (*140*) also summarizes some of the methods for predicting the decrease in void ratio upon wetting, as do Clemence and Finbarr (*173*) and Houston et al. (*175*). These authors and U.S. Navy (*105*) also discuss foundation treatment design.

Sudden unexpected settlements can be very detrimental to bridge and highway embankment foundations; thus it is important in advance of construction to identify sites likely to contain collapsible soils so that appropriate treatment measures can be instituted. Russman (*176*) describes the damage caused to two Interstate highway bridges by settlement of collapsible soils that were not identified during design. Settlements up to 1.3 ft (0.4 cm) occurred, probably because of moisture infiltration during the 16 years after construction. Both bridges had to be replaced with structures designed to accommodate possible future settlements.

According to Bara (*140*), 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. Dynamic compaction would also probably be feasible. For thicker deposits, ponding or flooding are ordinarily very effective, as is dynamic compaction, but explosives, displacement piles, and any of the above-mentioned vibroreplacement/vibrocompaction methods could possibly be used. Similar information can be found in Clemence and Finbarr (*173*) and U.S. Navy (*105*). Design information for the deeper stabilization methods is given in Chapter 6.

An interesting case history of the use of dynamic compaction to treat hydrocompactible soils on Interstate 90 in Montana has been described in detail by Yarger (*2*). Not only was the treatment very effective, but it was significantly cheaper than the excavation and replacement alternate.

Loose, Saturated Sands in Earthquake Country; Flow Slides, Etc.

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 (*17*). As this loss in shear strength is sudden, the effect on structures such as highway embankments supported on 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 in Chapter 3 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. Also possible are all forms of grouting, particularly compaction grouting, because it densifies the soil, other chemical stabilization, piles (both displacement and supporting the embankment), and some types of in situ reinforcement such as micro piles and soil anchors.

While upgrading existing facilities or even during a general review of the earthquake hazard potential for a highway, it may be discovered that existing embankments are on liquefiable soils. In this case, the recommendations of Ledbetter (*177*) are recommended. (Incidentally, he also gives an excellent review of site improvement methods and an extensive list of references on each method.) At the present time, there has been essentially no direct experience with remedial measures under existing structures at potentially liquefiable sites, and no one method appears to be generally applicable for all site conditions and structures. In evaluating possible remedial methods, the most important factors are the verifiability of improvement and the assurance that the method itself will not create any unstable conditions during earthquakes or under static loading.

CHAPTER FIVE

SUBSURFACE INVESTIGATION AND TESTING

SUBSURFACE INVESTIGATION

A general discussion of subsurface investigation is given in Synthesis 8 (*4*) and Synthesis 33 (*5*). The new AASHTO *Manual on Subsurface Investigations* (*178*) should be consulted for specific information and procedures on reconnaissance surveys and subsurface exploration including in situ testing and geophysical methods. The Manual also discusses the effects of groundwater, earthquakes, and other geologic factors, as well as environmental impact analyses. Although the purpose and types of laboratory tests are mentioned, detailed testing procedures are not given. Another good general reference is Clayton et al. (*179*), and some of the information on field investigations in TRB Special Report 176 (*15*) on landslides is also appropriate for embankment foundations.

For any project, a detailed study of the site geology should be routinely made and aerial photos of the site (including historical photos) should always be obtained and analyzed. The results of these studies will help with planning of the site investigation program.

For subsurface investigations of problem foundation soils, the principal recommendations are: (a) closely spaced borings, (b) for cohesive soils, frequent high-quality undisturbed sampling, and (c) for all types of problem soils, the use of appropriate in situ tests to complement the exploratory borings and undisturbed sampling. In many problem soils, obtaining good undisturbed samples is difficult and expensive, so in situ techniques such as penetrometers and geophysical methods are used to provide design property information. Undisturbed sampling and in situ testing are discussed later in this chapter.

The spacing of exploratory borings depends on local soil conditions as well as the type of foundation and soil improvement anticipated. For example, in a situation where a design on a very soft cohesive foundation is being investigated in which substantial but uniform postconstruction settlements would be accepted, closely spaced borings [often as close as 50 ft (15 m) on centers] are necessary in order to adequately define the extent and variability of the soft materials. This close spacing may also be useful for determining excavation and backfill quantities if that technique is used.

On the other hand, naturally deposited granular materials and waste fills tend to be extremely erratic and variable, and the primary objective of the investigation program is to get an idea of the variability, especially the extremes, of the density and other soil properties across the site. In these cases, spacings on the order of 100 to 200 ft (30 to 60 m) or greater, depending on the extent of the deposit, may be appropriate.

The new AASHTO Manual (*178*) gives some recommendations for spacing of borings for embankment foundations and other situations.

Depth of exploration and sampling is another question. How deep is deep enough? The answer is "deep enough so that what is below will not affect your embankment." Borings should be deep enough to provide design information for all possible treatment alternatives at the site. For example, if a pile foundation is likely, borings must provide sufficient information to design the piles. If there is soft muck at the site, the borings should go at least to firm bottom. Rules of thumb for the depths of borings are given in textbooks and in the new AASHTO Manual (*178*). One rule for high embankments is that the depth of the borings should be two to four times the proposed height of the embankment, unless, of course, bedrock or dense materials are encountered first. For very deep deposits of soft materials and very wide embankments, even four times the height may not be deep enough. To be safe and economical, each site and project must be evaluated individually, using geologic information as a guide.

Preliminary Subsurface Investigations

The necessity of conducting public hearings for new routes that probably will become binding makes it essential to obtain as much detailed subsurface information as possible as part of the preliminary soils investigation program. Consequently, undisturbed sampling at soft-soil sites is now often carried out during the preliminary site investigations or even during the planning phase before public hearings.

Depending on the geology and soil conditions, in situ test methods may be an economical way to provide good preliminary subsurface information. In situ testing will be discussed later in this chapter.

Always helpful in this regard are correlations between classification data and engineering properties, especially if they have been developed locally for areas with extensive deposits of problem soils. Such correlations are useful not only for preliminary foundation design but also for independently verifying laboratory and in situ test results. U.S. Navy (*66*) and the AASHTO Manual (*178*) are good sources for many generally accepted correlations, but there is no substitute for local experience.

Importance of Undisturbed Sampling

It is generally agreed that the best undisturbed samples of soft cohesive materials and peats are obtained by careful hand trimming from test pits and excavations. This is often impract-

ical at many sites because of a high water table and stability problems with the test pit walls. The Sherbrooke sampler (*180*) is probably the next best device for obtaining relatively undisturbed samples from a borehole. After that in quality are stationary piston samplers, and these are described in the new AASHTO Manual (*178*). The minimum diameter should be 3 in. (75 mm), and the borehole should be stabilized with drilling mud. Although commonly used, open-drive or Shelby-tube samplers produce partially disturbed samples, no matter how carefully they are taken. In all cases, the quality of the samples depends greatly on the procedures used and the care taken by the technician or driller. Well-trained, careful field personnel are absolutely essential to get high-quality undisturbed samples, no matter what procedure is used.

Although high-quality undisturbed samples of soft deposits are expensive and not easy to obtain, they are especially important in the case of embankment foundations because of their great influence on subsequent analyses comparing different soil improvement methods. For example, when poor-quality, partially disturbed samples are tested in the laboratory, they yield values for the preconsolidation pressure and coefficient of consolidation that are far too low. These effects may lead a designer to conclude that foundation treatment using consolidation and vertical drainage is necessary even though this is not the case in reality. If the sum of the vertical effective overburden stress and the stress induced by the embankment are less than the preconsolidation stress, foundation treatment to reduce settlement is almost certainly not required. But this may not be apparent if the samples are partially disturbed and yield preconsolidation stresses less than their true values. Similarly, if the coefficient of consolidation is too low because of poor-quality samples, the designer may conclude that foundation treatment is necessary, when in fact subsurface soils may consolidate rapidly enough without vertical drainage.

Sample disturbance also results in lower shear strengths than actual, which may lead to unnecessarily expensive measures to increase embankment stability, or even to an erroneous conclusion that the site is impossible to build on. One can imagine the difficulties and expense if the route is fixed and right-of-way has already been purchased or optioned.

A very effective way to evaluate the quality of tube samples and to decide what sections of each tube to test is to x-ray them as they are brought in to the laboratory. The Soil Mechanics Bureau of the New York Department of Transportation has been doing this routinely for about 25 years as has the U.S. Army Corps of Engineers soils laboratory at the Waterways Experiment Station.

In some of the problem soils and fills mentioned in Chapter 4, particularly the loose, granular soils, waste dumps and landfills, etc., the taking of good undisturbed samples is difficult and expensive. Thus, designers are forced to rely almost exclusively on in situ techniques such as the Standard Penetration Test (SPT) and other types of penetrometers, as well as geophysical methods, which are discussed next.

In Situ Testing

Since Synthesis 29 (*1*) was published in 1975, the use of in situ tests (see Table 8) in geotechnical engineering practice has grown dramatically. A number of in situ devices were just being developed about that time (piezocone penetrometer, borehole shear, screwplate, and self-boring pressuremeter), and some have actually been developed since then (acoustic cone, dilatometer, Iowa stepped blade). Many of these developments are briefly described in the new AASHTO Manual (*178*); additional details can be found in References *64, 66, 70,* and *181–189.*

For determining the undrained shear strength of soft clays, the field vane shear test has proved to be a versatile tool, provided it is conducted in a stabilized borehole at the recommended rate and is properly correlated for local conditions. It is not appropriate for fibrous peats nor for determining the strength of the dry crust found in many sedimentary deposits. The strength properties of these materials are very difficult to determine by any means (*190*). If good undisturbed samples can be obtained, laboratory tests are appropriate.

It should be emphasized that for soft-soil sites, the SPT is not sufficiently sensitive to provide useful direct information for design, except on a preliminary basis. However, it does provide a disturbed sample for visual classification, water content, and Atterberg limits at relatively low cost. This information can then be correlated with the shear and consolidation test data.

One of the most useful in situ tools for site improvement studies is the piezocone penetrometer. Basically this device is a Dutch cone modified to include a piezometer for measuring the pore water pressure induced during penetration. Piezocone results are especially useful at soft clay sites at which the use of preloading with vertical drains is contemplated. As shown in Chapter 6, among other things it is important to be able to determine the macrofabric and drainage boundaries at the site as well as the in situ consolidation and permeability characteristics. As indicated by Jamiolkowski et al. (*70*) and Holtz et al. (*88*), the piezocone is the most effective tool for investigating these items.

Other useful in situ tests for soil improvement studies include the pressuremeter and the Marchetti (*185*) dilatometer. The pressuremeter has been used extensively to determine the degree of improvement after dynamic compaction (*122*). The dilatometer is especially useful for determining the in situ stress and compressibility characteristics of loose sands and moderately soft clays. It is not so reliable for very soft clays, and cannot be used if the soils contain gravel. In this case, relative density and soil property information are derived from empirical correlations. Mitchell (*191*) and Welsh (*192*) discuss in situ testing as applied specifically to foundation treatment methods.

SOIL TESTING

Laboratory Testing

The conventional laboratory testing program for soils in soft-foundation areas includes the usual classification and identification tests (water content, density, Atterberg limits, specific gravity for organic deposits, etc.) as well as the tests for their engineering properties (permeability, compressibility, and shear strength). Permeability, if desired, is probably better obtained directly in the field (see *70, 88, 178*), although New York State has had success with a "block permeability" laboratory test (*193*). Compressibility and consolidation properties are determined using consolidation (oedometer) tests, and triaxial compression tests are used to obtain the shear strength.

For the consolidation properties, conventional incremental load oedometer tests are acceptable, although consideration should be given to the use of constant-rate-of-strain (CRS) tests. Although some interpretation is required to obtain the coefficient of consolidation and no information is obtained about secondary compression, CRS tests have the strong advantage that the results are obtained much more rapidly than with the conventional tests using 24 hours per load increment. Also, because a continuous compression curve is obtained, interpretation of the preconsolidation pressure is simplified. The ASTM standard for the CRS test is D 4186. It is, of course, possible (and permitted by ASTM Standard Test Method D 2435) to conduct a consolidation test with shorter times of loading and smaller than the usual double load increments. For example, New York State routinely uses 2.5 hours for each load increment (*193*).

Detailed procedures for the various laboratory consolidation and strength tests can be found in U.S. Army Corps of Engineers (*194*). Other references are listed in the new AASHTO Manual (*178*). For a discussion of how to interpret and use the results, see References *17, 65, 66,* and *178*.

For embankment-type loadings on soft, normally or lightly overconsolidated clays, the traditional approach has been to use the unconsolidated-undrained (UU) test. The strength obtained is in terms of total stresses, and it is appropriate for the end-of-construction condition, which is the worst case for embankments on soft foundations (*17*). Often, in testing soft clays for foundation design, unconfined compression tests are substituted for UU triaxial tests, because they are easier and faster to run. This procedure is satisfactory if the samples are homogeneous and completely saturated (*17*). If an agency goes to the expense of obtaining high-quality undisturbed (e.g., piston or block) samples, then it is wasteful not to conduct equally high-quality triaxial tests on those samples.

Using UU strengths for embankments on soft foundations often results in overly conservative designs, primarily because of sample disturbance, but also because of an inappropriate stress path. A better procedure is to reconsolidate the specimens back to their in situ stress conditions, and then shear undrained. Technically, this is a K_o-consolidated-undrained (CK_oU) test. It is a more complicated test to run than an ordinary CU test with equal all-around consolidation pressure (sometimes called a CI—for isotropic—U test). Furthermore, the samples have to be very high quality, otherwise reconsolidation results in an excessive overprediction of undrained strength. In fact, the amount of strain that occurs during reconsolidation is a good measure of how good the sample is and the quality of the testing procedure. Research is needed on how much strain to allow, but probably something like 3 or 4 percent is a reasonable upper limit. The Norwegian Geotechnical Institute pioneered with this type of testing for soft clays (*195*), and it is discussed by Jamiolkowski et al. (*70*) with some reservations. One of the principal difficulties is that a knowledge of K_o in situ is required, and as Jamiolkowski et al. (*70*) note, obtaining K_o in situ is not a simple task.

Especially for preloading projects in which stage construction is monitored by field piezometric measurements (Chapter 7), theoretically, stability analyses should be in terms of effective stresses, and therefore, pore pressures must be carefully measured during the triaxial tests. However, there are problems with this approach, and the recommendations of Ladd (*77*) in this regard are appropriate.

The shear strength of granular materials can also be determined in the laboratory by direct shear or consolidated-drained (CD) triaxial compression tests. However, because, as mentioned earlier, undisturbed sampling of granular materials is so difficult, their strength properties are best determined through correlations with in situ test results.

For a discussion of triaxial tests and when to use them, see Chapter 6 in the TRB special report on landslides (*15*) and References *17, 64, 66, 178,* and *196*.

In Situ Tests

It should again be emphasized that most soil properties required for the design of soil improvement techniques may be effectively determined by means of in situ tests, especially for deposits that are difficult to properly sample. Also, in many problem soils and sites, geophysical methods can be used to great advantage (see *178*).

Evaluation of Test Data

After the subsurface investigations and the laboratory and in situ testing are completed, it is essential to prepare generalized soil profiles, both longitudinally and transversely in areas of soft- or problem foundation soils. These generalized profiles should show the site stratigraphy and indicate the soil type and description, natural water content, Atterberg limits, compressibility, and strength data, all with depth. The locations of the various soil tests should also be indicated. Groundwater table information is also essential, as is any in situ test information obtained. Generalized profiles should, of course, be developed, based on geological reconnaissance as well as on the geotechnical investigations.

FIELD TEST SECTIONS

For major projects or when an agency is planning major construction in a locality where certain soil deposits are known to have caused problems in the past, full-scale field test sections are a very cost-effective way to obtain information for design, performance, and construction of likely feasible soil improvement techniques. This information can help achieve one or more of the following objectives: (a) prove or disprove design hypotheses, (b) refine the determination of soil properties through back analysis, (c) determine minimum cost designs for the entire project, (d) reduce the quantities of materials required, (e) determine if the most economical procedure is actually capable of providing satisfactory results, (f) establish the feasibility of construction techniques to minimize costs, and (g) serve as a training ground for field quality control and inspection personnel. Although they are expensive, depending on the experience of the agency with soil improvement techniques and the soils in the locality under question, field test sections can often pay for themselves many times over in construction cost savings. Occasionally, they provide information for design when nothing else will.

Field test sections should be considered either before final designs are made or as the first part of an overall project plan. It may also be feasible to construct a portion of the roadway in advance of the remaining sections, and treat the initial portion as a test section. Field test sections employing foundation consolidation have greatly increased value if a portion of the test area can be loaded to failure. This permits a realistic evaluation of the stability, and it may allow the size of berms to be reduced or perhaps eliminated entirely. Experiences gained during trial sections using new or unusual soil improvement techniques can be extremely valuable in writing specifications, developing inspection and quality-control procedures for the job, and establishing possible dimensional and other design improvements.

Finally, it should be noted that field test sections require a complete site investigation and soils testing program, as discussed above. The site and test embankment must be well instrumented and monitored to obtain full benefits from the test. Instrumentation and site monitoring will be discussed in Chapter 7.

CHAPTER SIX

DESIGN OF FOUNDATION TREATMENT METHODS

INTRODUCTION

A large number of methods and techniques for improving soft- or problem foundation soils were described in Chapter 3, but the procedures for actually designing the treatment were left to this chapter. As mentioned in Chapter 1, the detailed design of these methods is beyond the scope of this synthesis, but some design considerations, pertinent details, and selected references on design are presented in this chapter.

STABILITY ANALYSIS

As mentioned in Chapter 2, the first design requirement for any embankment is stability; the embankment must be stable during and at the end of construction and throughout its design life. Classical bearing capacity, circular arc, and sliding wedge analyses are performed on various typical sections of the proposed embankment, using the information obtained in the subsurface investigation program and the shear strengths obtained from the soils testing program discussed in Chapter 5. Depending on the geology and soil profile, certain assumed failure surface geometries (circular arc, sliding wedge, composite, etc.) may be more appropriate than others, but usually more than one type is assumed to obtain the minimum factor of safety.

Useful references on the types, assumptions, and mechanics of stability analyses can be found in the textbooks by Taylor (*197*), Terzaghi and Peck (*62*), and Perloff and Baron (*25*). Cheney and Chassie (*64*) have a good introduction to the subject; U.S. Navy (*66*) is also recommended, as is the chapter on stability analyses in the TRB special report on landslides (*15*). Terzaghi and Peck (*62*) describe the different modes of embankment failures and how they should be approached in design. The connection between stability analyses and appropriate measures of shear strength is also discussed in these references as well as in Holtz and Kovacs (*17*) and Duncan et al. (*196*).

Computer Programs

Laborious hand calculations can be avoided by the use of readily available design charts, programmable calculators, and computer programs, many of which are now available for desktop personal computers. Some highway departments, notably New York's, have written their own stability programs, but most rely on "canned" programs, i.e., those developed by others. One of the more popular stability programs among highway departments is STABL, which was developed at Purdue University for the Indiana Department of Highways (*198, 199*). Other popular programs have been developed at the Universities of California and Texas, VPI, and by the U.S. Army Corps of Engineers. As with all programs, the designer is responsible for assuring that the results are valid; spot checks, simple bearing capacity analyses and hand calculations, and design charts (*66, 197, 200*) are useful for this purpose.

Factor of Safety

What are the appropriate safety factors for stability analyses? The choice depends on tradition, the critical nature of the project (Table 3), the importance of the facility, the cost of failure, and the reliability of the subsurface information and soil properties. Traditional factors of safety for embankments on soft foundations for the end-of-construction condition are 1.3 to 1.5, and unless the site investigation program is unusually detailed and the sampling and laboratory testing program is almost research quality, these values still seem prudent today. U.S. Navy (*66*) recommends no less than 1.5 be used for permanent or sustained loading situations. If soil improvement by foundation consolidation is being done, then the minimum factor of safety may be less than 1.3, because consolidation and strength gain is expected to occur rapidly during construction. However, such practice should only be followed if (a) the site investigation, sampling, and testing program is excellent; (b) detailed calculations of expected strength gain are carried out; (c) a thorough construction monitoring and instrumentation program (Chapter 7) exists; (d) the designer is involved with the construction control; and (e) stability checks are made by the designer during construction.

It is probably time that geotechnical engineers begin to phase into practice a probabilistic approach to stability analyses, in which the probability of failure (rather than a factor of safety) is used to assess the reliability of a proposed design. Such an approach would seem to be appropriate for critical (Table 3) and high-risk foundation treatment projects. New York (*201*) has done some work on this, and the approach was utilized in FHWA-sponsored research on shallow foundations (*202, 203*). A good beginning reference on reliability-based design is Harr (*23*).

Stability of Berms and Flatter Slopes

The first method discussed in Chapter 3 to increase the stability of embankments on soft ground was berms and flatter slopes. A typical cross section of an embankment with berms is shown in Figure 3.

If stability analyses of a soft foundation indicate that the embankment with conventional highway side slopes of 2h:1v and 3h:1v does not meet the minimum allowable factor of safety, flatter slopes and/or berms are usually considered so that adequate resistance can be provided. Different slope angles and/or thicknesses and widths of berms are assumed, and the factors of safety of each assumed section are determined from the detailed stability analyses described above. Analyses should also be performed on the outer portions of the berm, even though trial failure surfaces of the maximum embankment section are stable. Berms may increase the total settlement, especially of the outer edges of the embankment, so settlement analyses should be done also.

For design of berms on peat foundations, the recommendations of Raymond (*204*) regarding design and construction are noteworthy.

Stability Analysis of Lightweight Fill Materials

As mentioned in Chapter 3, stability can be increased by using lightweight embankment fill materials. In addition, the required berm sizes can be greatly reduced or berms may be eliminated entirely, substantially reducing construction costs. Unit weights given in Table 6 are a guide to the kinds of reductions in applied stress that can be achieved from the use of lightweight fill. Compared with typical compacted earthfill, reductions in stress of 15 to 50 percent are possible (with expanded polystyrene, the reduction is 97 percent).

Design is, of course, relatively straightforward. A reduced unit weight is appropriately used in conventional stability and settlement analyses, and alternative materials are selected depending on cost including transportation and availability. Some of the construction, performance, and environmental problems discussed in Chapter 3 should also be kept in mind.

Stability Analyses of Reinforced Embankments

As mentioned in Chapter 3, one of the more innovative approaches to increasing the stability of embankments on soft foundations is to use some type of reinforcement. Geotextiles and geogrids are by far the most popular reinforcing materials, although as discussed in Chapter 3, bamboo fascines, corduroy, Columbus mats, landing mats, old automobile tires, and the Wager (*146*) short-sheet-pile method could also be used. As embankment reinforcement, all these materials function in a similar manner. Thus, methods of stability analyses considering all these materials are in principle the same—the only difficulty is how to obtain design values for the tensile strength of these materials.

Johnson (*205*) presented an excellent discussion of all aspects of the analysis and design of embankments, including those on soft foundations. The ways in which reinforced embankments on soft foundations can possibly fail has been described by Haliburton et al. (*206*) and Fowler (*149*), and these are shown in Figure 26. The possible failure modes suggest the types of stability analyses required. With very few exceptions, most of the design procedures for reinforced embankments are based on limiting-equilibrium type analyses, which are similar to conventional bearing capacity or slope stability analyses. Basically, they assume that the embankment is a very long strip or slope, and calculations for stability are made by assuming a series of potential sliding surfaces. The reinforcement acts as a horizontal force to increase the resisting moment. As with conventional slope stability and bearing capacity analyses, there are difficulties in analyzing composite soil-geotextile systems using limiting equilibrium. For example, the techniques assume a rigid, perfectly plastic stress-strain behavior, and they do not take into account the effect of system deformation on the embankment-reinforcement interaction. Redistribution of stresses in the embankment because of the reinforcement is also neglected, although research by Humphrey (*207*) indicates that classical stress distributions and limiting-equilibrium analyses are usually on the conservative side.

Humphrey and Holtz (*154*) reviewed published and unpublished case records of reinforced embankments and presented some summary charts that are useful for preliminary design. For a more formal design, Christopher and Holtz (*58*) list the steps in the calculation of the stability of a geotextile- or geogrid-reinforced embankment:

1. Check overall bearing capacity.
2. Check edge bearing capacity or slope stability.
3. Conduct a sliding wedge analysis for embankment splitting.
4. Perform an analysis to limit embankment deformations.
5. Determine geotextile/geogrid strength requirements in the longitudinal direction.

A detailed discussion of each of these steps is given in References *58, 152, 208*, and *209*.

Overall Bearing Capacity

The overall bearing capacity of the embankment must be satisfactory, with or without reinforcement. If it is not satisfactory, then there is no point in trying to reinforce the embankment. This can be seen by looking again at Figure 24. Imagine a very deep potential failure surface going from one edge of the embankment to the other. In this case, the reinforcement would have no influence on the stability, because it would never intersect the potential failure surface. Thus, reinforcement only protects against shallow failure surfaces or sliding.

Calculation procedures for overall bearing capacity are relatively straightforward. For deep soft deposits, the embankment is assumed to be a long strip footing on a clay foundation (*25, 62, 63*). If the thickness of the soft material is significantly less than the width of the embankment, then a "lateral squeeze" condition may exist; in this case, References *208, 210*, and *211* should be consulted.

If the overall bearing capacity is inadequate, stability will have to be improved by use of one or more of the techniques described above: adding berms, flattening the slopes and widening the base of the embankment, and using lightweight fill. In calculating the bearing capacity of the revised embankment, one must use the average height of the fill, rather than the full height, in determining the stress applied to the foundation—otherwise berms and flatter slopes will not work.

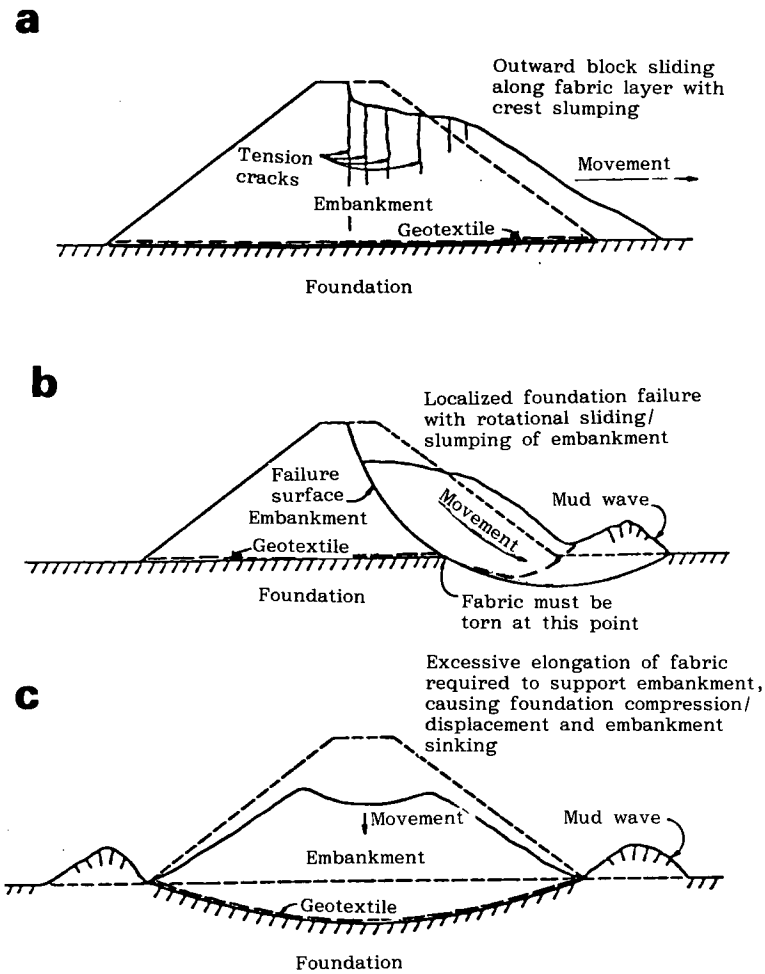


FIGURE 26 Possible modes of geotextile-reinforced embankment failure (after *149* and *206*).

Edge Bearing Capacity

Edge bearing capacity is calculated using a modified circular arc stability analysis. The tensile strength of the reinforcement must be sufficient to resist tearing or rupture at the intersection with the potential slip surface. The factor of safety of the critical surface is calculated as usual; then the additional resisting moment required to bring the factor of safety up to the minimum allowable is determined. Refer again to Figure 24, which is conceptually similar to how geotextile reinforcement functions. In the equation in Figure 24, the contribution of the reinforcement to stability is in the terms containing P, the tensile resistance of the reinforcement, and these terms are found in the numerator. Thus these terms increase the resisting moment and the factor of safety. Note that if the critical sliding surface is deep and its center is near the level of the reinforcement, the reinforcement moment arm, a, is small, and therefore, in this case, the contribution of the reinforcement to the resisting moment is small.

One or more layers of reinforcement with sufficient strength at tolerable strains are placed at the base of the embankment to provide the additional resisting moment. A number of procedures have been proposed for determining this required additional reinforcement, and the differences between them are primarily in the assumption of the direction of the resisting force provided by the reinforcement at its intersection with the critical surface (*58, 152, 208, 209*).

Rowe and Soderman (*212*) describe a simple approximate method for estimating the stability of reinforced embankments. Humphrey and Holtz (*153*) have modified the slope stability program STABL to consider reinforcement.

Sliding Wedge

As described in Chapter 3, embankments have a tendency to spread laterally because of the lateral horizontal earth pressure within the embankment. The earth pressure is transmitted to the foundation by shear stresses acting at the base of the embankment. For this step, a conventional sliding wedge analysis is used (*25, 63, 64, 66*). Instability occurs in reinforced embankments when either (a) the frictional forces between the reinforcement and the embankment or the reinforcement and the subsoil are insufficient to resist the applied shearing stresses

or (b) the shearing resistance of the foundation soils just below ground surface is insufficient to maintain equilibrium. Thus the soil-geotextile friction must be sufficient to resist sliding on its plane; and the tensile strength must be sufficient to resist rupture and tearing.

Geotextile Deformations and Lateral Spreading

This analysis is performed together with the analysis for lateral spreading. To limit lateral spreading, the modulus of the reinforcement must be sufficient to develop the equilibrium tensile strength at reasonable design strains. Usually a maximum of 5 or 10 percent is commonly assumed. Additional discussion of geotextile deformation is given by Christopher and Holtz (*58*) and Bonaparte et al. (*208*).

Longitudinal Reinforcement Strength

Because embankments are normally constructed continuously along the alignment of the roadway, there is the possibility of failure at the leading edge of the construction. Both rotational and sliding wedge analyses must be performed to be sure that this type of failure does not occur during construction. Because of the way the reinforcement is rolled out transverse to the centerline, the transverse seams are the weak link in the system.

Factor of Safety

Minimum recommended factors of safety were discussed at the beginning of this chapter. They are appropriate for reinforced embankments as well as unreinforced. Two situations exist with regard to factor of safety for embankments: (a) when the calculated minimum for the unreinforced case is less than the minimum allowable and (b) when the calculated minimum for the unreinforced case is less than one. In the first case, the reinforcement is used to bring the factor of safety up to the minimum allowable, and the reinforcement is acting as a "second line of defense." In the second case, the reinforcement must work—it is the difference between success and failure. In this case, construction considerations become crucial to the success of the project.

Other Design Approaches

Although the finite-element method (FEM) has been used for a number of years to study the behavior of reinforced embankments, until recently it never developed to the stage of being a useful design tool to be used in conjunction with conventional analyses. Rowe (*213*) has adapted the FEM for use in design, and a recently completed research project at Purdue University (sponsored by FHWA and Indiana Department of Highways) had as one of its objectives to develop a simplified FEM suitable for designing reinforced embankments using a large microcomputer. This work is reported by Humphrey et al. (*214*).

Properties, Specifications, and Construction

Detailed discussion of the geotextile properties required for design is found in Christopher and Holtz (*58*). Bonaparte et al. (*208*) also include a discussion of the properties of grids. Recommendations for the determination of these properties both for design and quality-control testing are given in Christopher and Holtz (*58*).

The importance of proper construction procedures for reinforced embankments, especially on very soft foundations, cannot be overemphasized. A controlled construction sequence is required to avoid failures during construction. Factors such as site preparation, type of spreading and hauling equipment, lift thickness, type of fill, placement of the first construction lift, and placement and compaction procedures for the rest of the embankment must be considered, and these items are discussed in detail in Christopher and Holtz (*58*) and Bonaparte and Christopher (*209*). In all embankment reinforcement situations, geotextiles must be field sewn and grids positively connected along their longitudinal joints. No transverse butt seams should be permitted. As the seams are the "weak link" in any reinforcement situation, good inspection is very important to the success of reinforcement projects.

Design of Other Reinforcement Systems

As mentioned in Chapter 3, the use of other reinforcement techniques and systems besides geotextiles and geogrids to reinforce embankments is likely to be only rarely feasible. For their design, consult the references given in that chapter for the particular technique under consideration.

Stability Analyses for Other Site Improvement Techniques

In a few other foundation treatment methods, stability calculations are necessary as part of the design (e.g., surcharge/preloading embankments, displacement fill embankments). In others, such as pile-supported embankments, lime columns, and stone columns, some improvement in the stability conditions may be expected, and thus stability calculations for these systems must also be made.

Surcharge/Preloading Embankments

For surcharge/preloading embankments, the usual embankment stability calculations as discussed above are used. Some consideration of dissipation of excess pore pressure and strength gain with time is appropriate, especially with staged construction. Krizek and Krugmann (*72*), Masi (*76*), and Ladd (*77*) are recommended for more information.

Hartlén (*41*) has some comments appropriate to displacement fill design, as discussed later in this chapter.

Pile-Supported Embankments

Stability calculations for pile-supported embankments are basically no different than for any pile-supported foundation. The piles must have adequate bearing capacity, whether designed as point bearing or friction piles. The only complication results because of the tendency for embankments to spread laterally. Thus, embankment piles must be able to resist lateral as well as vertical loads. Design of piles is discussed by Vesic (*6*), Poulos and Davis (*215*), Reese (*216*), Meyerhof and Fellenius (*217*), and Vanikar (*218*). The best discussion on the design of pile foundations for embankments is given by Broms (*31*) and Broms and Wong (*52*). They give procedures for calculating the effect of piles to resist toe and translatory embankment slides, and recommendations for including the effect of batter piles on stability.

Lime Columns, Stone Columns, Etc.

How to make the calculations for the stability and bearing capacity of lime column foundations are given by Broms and Boman (*92*), Broms (*31, 95*), and Broms and Anttikoski (*39*). Barksdale and Bachus (*135*) as well as Munfakh et al. (*138*) recommend procedures for design of stone column groups under embankments. Both bearing capacity and potential slide stabilization are included.

Shallow Surface Excavations

As discussed in Chapter 3, the excavation and replacement of soft-foundation materials must, of course, remove the soft materials; otherwise their consolidation could adversely affect the pavement or shoulders of the roadway. The extent of the removal of soft materials depends on their thickness, and design of the excavation is ordinarily made by empirical rules, as discussed by Moore (*34*), Broms (*31*), and Hartlén (*41*) and discussed in Synthesis 8 (*4*). These rules are shown in Figure 27. In Figure 27a and b, the width of the excavation depends only on the height and slope of the embankment. Note that in Figure 27a, the depth of unsuitable materials is limited to deposits less than 5 ft (1.5 m) in thickness. In Figures 27c and d, the width of the excavation also depends on the thickness of the unsuitable material (*4*).

Ordinary slope stability calculations could be used to verify that these excavated slopes will remain stable during construction, but Moore (*34*) has found that lesser widths of excavation than indicated by stability analyses are often adequate. Further, if the water level is maintained at its natural level, slope stability during excavation is usually not a problem.

As noted by Holtz and Kovacs (*17*), the critical condition for excavations is the long term. However, short-term stability can usually be satisfactorily evaluated by total stress analyses using UU strengths. If the clays are fissured, these procedures are not so reliable, and the excavation should be made with considerable care.

Thin layers (i.e., about 5 ft or 1.5 m thick) of soft soils at the ground surface are often stripped before placement of the embankment fill. In addition, the near-surface soils, especially where they are highly organic, are sometimes excavated to shallow depths (i.e., about 10 ft or 3 m) and replaced with suitable fill materials, even where soft soils exist to considerable depths. This process is expensive and often counterproductive because of stability problems. These surface soils are often desiccated, fairly stiff, and contain roots and other natural reinforcement. Also, the excavated materials have to be properly disposed of, which may present problems (Chapter 3).

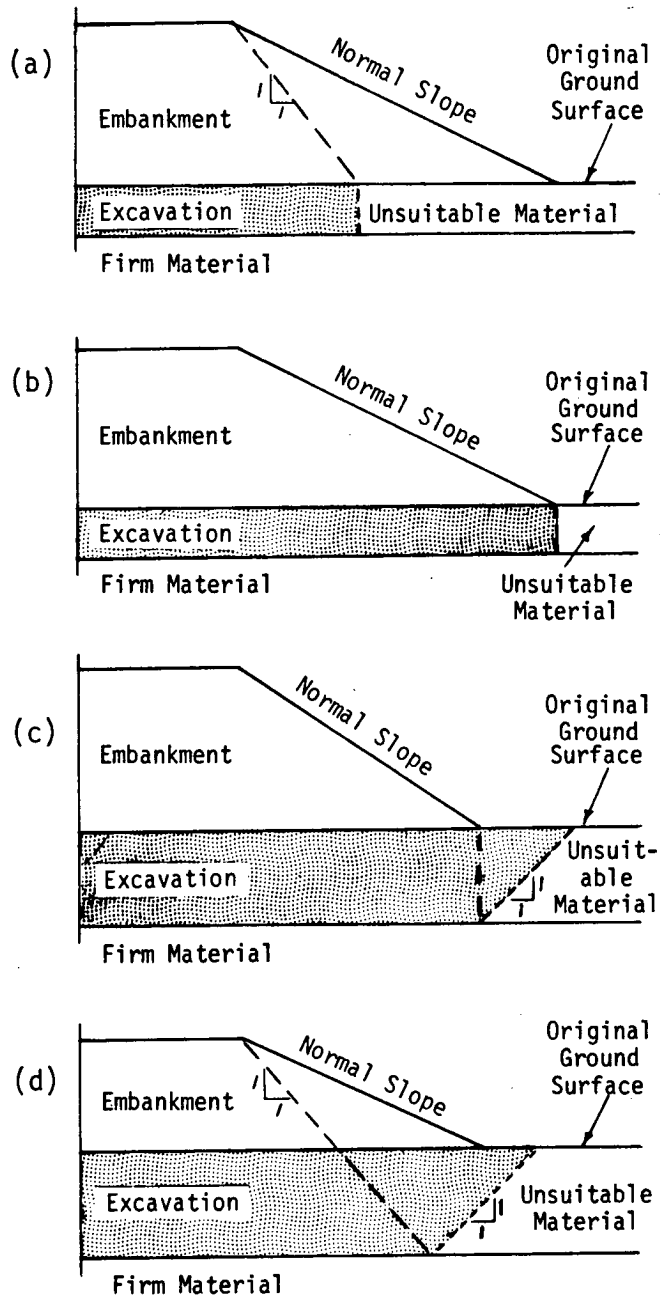


FIGURE 27 Empirical rules for excavation of soft soils (*4*).

For high fills, it is doubtful whether it is necessary to excavate the soft organic surface soils, especially if they are peats. Such soils consolidate rapidly, and primary consolidation is probably completed while the embankment fill is being placed. With low fills, organic soils probably should be excavated or otherwise treated (e.g., by surcharge) to avoid rough-riding pavements

because of differential settlements. At sites where fibrous peats and other organic soils are found overlying soft inorganic clays and vertical drains are used to stabilize the underlying soft clays, the peaty surface materials can often be left in place because they will be stabilized together with the underlying soft soils. Another situation where the surface materials should be left in place is when the embankment is going to be reinforced. The near-surface vegetative or root mat and/or dry crust will help increase stability, even if the contribution of these materials cannot be accurately accounted for in stability calculations. On the other hand, if the displacement method (Chapter 3) is being used to remove soft materials, then the surface vegetative mat and dry crust must be at least partially excavated or broken up to facilitate displacement (*61*).

For backfilling and replacement of the excavated materials, if the excavation is dry, ordinary embankment construction procedures are followed (*4*). Problems associated with fill placement underwater were discussed in Chapter 3.

SETTLEMENT ANALYSES

If the stability analyses indicate an acceptable factor of safety, the next step is to estimate settlement of the embankment. The three components of settlement (immediate, consolidation, and secondary) were discussed in Chapter 3. Depending on the soil type, one or more components may dominate; all should be checked, at least on a preliminary basis, to see if detailed calculations are warranted.

Immediate settlements of highway embankments on cohesive soils occur during construction and only rarely are of importance. If necessary, they may be estimated by elastic theory; good discussions of usual practice can be found in References *25, 62,* and *63*. For granular soils, see these references as well as *8* and *64*.

Consolidation (primary) settlements are usually estimated assuming one-dimensional compression, although for narrow embankment widths and large thicknesses of compressible materials, two-dimensional loading is more appropriate. For these cases, a two-dimensional stress distribution using elastic theory or the 2:1 method (*17, 25*) is used to predict the applied stresses on the compressible layer. TRB Special Report 163 (*65*) describes procedures for estimating consolidation settlements, as do the textbooks by Terzaghi and Peck (*62*), Perloff and Baron (*25*), and Holtz and Kovacs (*17*). Secondary compression is also discussed in TRB Special Report 163 (*65*) and by Holtz and Kovacs (*17*).

There is another source of settlement that is not ordinarily considered in settlement analyses, but in certain circumstances could be a significant contribution. This is the settlement that results from horizontal deformations that occur in the foundation at the edge of the embankment. Such settlements are significant when the safety factor against sliding is low (about 1.25 or less), the thickness of the compressible layer is relatively large, and consolidation occurs slowly. Finite-element analyses (*219*) and field observations (*220–222*) have shown that under these conditions, lateral creep deformations at the edges of embankments result in substantial vertical settlement. Where consolidation is expected to occur slowly, excessive lateral creep deformations can be reduced by using safety factors against sliding of at least 1.4 or 1.5. If this is not feasible for some reason and accurate predictions of both vertical and horizontal deformations are required, then special laboratory tests and either finite-element analyses or empirical procedures (*222*) must be used. Lateral creep effects can be serious for structures such as culverts whether they are in the embankment or foundation.

Settlement analyses, as mentioned, are an integral part of any foundation design, with or without foundation treatment. However, in the case of soil stabilization by consolidation, settlement analyses are the key to the design. They are also an important part of the design of partial excavation and replacement, reinforcement with geotextiles and geogrids, and stabilization with lime and stone columns. All these topics will be discussed in this section.

Stabilization by Consolidation: Surcharge/ Preloading Design

For the design of surcharge fills and stage construction, procedures suggested by Johnson (*71*), Krizek and Krugmann (*72*), Masi (*76*), U.S. Navy (*66*), Jamiolkowski et al. (*73*), Choa (*75*), and Ladd (*77*) are recommended. As mentioned in Chapter 3, the purpose of the surcharge loading is to minimize the immediate settlement if it is a problem, eliminate the primary consolidation as much as possible, and in some cases reduce the secondary compression. The design objective may be either to (a) determine the magnitude of the surcharge (p_s) required to assure that in a given length of time the total settlement caused by the final load (p_f) will be complete or (b) determine the length of time required to achieve a certain amount of settlement under a given surcharge load (*75*). Of course, the preloading design should be optimized with respect to cost, time, and the desired performance.

Designs to reduce the length of time for primary settlement to occur are quite straightforward (see Figure 10). From the settlement curves for the two loads p_f and p_{f+s}, the time for surcharge removal t_{sR} can be determined. Without vertical drains, this time can be quite long, with the middle portion of the clay layer still consolidating, as shown in Figure 28. At this time, the average degree of consolidation is

$$\overline{U}_{sR} = s_f/s_{f+s} \tag{3}$$

To avoid further consolidation settlements under the load p_f, it is necessary to remove the surcharge p_s no earlier than the time t_{sR}, at which the average degree of consolidation is not less than

$$\overline{U}_v = p_f/p_{f+s} \tag{4}$$

If this condition is satisfied, then upon removal of the surcharge p_s, all the succeeding compression will be secondary. Design of surcharge fills should almost always eliminate the primary consolidation.

The reduction of secondary compression settlements to tolerable values is applicable to practically all nongranular soil types, including peats, provided adequate designs are made. A number of projects have been designed to remove one cycle of secondary compression settlements (*71, 83*), and this is recommended for heavily traveled roads. Reduction of secondary compression may be important not only with peats that consolidate rapidly and have a large secondary compression, but

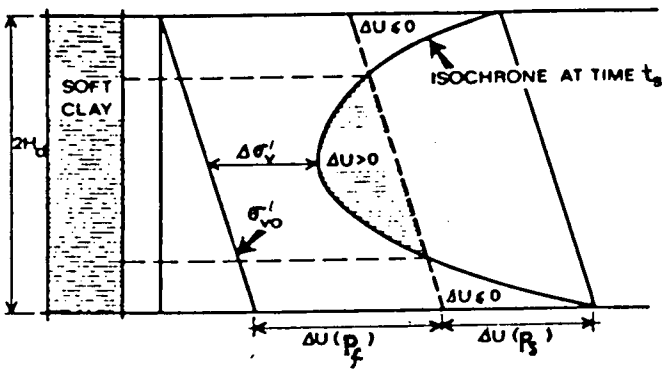


FIGURE 28 Pore pressure after surcharge removal at $t = t_{sR}$ (*71, 73*).

also when vertical drains are used, because the primary consolidation can take place quite rapidly.

Jamiolkowski et al. (*73*) suggest that a rough estimate of the amount of surcharge (AOS) required to achieve a given reduction of the rate of secondary compression may be found from Figure 29, which is a compilation of field data by Ladd (*223*). AOS is defined as

$$\text{AOS} = \frac{\sigma'\text{vs} - \sigma'\text{vf}}{\sigma'\text{vf}} \times 100\ (\%) \qquad (5)$$

where

σ'_{vs} = in situ consolidation stress caused by both p_f and p_s,
σ'_{vf} = final in situ consolidation stress caused by p_f only,
$C_{\alpha\epsilon}$ = coefficient of secondary compression ($\Delta\epsilon/\log t$),
t_p = time for primary consolidation, and
t_{sR} = time at which the surcharge is removed; $t_{sR} \geq t_p$.

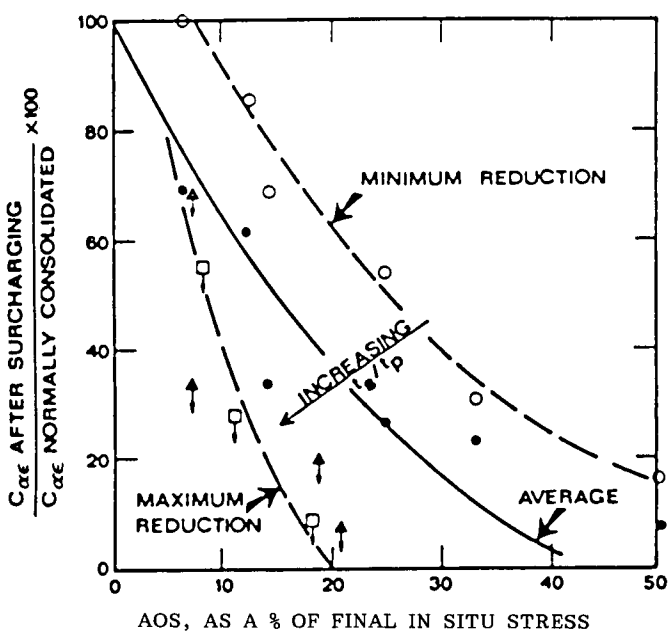


FIGURE 29 Amount of surcharge (AOS) to reduce secondary compression (*223*).

Note that AOS is not identical with p_s, but it does depend on the magnitude of p_s. To effectively reduce the rate of secondary settlement, the time of surcharge t_s must be greater than t_p; then the value of $C_{\alpha\epsilon}$ corresponds to the overconsolidated state and hence a reduced rate of secondary compression can be expected. Otherwise, the rate of secondary compression will only be temporarily but appreciably reduced, and after some unknown time it will reappear at a rate controlled by the coefficient $C_{\alpha\epsilon}$ corresponding to the normally consolidated state. These effects are shown in Figure 30.

As noted by Choa (*75*), this proposed method of precompression design is somewhat controversial and may lead to significant errors. But the alternative is to use a more complex analysis based on, for example, the Gibson and Lo (*224*) model, although modifications to this model by Edil (*225*) look promising for peats.

Preload design should consider the effect of submergence of the lower portion of the embankment into the groundwater table as consolidation takes place. This, of course, will reduce the effectiveness of the available weight of embankment and surcharge loading because the total unit weight becomes a sub-

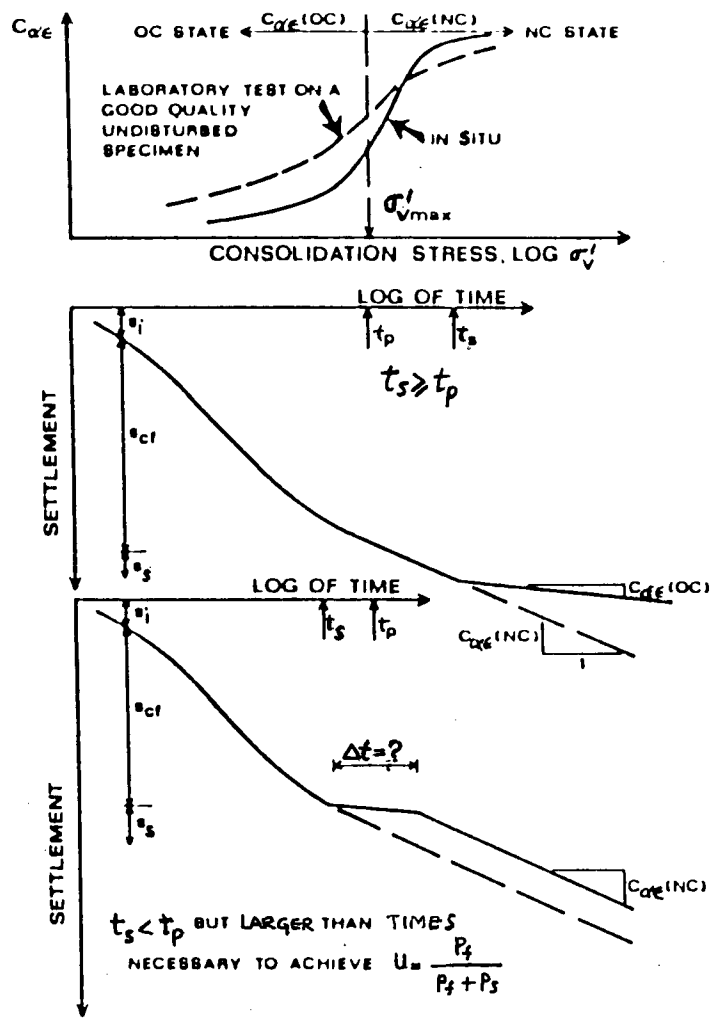


FIGURE 30 Effect of t_s/t_p ratio on rate of secondary compression (*73, 75*).

merged unit weight. Another practical consideration is the placement of the surcharge fill on the final embankment. The surcharge fill should not extend beyond the toes of the final embankment, otherwise there will be additional settlement caused by the load outside the final embankment limits. Because of the stable angle of repose with some granular fills, this requirement may be difficult to achieve, and alternative surcharge methods discussed in Chapter 3 [e.g., the vacuum method (*80*)], may be appropriate instead.

Stabilization by Consolidation: Vertical Drain Design

All of the common procedures for the design of vertical drains are based on extensions of the Terzaghi consolidation theory to radial drainage made by Barron (*226*) and Kjellman (*227*). Good references on design include Johnson (*83*) for sand drains, and for prefabricated vertical drains, the papers by Hansbo (*84*), Jamiolkowski et al. (*73*), Choa (*75*), Rixner et al. (*86*), and Holtz et al. (*88*) are especially recommended.

Soil and Drain Parameters Required

The parameters required for design and performance of precompression with vertical drains (and without, too, for that matter) on soft cohesive deposits are:

- Coefficients of consolidation for horizontal (c_h) and vertical (c_v) flow;
- Stress history of the deposit (σ_p', OCR);
- Extent of the smeared zone (d_s), and permeability coefficient of the remolded clay within the smeared zone (k_R).
- Discharge capacity of the drains (q_w) and its variation with total lateral stress (σ_h) and time (t). This factor is important for very long drains.

Site Investigation and Laboratory Testing Program

Because the preliminary subsurface investigation program is unlikely to provide sufficient information for a detailed design of the vertical drainage system, additional sampling and in situ testing will probably be necessary. The particular program will depend on the size of the project and a preliminary assessment of whether the clay deposit is likely to possess a strong macrofabric. As shown by Jamiolkowski et al. (*73*), for the successful use of precompression techniques and vertical drainage to speed up consolidation of a soft-clay deposit, a comprehensive program of in situ and laboratory tests is recommended. The program should determine the following information:

- Macrofabric, geometry of the drainage paths, and the drainage boundaries;
- Consolidation and permeability characteristics in both vertical and horizontal directions;
- Stress-strain and strength characteristics; and
- Stress history of the deposit.

The importance of the first two items for the prediction of the magnitude and rate of settlement is obvious. The stress-strain and strength properties are required because a stability analysis of the preloading fill will probably be necessary. Depending on the strength of the foundation, stage construction may be required to avoid a failure during construction; therefore, an evaluation of the partially consolidated strength may also be necessary. Stability analyses are discussed earlier in this chapter. The importance of the last item, stress history, and its impact on design has often been neglected by designers. Whether there is a real need for preloading and vertical drainage depends on the relationship between stress history and the magnitude of total stress imposed on the deposit. Also, as shown in Figure 31, there is a strong dependence of c_h on the stress history of the deposit. This figure also shows why it is important to select c_h at an effective stress level corresponding to that in situ; otherwise erroneous results will be obtained. These points are discussed in some detail by Jamiolkowski et al. (*73*) and Holtz et al. (*88*).

To obtain more reliable predictions of settlement and consolidation rates, the recommendations in Table 11 have been developed. They represent a compromise between in situ and laboratory techniques. The advantages and limitations of each and the uncertainties involved in the evaluation of c_h (and c_v) from pore pressure and settlement data obtained from instrumented trial embankments are discussed by Jamiolkowski et al. (*73*) and Holtz et al. (*88*).

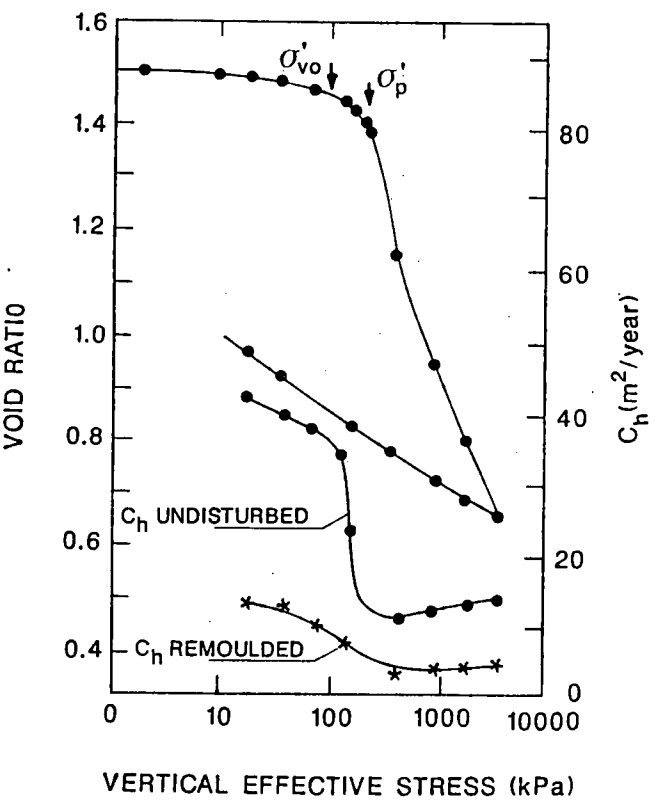


FIGURE 31 Effect of stress history and soil disturbance on the coefficient of consolidation (*73, 88*).

TABLE 11

RECOMMENDED PROCEDURES TO IMPROVE THE RELIABILITY OF PREDICTIONS OF SETTLEMENT AND CONSOLIDATION RATES USING VERTICAL DRAINS (*73, 88*)

For All Projects	Investigate the macrofabric and drainage paths of the deposit by means of borings with preferably continuous undisturbed sampling and piezocone testing	
When dealing with deposits of:		
	Uniform Clay	Clay with Macrofabric
For Small Projects	Laboratory c_h	Field k_h + laboratory m_v
	Piezocone (dissipation tests)	Piezocone (dissipation tests)
For Large Projects	Field k_h + laboratory m_v	Instrumented trial embankment
	Piezocone (dissipation tests)	Field k_h + laboratory m_v
	Instrumented trial embankment[a]	Piezocone (dissipation tests)

[a]may not always be necessary

Selection of the Drainage System

As mentioned in Chapter 3, today sand drains have been made virtually obsolete by prefabricated vertical (wick) drains. However, there may be situations where a comparison with sand drains would be helpful. To this end, Holtz et al. (*88*) have provided information on the various types of sand drains and their suitability for certain soil and project conditions.

Design of the Drains

The area needed to be drained can be determined from the site plan. The soil profile established during the subsurface investigation program together with the anticipated applied loads usually will determine the lengths of the drains required. The input required to the design equations includes:

- Soil properties,
- Boundary and drainage conditions,
- Desired degree of consolidation,
- Drain installation pattern assumed (square or triangular),
- Equivalent diameter of the prefabricated drains.

Output obtained is the drain spacing. The Barron-Kjellman design equations can be found in the references given at the beginning of this section. The equations are easily solved using a programmable calculator or microcomputer; in addition, Hansbo (*84*) has provided charts to help solve the equations. The drain spacing may be varied to optimize the design. Closer spacings increase installation costs. Hansbo (*74*) gives an example comparing the cost of preloading with vertical drains versus piles for a small building site on soft clay. The better choice depends on the cost of surcharge fill, piles, and how much time is available for the preloading.

It may be necessary for critical designs to also consider analytically the smear, well resistance, combined radial and vertical drainage, and time-dependent application of the embankment load. Details for making these calculations are given in Jamiolkowski et al. (*73*) and Holtz et al. (*88*). Rixner et al. (*86*) also discuss how to take into account smear and well resistance.

Finally, the material characteristic of the prefabricated drains themselves must be appropriately considered. Required information includes:

- Transverse permeability of the filter sleeve,
- Discharge capacity of the drain, considering lateral stress, possible drain folding or kinking, and siltation,
- Mechanical properties, and
- Durability.

Information on these factors is presented by Rixner et al. (*86*) and in somewhat more detail by Holtz et al. (*88*). Evaluation of new drains is also discussed in these two references. Rixner et al. (*86*) and Holtz and Christopher (*228*) give sample drain specifications.

Installation, Inspection, and Evaluation of Results

The details of most foundation treatment construction operations are usually left to the contractor, but in the case of vertical drainage projects, there are some construction and installation details that the designer should consider. These include the type and shape of the mandrel, the end shoe, method of installation of the mandrel, and the sequence of the installation operations. Other construction considerations include site preparation, a working platform, fill placement rates and/or stages, removal of the surcharge, and alternative surcharges (water, vacuum). As with most geotechnical construction, well-trained and conscientious field personnel are often crucial to the success of the project. Visual inspection by the design engineer or his/her representative of all aspects of the construction (site preparation, drain installation, fill placement, surcharge removal, if any, etc.) is essential.

In many preload and vertical drainage designs, it is necessary to be able to control the rate of fill placement, and this is best done by a program of field instrumentation and measurements. Required are piezometers, settlement platforms, inclinometers, and plan and elevation surveys (Chapter 7). For major projects, such instrumentation programs are essential, and they may also be required for small projects in which fill failure would be disastrous. Site monitoring should, of course, be carried out during construction, but also as long as practical afterwards. The objective of the long-term measurements is to verify settlement predictions made during the design phase.

If the Drains Do Not Perform as Expected...

Sometimes the soil conditions at a site are actually worse than originally anticipated, and the drains may not perform as expected. In this case, alternative possibilities are to:

- Install additional drains at decreased spacing,
- Increase the surcharge level,

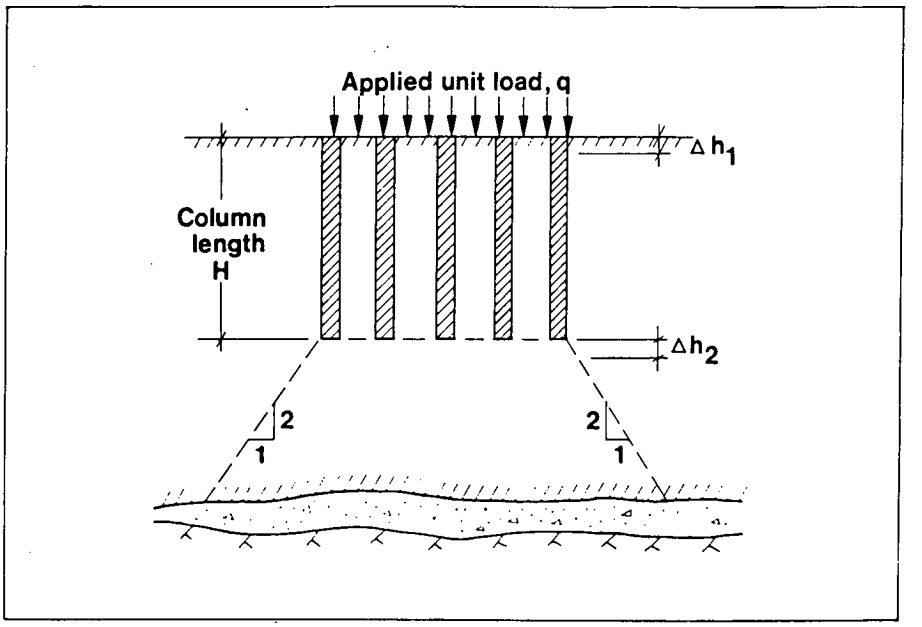


FIGURE 32 Calculation of settlement of lime columns when the creep strength of the columns is not exceeded (*92*).

- Increase the time for preconsolidation, and
- Reduce the acceptance criteria.

To aid in the decision as to what course of action to take, an investigation is advisable to find out what went wrong and why. This information will be useful for the redesign.

Lime and Stone Columns

Settlement analyses are an important part of the design of foundations stabilized by lime columns and stone columns. Both total and differential settlements must be estimated, and these calculations become especially important when the stabilized depth is less than the thickness of the compressible material. This case is more common with lime columns because the maximum length of the columns is 10 to 15 m (35 to 50 ft), and the soft-clay sites where they are used may be much deeper.

Broms and Boman (*92*), Broms (*31*), and Broms and Anttikoski (*39*) give details for the settlement analyses for lime columns. For total settlement, two cases are analyzed. The first is applicable to situations where the applied loads are relatively low, and the so-called "creep strength" of the columns is not exceeded (Figure 32). In this case, the relative stiffness of the columns with respect to the unstabilized soil governs the load distribution; compression is calculated using elastic theory. Settlement of the compressible material below the ends of the columns is calculated as if all the stress were transferred to the tips of the columns and then distributed to the compressible layer using a 2:1 stress distribution. For the case of high loads exceeding the "creep limit," the analysis is slightly more complex but appears to give reasonable results (Figure 33). The analysis

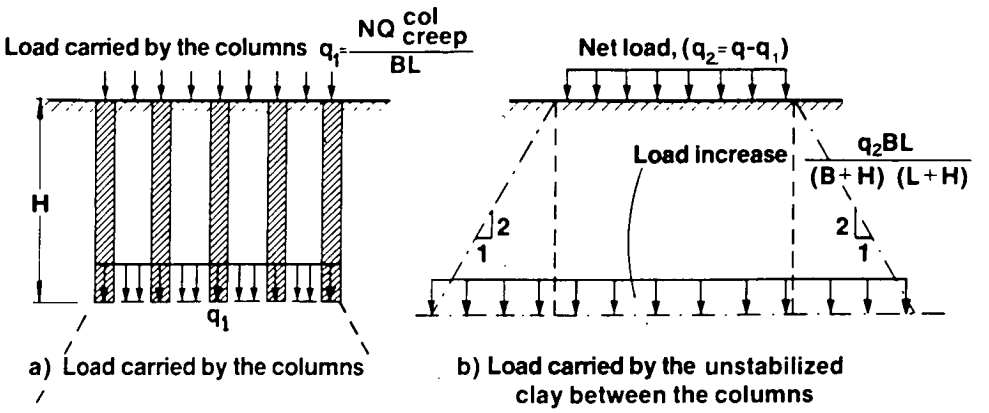


FIGURE 33 Calculation of settlement when the creep strength of the lime columns is exceeded (*92*).

for differential settlement shows that in order to limit the maximum amount of differential settlement, the column lengths should be of the same order of magnitude as the width of the loaded area (which is a rather narrow embankment with 15 m long columns). There is some evidence that the columns act as drains, which should increase the rate of settlement significantly.

Settlement analyses for foundations stabilized with stone columns are either (a) simple, approximate methods or (b) rather sophisticated methods using the theories of elasticity and/or plasticity, according to Barksdale and Bachus (*135*). They and Munfakh et al. (*138*) summarize the various methods in each category, but no single method stands out as the one to use. When theoretical predictions are compared with field measurements, the results are not very encouraging as far as settlements are concerned, but these analyses have shown that significant reductions in settlement occur if the columns are closely spaced and penetrate completely the clay layer. Like lime columns, stone columns act as drains so the time rate of consolidation for a foundation with stone columns is significantly increased.

Although theoretical settlement predictions of partially penetrating stone columns can be made, there is little field evidence to verify the results. However, charts developed using elastic theory and finite-element analyses by Balaam et al. (*229*) are simple to use and probably satisfactory for preliminary design purposes.

Reinforced Embankments

In the discussion of the design of reinforced embankments earlier in this chapter, the analysis to limit deformation of the reinforcement caused by the tendency for the embankment to spread laterally was described. In addition, a conventional settlement analysis of the reinforced embankment must also be a part of any reinforcement design, and for this the usual settlement analysis procedures can be followed.

There is some limited evidence from finite-element studies (e.g., *230*) that differential immediate (undrained) settlements may be reduced somewhat by the presence of the reinforcement. Long-term consolidation and secondary settlements are probably the same, whether reinforced or not, because the compressibility and applied stresses are hardly altered by the presence of the reinforcement. See Holtz (*152*) for a discussion of the relative creep rates of the reinforcement and the subsoil.

DESIGN OF OTHER FOUNDATION TREATMENT ALTERNATIVES

Pile-Supported Roadways

The best discussion on the design of embankment pile foundations is given by Broms (*31*) and Broms and Wong (*52*).

Bearing capacity and stability of pile-supported embankments were discussed earlier in this chapter. Settlements of embankment pile foundations are treated much the same as for other types of pile foundations, although because of the batter piles and often very soft clays at embankment pile sites, lateral displacements and negative skin friction can be problems.

Broms (*31*) and Broms and Wong (*52*) give some recommendations for the required size of the pile caps, which depends on the spacing of the piles and the height of the embankment (Figure 34).

Excavation/Removal and Replacement

There are a number of interesting geotechnical design problems involved in stabilization by excavation/removal and replacement (Chapter 3). For one thing, the excavated slopes must be stable until the backfilling operations can be completed. Design of excavated slopes follows common slope stability analysis procedures, some of which were mentioned earlier in this chapter. Problems associated with shallow excavations were also treated in that same section. Bearing capacity of the new foundation should be no problem, provided the excavation or displacement was sufficiently deep in the first place. Settlement analyses for the soft materials left below the bottom of a partial excavation (Chapter 3) are carried out exactly as for ordinary embankments on soft foundations. Appropriate subsurface information and geotechnical parameters are, of course, necessary.

When soft soils are removed by displacement using the embankment fill, perhaps in conjunction with partial excavation or blasting, some rather specialized design procedures are required. Hartlén (*41*) outlines the experience in Sweden with this stabilization method. As the displacement operation requires progressive failure of the foundation, the height of fill required to cause failure (Figure 35) can be calculated from, e.g., Taylor's (*197*) stability chart, provided the shear strength of the soil is known; or $H = c_u/0.181(\gamma)$. This analysis neglects any shearing resistance of the fill itself; to account for the fill strength, a rough estimate is to increase the calculated surcharge height (ΔH in Figure 35) 25 to 35 percent. As mentioned in Chapter 3, this height can be reduced if a partial excavation is provided in front of the fill. Hartlén (*41*) also gives some procedures for estimating the depth of penetration of the fill into the subsoil, and discusses how to use explosives to soften the soils ahead of the displacement operations (Figure 36). Swedish practice places

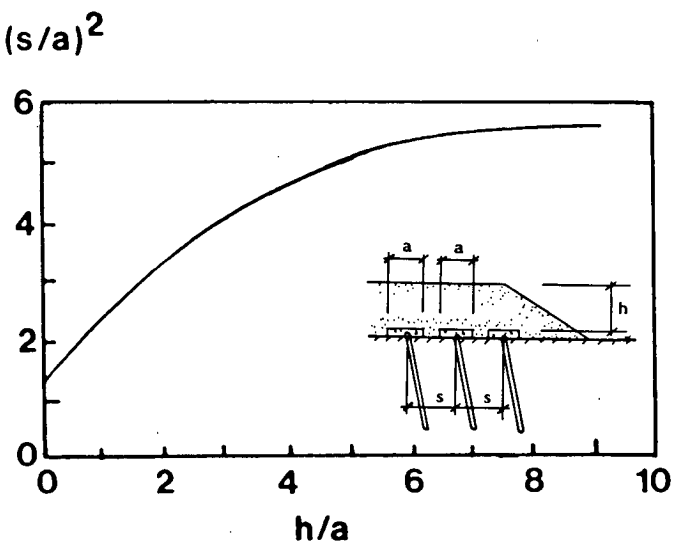


FIGURE 34 Relationship between the size of the pile cap, height of embankment, and pile spacing used by the Swedish Road Board (*31, 52*).

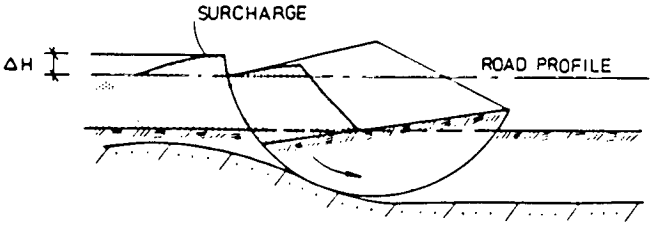


FIGURE 35 Calculation of failure in displacement operations (*41*).

the charges in a square pattern, with holes 4 to 8 m (13 to 26 ft) apart; the quantity of explosives varies between 0.05 to 0.1 kg/m^3 (0.08 to 0.16 lb/yd^3) of soil treated. Casagrande (*61*) gives a very comprehensive discussion of the displacement of soft soils and peats by blasting, including valuable design recommendations.

Chemical Alteration and Stabilization

Stabilization of soft and problem soils by chemical admixtures and other alteration techniques was described in Chapter 3. Some techniques, such as lime columns, involve a complete geotechnical analysis and design, whereas the design of others, such as grouting and electro-osmosis, is usually left to the specialty contractors who do such work.

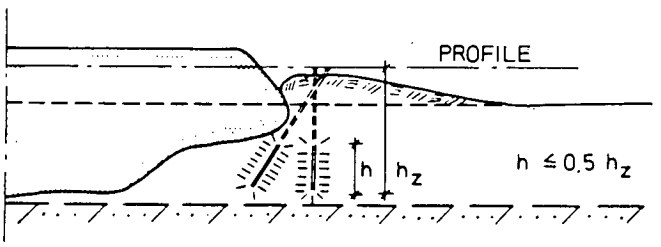


FIGURE 36 Example of front edge blasting (*41*).

The various aspects of lime column stability and settlement analysis were discussed in those respective sections earlier in this chapter. Basic references on design include Broms and Boman (*92*), Broms (*31*), and Broms and Anttikoski (*39*).

As mentioned, the design of grouting and injections is often left to the specialty contractors. However, some design information is given by Mitchell (*32*), Baker (*101, 102*), Anagnosti (*107*), Miki (*108*), and Welsh (*40*).

Specialty contractors are usually heavily involved in the design of electro-osmosis, electro-kinetic stabilization, ground freezing, and thermal stabilization. Many of the references given on these techniques in the discussion in Chapter 3 also include some design information.

Physical Stabilization and Densification

As discussed in Chapter 3, stabilization of loose deposits of granular and waste materials can be achieved by such methods as dynamic compaction, blasting, and a number of vibrocompaction and vibroreplacement techniques. Even though many of these techniques would actually be applied in construction by specialty contractors, virtually all of them are amenable to a thorough geotechnical analysis and design.

For the geotechnical design of treatment by dynamic compaction, the comprehensive report by Lukas (*122*) should be consulted. Also, Welsh et al. (*125*) present some useful information for design using this technique. Design variables include the amount of energy applied at each drop point and per unit area, the desired depth of improvement, how that improvement is to be measured, field test sections, etc.

Blasting for densification is discussed by Mitchell (*32*) and Sutton and McAlexander (*96*).

Geotechnical design of vibrocompaction systems for loose granular materials is discussed by Mitchell (*32*), U.S. Navy (*105*), Massarsch (*130*), and Welsh et al. (*125*). The various aspects of stability and settlement analyses for stone columns have been discussed earlier in this chapter. Good design information for sand compaction piles as well as stone columns is available in Broms (*31*), Mitchell (*32*), Barksdale and Bachus (*135*), Broms and Anttikoski (*39*), and Munfakh et al. (*138*).

CHAPTER SEVEN

CONSTRUCTION AND PERFORMANCE MONITORING

INTRODUCTION

The geotechnical design process does not end when the plans and specifications have been signed, sealed, and delivered. The design is not finished until the construction has been completed and the embankment is performing satisfactorily. This is true for highway embankments, and it is especially true for embankments in which some type of soil improvement and foundation treatment has been carried out. To ensure satisfactory construction and performance of the completed embankment, careful, competent inspection during construction is essential. Visual observations and physical testing are obviously important components of construction inspection; perhaps not so obvious is that geotechnical instrumentation for making 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.

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 on site. Good inspection is a requirement for almost all foundation work, because it is easy in such work for mistakes to get buried. With most if not all ground improvement techniques, so much is dependent on the success of the treatment that inspection is not the place to be "penny wise and pound foolish."

In selecting foundation treatment alternatives, the designer should keep in mind the level of inspection required by the treatment method and the competency of the field personnel likely to be available on site. Ideally, the designer should have direct control over the field personnel on the project to ensure that the construction procedures of the treatment method are properly carried out. In lieu of this, the designer should become involved in training project engineers and field inspectors before construction, so that they will be aware of the important design concepts and key construction details in the treatment method. This is especially important if construction inspection and field control are by outside contract. To ensure success with almost all foundation treatment alternatives, the on-site project engineer must have an intimate knowledge of 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.

VERIFICATION OF FOUNDATION IMPROVEMENT

In many cases of foundation treatment, verification that the method was successful is simply a matter that no obvious failures occurred and that the embankment was successfully completed. Such informal verification may be sufficient for some sites and treatment methods; for others, however, additional measures need to be taken to be sure the degree of treatment is sufficient and that it covers the intended area and depth. Of particular interest in this regard are some of the problems soils and deposits discussed in Chapter 4, in which changes in the strength and compressibility may be taking place gradually with time.

In other cases, it would be desirable to be able to verify that the treatment has, in fact, been effective before, for example, an earthquake occurs, additional embankment is constructed, or the final pavement is placed.

Ledbetter (*177*) mentions that often the required verification can be obtained by in situ testing, geophysical techniques, laboratory tests on undisturbed samples of the treated soil, and even certain analytical studies. The validity of these studies will depend, of course, on how much reliable "before treatment" information of a similar nature is available. Which of these techniques is appropriate depends on the soil or problem materials at the site, the site conditions, method of treatment employed, and time since treatment.

INSTRUMENTATION

Geotechnical instrumentation is an essential aspect of foundation engineering. Instrumentation cannot be considered "research" or "experimental," especially for many ground improvement techniques. In addition to visual observations by field personnel, measurements of movements and pore pressures are often essential to the success of a particular treatment method. In almost all the treatment methods described in Chapter 3, information from construction monitoring is an essential part of the geotechnical design. The design, as mentioned, is not complete until construction is complete, and construction cannot, in some cases, be completed safely without the information and measurements obtained from geotechnical instrumentation. Similar to the comments above regarding competent inspection, the designer should consider the possibilities for

instrumentation and field measurements in selecting the foundation treatment alternative. If there is doubt that the required instrumentation can be procured or that such measurements can be competently made, then the alternative requiring less instrumentation, even though it might mean more expensive construction, should be selected.

A valuable source of information about geotechnical instrumentation is the collection of papers in *Highway Focus* for June 1972 (*231*). The notes for the FHWA training course on instrumentation (*232*) are probably the best overall reference on the subject for highway construction, and the sections on planning instrumentation programs, specifications and procurement, and contractual arrangements are particularly recommended. Dunnicliff (*7*) is a useful summary of the training course notes; Hanna (*233*) and Dunnicliff (*234*) are good general references on geotechnical instrumentation. Another good reference is Chapter 5 in the TRB special report on landslides (*15*).

Instrumentation and Monitoring During Construction

Little or no instrumentation is required where the method of construction consists of an elevated structure of the conventional type or where soft soils are completely excavated and replaced by suitable fill materials. (Of course, a check boring should be made in the latter case.) Also, if the agency has lots of experience with pile-supported embankments or with lightweight fills, extensive foundation instrumentation may not be required. In contrast, considerable instrumentation and construction monitoring is needed where foundation treatment consists of a surcharge fill and stage loading with or without vertical drains, for many of the chemical and physical stabilization methods, and for reinforced embankments.

Appropriate instrumentation for monitoring and controlling the construction of embankments on soft foundations is listed in Table 12 and shown schematically in Figure 37 (*7, 234*). Geotechnical problems are potential failure, excessive settlements, and excessive horizontal movements, and the instruments shown are designed to provide useful information about these potential problems. Details of embankment instrumentation are discussed in DiBiagio and Myrvoll (*235*) and the other references given above. The amount and extent of instrumentation installed on a project depends on its size and particular circumstances. If a project is large and the required construction time will be long, it is desirable to install a lot of instrumentation in the early phases of the project because it may permit, for example, a reduction in size of berms or other features that could substantially reduce construction costs. Some foundation treatment methods may require special instrumentation to see that the treatment is progressing satisfactorily or that it is being applied uniformly. In other cases, it may be used to control the process (e.g., dynamic compaction, grouting) or the rate of construction of the embankment. In many reinforcement stabilization projects, it is desirable to have specialized strain gages, for example, attached to the geotextile or geogrid, or other specialized instrumentation to get an idea of the reinforcement efficiency as construction proceeds.

Although it may seem unlikely, field-observation data are often obtained but not analyzed during construction. A principal value of field measurements is to obtain information that relates to the progress of the work; hence, the data obtained have meaning only if promptly analyzed and reported. This requires a deliberate organizational effort to designate responsible personnel for this purpose, and Dunnicliff and Sellers (*232*) and Dunnicliff (*7*) give some recommendations on this point.

TABLE 12

INSTRUMENTATION FOR EMBANKMENT FOUNDATIONS

FUNCTION	TYPES AND LOCATION
Vertical settlements	Settlement platforms on original ground surface
	Settlement monuments on surface of surcharge fills and outside embankment
	Full-profile settlement gages under embankment
	Subsurface settlement gages at intermediate depths
Horizontal movements	Survey monuments on and outside embankment
	Inclinometers at toe of embankment and on slope
	Multiple point extensometers
	Inclinometers at toe of berms
Pore pressures	Piezometers at several depths and locations in foundation
	Piezometers midway between vertical drains, lime columns, stone columns, etc., if used

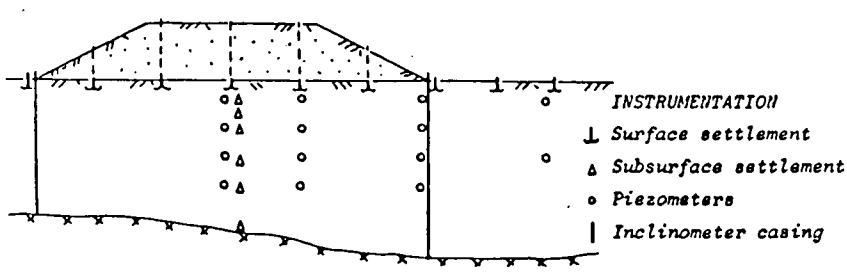


FIGURE 37 Instrumentation normally used for embankments on soft foundations (*7, 235*).

Postconstruction Monitoring

A related but perhaps even more difficult problem experienced by some agencies is that of providing for long-term postconstruction monitoring of projects in which some type of foundation improvement was attempted. If the project is apparently successfully completed with no failures or excessive deformations, there is the temptation to forget it and get on with the next job. Especially if the foundation treatment method is new to the agency and designer, it is worth considering setting up a special program to obtain the maintenance records for the project and perhaps even to obtain at the same time some profile surveys of centerline and shoulder grades. If instrumentation was installed, and it is still functioning, periodic readings and plotting of the results is also very valuable. In the case of a new treatment method or unusual soil or site conditions, a good case could be made for some use of highway planning and research funds for a formal follow-up and report to benefit not only the particular project designer, but also the geotechnical profession at large. As has probably become apparent throughout this synthesis, the great progress made in soil and site improvement during the past 10 to 20 years has largely come about through the willingness of geotechnical engineers and designers to report to others their successes and their failures. By this process, the art and practice of geotechnical engineering is advanced to the benefit of the driving public.

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AUTHOR INDEX*

*Numbers correspond to position in References.

APPENDIX A

SURVEY OF PRACTICE

1. Do you have soft or problem foundation soils and deposits in your state/region/area? Yes_____ No_____.

   If yes, what type? (Check all applicable.)

   Inorganic silts/clays _____
   Organic silts/clays _____
   Peat/muskeg _____
   Loose sands/silts _____
   Sanitary landfills _____
   Unsanitary fills (dumps) _____
   Mine wastes _____
   Strip mined areas _____
   Other (explain) _____

   _____

2. Have you had stability problems or failures with embankments constructed on these soils? Yes_____ No_____.

   Are there any particular "bad actors"? (Explain.) _____

   _____

   _____

3. What measures are usually taken to prevent instability of embankments on these soils? (Check all applicable.)

   Berms _____
   Excavation and replacement ____
   Displacement methods ____
   Lightweight fill ____
   Grouting, chemical stabilization ____
   Lime columns, cement columns ____
   Reinforcement, particularly geotextiles ____
   Vibro-replacement methods (vibroflotation, stone columns, etc.) ____
   Embankments or relief piles ____
   Electro-osmosis ____
   Other(s) (Explain) _____

   _____

   _____

4. What measures, if any, do you commonly use to reduce excessive settlements? (Check all that apply.)

   Do nothing -- accept the settlement _____
   Excavation and replacement _____
   Displacement methods _____
   Lightweight fill _____
   Preloading/surcharge _____
   - without drainage _____
   - with sand drains _____
   - with prefabricated wick drains _____
   Vibroflotation _____
   Stone columns _____
   Lime columns _____
   Dynamic compaction _____
   Blasting _____
   Grouting _____
   Embankment or relief piles _____
   Electro-osmosis _____
   Other(s) (Explain) _____

   _____

   _____

5. If you have used any of the methods of foundation soil improvement listed in No. 3 and 4 above, would you give us your opinion as to how successful the projects were?

   A. Success or Failure: By each technique, please rate the success of the project as

Highly Successful	Good	OK	Disappointing	Highly Unsatisfactory
5	4	3	2	1

   B. Why? Especially for the "disappointing" and "highly unsatisfactory" projects, would you please give us your opinion as to why this occurred?

   Possible reasons include:

   _____ a. Inadequate or erroneous geological and subsurface information; poor site investigation program

   _____ b. Poor definition of soil parameters needed for design; poor laboratory test results

   _____ c. Inadequate or inappropriate design calculations

_____ d. Poor or inadequate specifications

_____ e. Poor contractor or construction techniques

_____ f. Poor inspection during construction; inadequate instrumentation; poor follow-up

_____ g. Others? (Explain) _____

_____

_____

C. New Construction or rehabilitation and upgrading? Please tell us what the techniques were used for.

In the table below, use the above number and letter symbols in the blanks beside the improvement techniques -- or you may write in your answers, if you wish

Improvement Method	New or Upgrading?	Success Rating 5-1	If poor performance, the reason why a-g
Berms			
Excavation and replacement			
Displacement methods			
Lightweight fill			
Grouting			
Reinforcement (geotextiles)			
Dynamic compaction			
Blasting			
Vibroflotation			
Stone columns			
Lime columns			
Embankment or relief piles			
Electro-osmosis			
Preloading/surcharge o without drains o with sand drains o with prefab. drains			
Others _____			

6. Lastly, if available, would you please include with this questionnaire a copy of your specifications or special provisions for ground improvement techniques?

Thanks very much for your taking the time to fill out this questionnaire.

Name: _____

Title: _____

Agency: _____

Address: _____

_____

_____

Telephone: _____

PLEASE RETURN TO:

Mr. Thomas L. Copas, P.E.
Senior Program Officer
Transportation Research Board
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