NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 159

DESIGN AND CONSTRUCTION OF BRIDGE APPROACHES

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

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C.

JULY 1990

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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

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

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NCHRP SYNTHESIS 159

Project 20-5 FY 1986 (Topic 18-03)
ISSN 0547-5570
ISBN 0-309-04905-9
Library of Congress Catalog Card No. 90-70404

Price $8.00

Subject Areas
Pavement Design and Performance
Structures Design and Performance
Soil Foundations

Mode
Highway Transportation

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Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

Printed in the United States of America

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, bridge construction, and maintenance engineers and others interested in design, construction, and maintenance of embankment approaches to bridge abutments. Information is provided on available techniques to minimize problems associated with the "bump at the end of the bridge."

Administrators, engineers, and researchers are 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.

The transition from a roadway to a bridge structure entails design, construction, and maintenance problems. This report of the Transportation Research Board describes those problems as well as the many solutions that are applicable to specific situations.

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.

CONTENTS

ACKNOWLEDGMENTS

This synthesis was completed by the Transportation Research Board under the supervision of Robert E. Skinner, Jr., Director for Special Projects. The Principal Investigators responsible for conduct of the synthesis were Herbert A. Pennock and Martin T. Pietrucha, Special Projects Engineers. This synthesis was edited by Judith Klein.

Special appreciation is expressed to Harvey E. Wahls, Professor and Associate Head, Civil Engineering Department, North Carolina State University, who was responsible for the collection of the data and the preparation of the report.

Valuable assistance in the preparation of this synthesis was provided by the Topic Panel, consisting of Robert E. Boagni, Construction and Design Engineer, Louisiana Department of Transportation and Development; Bernard E. Butler, Associate Soils Engineer, New York State Department of Transportation; Albert F. DiMillio, Highway Research Engineer, Federal Highway Administration; Edward G. Keane, Supervisor, Geotechnical Design, Utah Department of Transportation; Donald R. McNeal, District Engineer, Pennsylvania Department of Transportation; Vasant Mistry, Construction Engineer, Federal Highway Administration; and Gary C. Whited, State Materials Engineer, Wisconsin Department of Transportation.

G.P. Jayaprakash, Engineer of Soils Geology and Foundations, Transportation Research Board, assisted the NCHRP Project 20-5 Staff and the Topic Panel.

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

DESIGN AND CONSTRUCTION OF BRIDGE APPROACHES

SUMMARY

Differential settlement at bridge abutments produces the common "bump at the end of the bridge." The problem usually is caused by compression of the embankment and foundation soils or inadequate compaction of the approach embankment. The successful design, construction, and maintenance of bridge approaches require collaboration among engineers with expertise in geotechnical, pavement, and bridge design and in construction and maintenance.

The behavior of the foundation soils beneath the embankment and abutment is one of the most important factors that affect the performance of bridge approaches. Post-construction consolidation of soft foundation soils is the major cause of embankment settlement. Lateral plastic displacements of soft deposits also may contribute to embankment settlement and, in addition, be a major cause of the lateral movement of abutments founded on spread footings or piles. On the other hand, foundation materials that are relatively incompressible create very few problems. Consequently, it is extremely important to obtain adequate information about the subsurface conditions and to analyze the responses of the foundation soils to embankment and bridge loads. The magnitude and type of site investigation should be developed by a geotechnical engineer for the specific site conditions of each project.

When the behavior of a foundation soil is inadequate for the satisfactory performance of a bridge approach, the soil behavior can be modified by reducing the embankment and abutment loads, transferring the embankment and abutment loads through the weak soil to more competent layers, or improving the properties of the foundation soil. The weak foundation soil can be removed and replaced if it is near the surface, or it can be improved by in situ densification or by reinforcement.

Most bridge-approach embankments are constructed by conventional rolled earth procedures. Because only a small amount of settlement can be tolerated at the abutment, it is good practice to specify select materials and increased compaction requirements near the abutment. The analysis and design of conventional earth embankments involves consideration of stability and settlement. The compression of the embankment also should be calculated. Reinforced embankments and mechanically stabilized walls can be used in place of rolled embankments to provide increased stability against deep slides, steeper side slopes, and greater tolerance for differential deformations.

Bridges are supported at their ends on conventional abutments (closed, stub, or spill through), on abutments that are integral with the bridge deck, or on mechanically stabilized soil walls. The abutments may be supported on spread footings, piles, or drilled shafts. The selection of a safe and economical foundation requires consideration of structural loads, environmental factors, subsurface conditions, performance criteria, construction methods, and economics. All abutments and their foundations are likely

to move. As a part of the abutment design process, the predicted movements must be compared with tolerance limits that are established to maintain the safety and function of the bridge. Also, it is necessary to consider the relative settlement of the abutment and the approach fill.

The safety and function of a bridge is affected by the post-construction movements of abutments and approach fills. Movements that occur before construction of the bridge superstructure do not affect performance or maintenance costs. Frequently the construction sequence and time schedule can be arranged to minimize the post-construction movements. This is particularly important for construction over soft, compressible, cohesive soils. Sequencing of abutment and embankment construction and stage construction of embankments are techniques that can be used to minimize post-construction settlement.

Reinforced concrete approach slabs are used to provide a smooth transition between the bridge deck and the roadway pavement. The slab usually is designed to withstand some embankment settlement and a reduction of support near the abutment. Joints must be provided to accommodate cyclic thermal movements of the bridge deck, abutment, and roadway pavement.

The role of maintenance often is overlooked in the design of bridge-approach systems. Potential maintenance costs, as well as initial construction costs, should be considered in the selection of the most cost-effective bridge and approach fill designs. However, factors other than cost also must be recognized. For example, the impact of maintenance operations on traffic should be assessed. Maintenance that requires temporary closing of one or more traffic lanes will inconvenience the public and may be unacceptable in congested urban areas, especially during periods of high traffic volume. It should be recognized that the minimum maintenance design may not be the optimal design choice. Life-cycle economics requires the combined consideration of initial and maintenance costs to determine the most cost-effective design.

CHAPTER ONE

INTRODUCTION

Differential settlement at bridge abutments is a persistent problem for highway agencies. Relatively small differential movements produce the common "bump at the end of the bridge," which is unpleasant and often hazardous to the motoring public. In addition, the impact of heavy vehicles at the bump may contribute to accelerated damage to the approach slabs and bridge decks. Frequent maintenance may be required to prevent progressive deterioration of the approach fill/abutment system, which could lead to more serious problems requiring major rehabilitation. All highway agencies have had cases in which the differential movements associated with the transition from roadway to bridge structure required considerable maintenance. Examples of the effects of differential settlement are shown in Figures 1 and 2.

Maintenance costs can be significant over the design life of bridge approaches and should be considered at the design stage of new projects. Thus it is important to identify the primary causes of differential movements and to consider designs and construction procedures that will eliminate or minimize the differential movements at the abutment-roadway transition and hence minimize future maintenance expenses. However, it also should be recognized that the minimum maintenance design may not be the optimal design choice. Life-cycle economics requires the combined consideration of initial and maintenance costs to determine the most cost-effective design.

In 1969 approach fill-abutment interaction problems were of sufficient concern to transportation agencies that bridge-approach design and construction practices were the subject of Synthesis 2 (*1*). Almost 20 years later, a survey by the Kentucky Department of Transportation (*2*) indicated that problems associated with the design, construction, and maintenance of the transition from roadway to bridge structure remain of concern to many highway agencies. Much of the information presented in Synthesis 2 still is applicable, but many new construction techniques and materials have been developed since that report was published. The purpose of this synthesis is to revise and update the 1969 report.

FIGURE 1 Settlement in excess of 2 in. at abutment (*3*).

FIGURE 2 Wingwall settlement (*3*).

CAUSES OF BRIDGE-APPROACH PROBLEMS

The performance of a bridge approach is affected by the design and construction of the bridge deck, abutment, and foundation, as well as by the roadway pavement system, embankment, and embankment foundation. Major problems usually are attributed to excessive compression of the embankment and foundation soils or inadequate compaction of the approach embankment or both. Localized soil erosion, usually associated with inadequate provisions for drainage, also may be a contributing factor. Frost heave or swelling soils can also be problems in some areas.

A recent study of 20 sites by the Colorado Department of Highways (*3*) concludes that the primary causes of bridge-approach settlements are one or more of the following:

- time-dependent consolidation of the embankment foundation,
- time-dependent consolidation of the approach embankment,
- poor compaction of abutment backfill caused by restricted access of standard compaction equipment,

- erosion of soil at the abutment face, and
- poor drainage of the embankment and abutment backfill.

All of the preceding factors may contribute to the embankment settlement. In addition, lateral creep of foundation soils and lateral movements of the abutment may cause additional detrimental movements of the embankment. However, the major bridge-approach problems are directly related to the relative settlement of the approach slab and the bridge abutment. Hence, the approach-slab design and the type of abutment and foundation also affect performance.

Abutments supported on shallow footings within the embankment will settle with the embankment and usually cause less differential settlement than abutments supported on piles. Also, the differential settlement often can be minimized by the sequence of construction of the embankment, approach pavement, abutment, and superstructure. Finally, the design of the approach slab may minimize the effects of the embankment settlement on the performance of the bridge-approach system.

ENGINEERING RESPONSIBILITIES

Satisfactory performance of the bridge approach requires the interaction of design, construction, and maintenance engineers. During the design phase the collaborative expertise of geotechnical, pavement, and bridge design engineers is required to provide adequate consideration of the interaction of the embankment, roadway, and bridge responses to traffic loads and environmental factors. Also, potential construction and maintenance procedures and costs may affect design decisions; thus, construction and maintenance engineers should be consulted during the design phase.

Similarly, although the construction engineer is responsible for construction in accordance with the design plans and specifications, it is desirable for liaison to be maintained between the construction and design engineers during construction. The involvement of geotechnical engineers is especially important because of the frequent variability of subsurface conditions. Often revisions of the geotechnical design may be required to accommodate unforeseen conditions that are encountered during construction. Changes in the design or construction procedures should be developed jointly by the construction, geotechnical, and bridge design engineers. When the design includes special foundation treatments, special instrumentation often must be installed to monitor the progress and effectiveness of the treatment during and after construction. Again, close collaboration between the construction and geotechnical engineers is required.

Maintenance engineers are responsible for maintaining the quality of the constructed facility after construction is completed. In some instances, some acceptable level of periodic maintenance may be anticipated as a part of the most cost-effective design. For example, the cost of a periodic pavement overlay at the approach slab-abutment interface may be more economical than an alternative design that would eliminate the need for the overlay. In other instances, the required maintenance may exceed that which was anticipated during design and construction. This may result from factors that were inadequately considered in the design, improper construction procedures, or unanticipated changes in design criteria (e.g., increases in traffic volume or allowable wheel loads). In all cases, communications among design, construction, and maintenance engineers are essential. When the required maintenance exceeds expectations, the design and construction engineers often can assist in the identification of the causes and the selection of the most appropriate remedial measures. Also, maintenance engineers may alert design and construction engineers to necessary modifications to future design standards and construction procedures.

Currently, most transportation agencies are identifying many bridges that, as a result of age or increased service requirements, are in need of replacement or major rehabilitation. Often major modifications of the bridge-approach systems are associated with these projects. Some of the innovations in geotechnical design and construction methods (e.g., in situ ground improvement, ground anchors, geosynthetics, and mechanically stabilized walls and abutments) are very suitable for rehabilitation projects. Consultation among geotechnical, design, construction, and maintenance engineers is essential to the selection of technically feasible and economical alternatives for the rehabilitation of bridge approaches.

In summary, the successful design, construction, and maintenance of bridge approaches require effective collaboration among engineers with expertise in geotechnical, pavement, and bridge design and in construction and maintenance. Because of uncertainties in subsurface conditions, it is especially critical that the geotechnical engineer is involved in all phases of a project. However, the organizational structure of many transportation agencies does not encourage active communication among design, construction, and maintenance engineers until problems arise. Often this is too late for the most effective resolution of problems. Some agencies have established standing geotechnical committees with representatives from roadway and bridge design, construction, and maintenance to facilitate interaction on projects before major problems develop.

SCOPE

The scope of this synthesis includes the consideration of embankment foundation conditions, approach embankment design and construction, abutment design and construction, and approach slabs. A section is devoted to methods for maintenance and rehabilitation of existing bridge-approach systems. Examples of several recent innovative bridge-approach design and construction projects are included. Special attention is devoted to techniques and materials that have been developed since Synthesis 2 was written. However, recent methods for treatment of soft embankment foundations, which are the subject of another synthesis (*4*), are introduced but not discussed in detail.

CHAPTER TWO

EMBANKMENT FOUNDATIONS

The behavior of the foundation soils beneath the embankment and abutment is one of the most important factors that affect the performance of bridge approaches. Post-construction consolidation of soft cohesive foundation soils generally is recognized as the major cause of embankment settlement. Lateral plastic displacements of soft deposits also may contribute to embankment settlement and, in addition, be a major cause of the lateral movement of abutments founded on spread footings or piles. On the other hand, foundation materials that are relatively incompressible (e.g., stiff clays, sands, gravels, or rock) create very few problems. Consequently, it is extremely important to obtain adequate information about the subsurface conditions and to analyze the responses of the foundation soils to embankment and bridge loads.

This chapter addresses the requirements of the geotechnical investigation of the foundation soils, including subsurface exploration, laboratory testing, and analyses of behavior. Also, methods for improvement of the foundation soils are presented. However, the treatment of embankment foundations is the topic of Synthesis 147 (*4*), which is an update of Synthesis 29 (*5*). Also, much information related to this chapter already is available in Syntheses 8, 33, and 107 (*6–8*). Consequently, this chapter emphasizes specific requirements for bridge approaches and only summarizes requirements for general embankment foundations.

SUBSURFACE INVESTIGATIONS

An adequate subsurface investigation is an essential prerequisite for a successful geotechnical project. Transportation agency practices for acquisition and presentation of subsurface information were summarized in Synthesis 33 (*7*), which was published in 1976. Guidelines and very comprehensive information on methodology for subsurface investigations are presented in the new edition of the American Association of State Highway and Transportation Officials (AASHTO) *Manual on Subsurface Investigations* (*9*). The reader is referred to this manual for more detailed information.

The subsurface exploration for bridge approaches should be integrated with the investigation for the bridge structure. The size and type of exploratory program should depend on the magnitude of the proposed project and the complexity of the site conditions. The investigation should provide information on the depth, thickness, and classification of all soil strata. Strength, compressibility, and sometimes permeability of critical strata must be determined. For cohesive soils, undisturbed samples usually are required for laboratory evaluation of strength and compressibility properties. However, for cohesionless soils and sometimes for very soft cohesive soils, which are difficult to sample, in situ evaluations of soil properties often are made with various types of borehole tests. The depth to rock as well as the type and quality of the rock also should be determined. Finally, the groundwater elevations should be reported.

All transportation agencies have the capability to conduct the necessary subsurface borings, including disturbed and undisturbed sampling, or are able to obtain them by contract drilling. Many are beginning to use various in situ tests (e.g., cone penetrometers, pressuremeters, or dilatometers) on selected projects. Rock coring is performed routinely by most agencies. Almost all now have adequate facilities for laboratory evaluation of strength and compressibility, including consolidation testing, which is essential for the evaluation of embankment foundations. In addition, the services of commercial laboratories are readily available when needed.

The magnitude and type of site investigation should be developed by a geotechnical engineer for the specific site conditions of each project. The AASHTO subsurface investigation manual (*9*) presents suggested guidelines for the spacing and depth of borings for structures and embankments. For embankments higher than 15 ft (4.5 m), the recommended boring spacing is a maximum of 200 ft (60 m), with the interval decreased to 100 ft (30 m) when erratic conditions or compressible soils are encountered. For each bridge abutment, a minimum of two borings is recommended, and additional borings are suggested when the abutment exceeds 100 ft (30 m) in length or has wingwalls more than 20 ft (6 m) long. The general recommendation for the depth of borings is the depth at which the net stress increase caused by imposed foundation loads is less than 10 percent of the effective overburden pressure at that depth, unless rock or dense soil known to lie on rock is encountered above that depth. This criterion should be used for borings for approach embankments. However, for abutments, it is recommended that all borings should penetrate to rock, with selected borings extending 10 to 20 ft (3 to 6 m) into rock, especially if deep foundations are contemplated for support of the abutments or special geologic features (e.g., solution cavities or mines) are anticipated.

When considering the depth and location of borings, it should be noted that, except at very shallow depths, the stress increase caused by the embankment loading generally will be significantly greater than that caused by structural loads. This is illustrated in Figure 3, which is reproduced from Synthesis 8 (*6*).

ANALYSES OF FOUNDATION BEHAVIOR

Both the compression and the stability of the embankment foundation must be considered. A good discussion of both as-

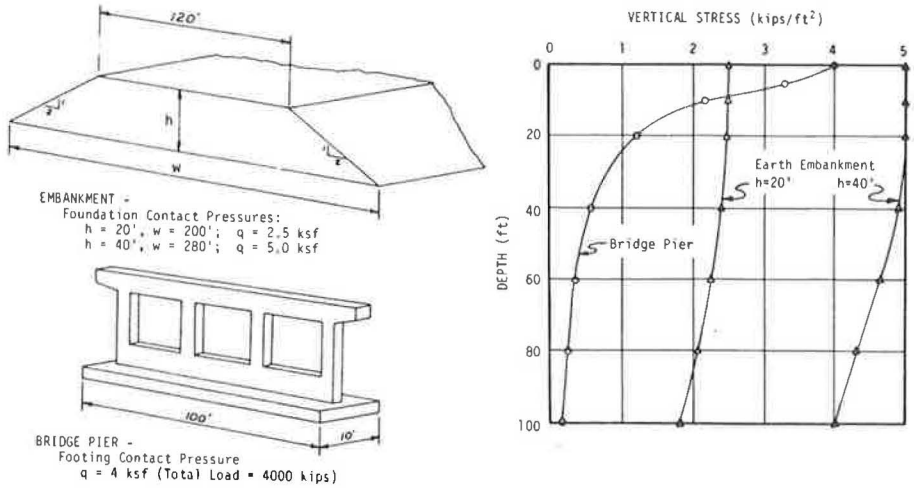


FIGURE 3 Comparison of vertical stresses beneath center lines of bridge piers and earth embankment (*6*).

pects of the behavior of soft clay foundations has been presented by Bjerrum (*10*).

Settlement Analysis

The computation of the predicted compression of the foundation soil is a very important and relatively straightforward procedure. The compressibility characteristics of the soil are used to compute the vertical strain caused by the stress increases caused by the applied loads. The dead loads of the embankment and the bridge are the primary loads that affect the foundation soil, and as seen in Figure 3, the weight of the embankment usually is predominant except at very shallow depths. The stress increase in the foundation soil usually is computed from the Boussinesq elastic theory by treating the embankment as a trapezoidal surcharge acting at the base of the embankment. Bozozuk and Lo (*11*) have noted that this assumption neglects stress distribution effects within the embankment, which leads to overestimates of the vertical stress beneath the center of the embankment. They suggest the use of the elastic stress distribution solution by Perloff et al. (*12*), which accounts for this effect. However, current practice considers the errors introduced by the use of the simple trapezoidal surcharge to be conservative, acceptable, and minor in comparison with potential errors associated with the selection of appropriate soil properties.

The compression consists of three components: initial compression, primary consolidation, and secondary consolidation. The initial compression occurs simultaneously with the application of load and thus before final grading and construction of the pavement system; consequently, it usually is not relevant to bridge-approach problems. However, both primary and secondary consolidation are time-dependent processes that are likely to continue beyond the construction period. Both the magnitude and the rate of consolidation are important to bridge-approach design and construction procedures.

For cohesive soils, the preconsolidation pressure and compression indexes usually are determined from laboratory consolidation tests and used to compute the magnitude of primary and secondary consolidation. Procedures for the settlement computation, including secondary compression effects, are presented by Bjerrum (*10*) and in Transportation Research Board Special Report 163 (*13*). Secondary compression frequently is small and neglected; however, for some soils (e.g., organic or sensitive clays) the long-term secondary compression may cause settlements that contribute to bridge-approach problems during the useful life of the facility (*14*).

Consolidation test data also are used to compute the rate of primary consolidation. This computation is used to estimate the amount of compression that will occur during the construction period and to assess the feasibility of minimizing post-construction settlement by using special construction sequences or procedures (e.g., preloads, temporary surcharges, wick drains, and stage construction). However, estimates of the rate of consolidation are less reliable than estimates of the final settlement. Hence, if the rate of consolidation is critical to construction schedules, it is essential to install instrumentation to monitor and to control the progress of consolidation during construction. Some of the construction procedures used to minimize the effects of consolidation are discussed more fully elsewhere in this synthesis.

Observed settlements of embankment foundations have been reported by many transportation agencies [e.g., Kentucky (*14*), Colorado (*3*), and South Dakota (*15*)]. Several researchers (*11, 16*) have compared predicted and observed settlements of the Gloucester Test Fill in Canada. At a 1986 workshop in Ottawa, numerous other researchers used more than 20 years of observations at this site for detailed evaluations of the reliability of various methods for the prediction of ground movements under embankments.

Cohesionless soils generally are much less compressible than normally or lightly overconsolidated clays. Also, the rate of consolidation is very rapid, and primary consolidation usually will be completed during construction. Consequently, cohesionless foundation soils are unlikely to cause significant post-construction settlement problems at bridge approaches. A recent Federal Highway Administration (FHWA) study (*17*) of 21 abutments and piers for 10 bridges supported on shallow foundations on sand reported an average observed total settlement of 0.75 in. (19 mm) but an average of less than 0.25 in. (6 mm) after construction of the bridge deck. A typical time-settlement record from this study is shown in Figure 4.

It usually is not necessary to compute the compression of sand and gravel foundation soils. For the occasional cases for which the compression of loose cohesionless soils under an embankment must be estimated, the methodology is not well established. Numerous methods have been proposed for the computation of the settlement of shallow spread footings on cohesionless soils, but most are empirical methods that are not easily adaptable to the geometry of embankment loadings. Twenty methods are noted in the FHWA report (*17*), but only a few [e.g., Schmertmann (*18, 19*) and Hough (*20*)] appear appropriate for embankment loadings. These two procedures are summarized in Synthesis 107 (*8*).

Stability Analyses

If the embankment loads produce stresses that approach the shear strength of the foundation soils, these soils may be displaced laterally, causing major movements and disruption of the embankment. This type of sliding failure in the foundation soil must be investigated carefully when the foundation is a soft cohesive soil, including peat, marl, inorganic and organic silt, and clay. However, deep basal slides generally will not occur in sands, gravels, and other materials that develop shear strength from internal friction.

The factor of safety against sliding is determined by conventional slope stability analyses, which are based on limiting equilibrium concepts. Methods that assume circular, linear, or composite sliding surfaces are widely available in the literature (e.g., *21–25*). Furthermore, the computations required for these methods may be accomplished with a wide variety of available computer software, which has been developed for hardware ranging from large mainframes to personal computers. For example, ICES LEASE is a versatile program that currently can only be used on a mainframe as a subsystem of ICES (Integrated Civil Engineering System), which was developed at MIT. Several other programs, including STABL, STABR, and UTEXAS, initially were developed for mainframes but now also are available for use on personal computers.

STABL, which was developed at Purdue University for the Indiana Department of Highways, is available from the FHWA and is used by many transportation agencies. However, some agencies have expressed concern that one version of this program, STABL4, may be overly conservative for some design conditions, especially those that require estimates of pore pressures from phreatic surfaces. More recent versions of the program have been modified to alleviate these concerns. Computer programs make it possible to investigate a large number of design alternatives, but it is good practice to perform manual checks of the computer results for the critical design conditions.

For embankments on soft cohesive soils, the end of construction is the most critical design condition for stability because the foundation soil should consolidate and get stronger with time. For this condition, it is convenient and appropriate to conduct the analysis in terms of total stresses using the undrained strength of the soil. Generally, unconsolidated undrained (UU) triaxial compression tests are recommended, but unconfined compression, in situ vane shear, and occasionally CU triaxial tests also are used (*26*). In some cases, the measured strength is corrected for disturbance, anisotropy, or strain rate effects [e.g., Bjerrum's correction factor for vane shear results (*10*)]. For major projects, the SHANSEP procedures of Ladd and Foote (*27*) may be justifiable to obtain more reliable estimates of the in situ undrained strength.

Most transportation agencies require a factor of safety of 1.25 to 1.3 for highway embankments. However, a recent study in Kentucky (*14*) suggests that large shear strains may develop in soft foundations soils near the toes of the embankment slopes when the factor of safety is less than 1.5. This is illustrated for six cases in Figure 5. Lateral creep, or plastic deformation, of the foundation soil occurs and causes settlement of the embankment and both lateral and vertical movements of abutments, even for abutments supported on piles. Similar correlations between the factor of safety against basal failure and the lateral movements of braced excavations in soft clay have been reported by Mana and Clough (*28*). Relatively complex finite-element analyses are required to estimate the magnitude of these movements (*28, 29*), and the reliability of such computations is very sensitive to the deformation properties selected to represent the foundation soil. Consequently, such finite-element analyses are not routinely used in current geotechnical practice. More commonly the potential for plastic deformation is minimized by maintaining a factor of safety of at least 1.5 for the long-term stability of the foundation soil.

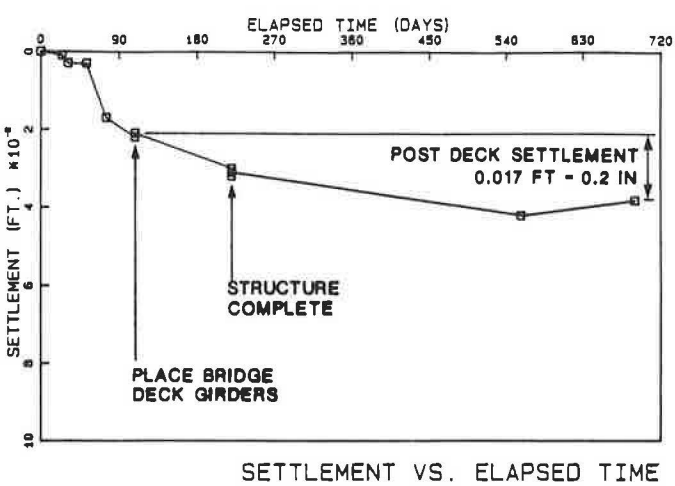


FIGURE 4 Typical time-settlement curve for abutment on spread footing on sand (*17*).

FOUNDATION IMPROVEMENT

When the behavior of a foundation soil is inadequate for the satisfactory performance of a bridge approach, the soil behavior can be modified by reducing the embankment and abutment loads, transferring the embankment and abutment loads through the weak soil to more competent layers, or improving the properties of the foundation soil. Relocation of the bridge site or replacement of the embankment with bridging are other alternatives that may be considered.

A soil whose behavior is inadequate for a specific design application often is called an "unsuitable" soil for that design. Most natural soils will be unsuitable for some applications but suitable for others with less stringent behavioral requirements. Similarly, unsuitable natural soil often may be made suitable for a given application by improving the soil properties that are relevant to that application. Volume stability, strength, durability, and

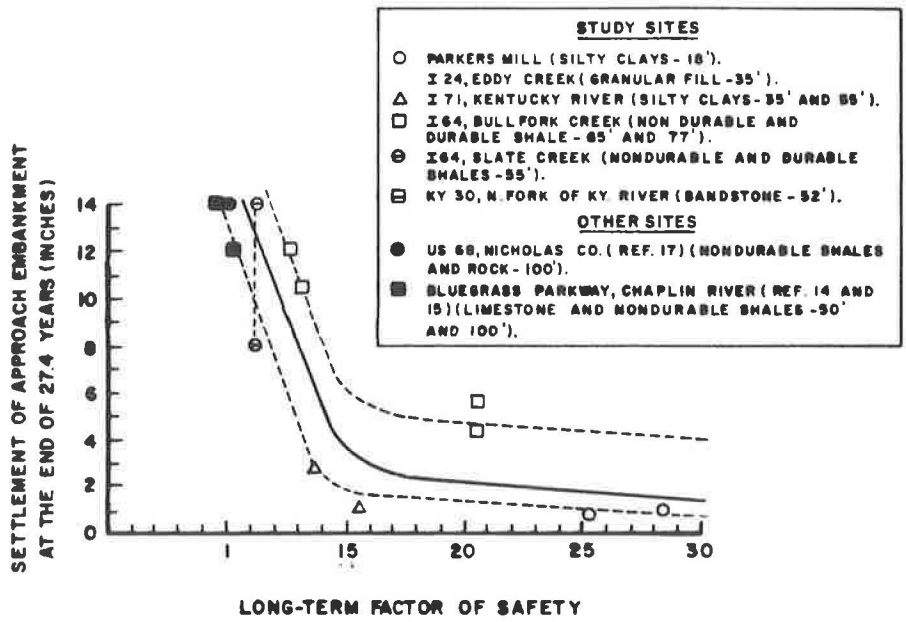


FIGURE 5 Long-term settlement versus factor of safety of foundation soil (*14*).

permeability are the most important properties for support of bridge approaches. The improvement of one or more of these properties of the foundation soil often is the most cost-effective design alternative for bridge approaches.

Several soil-improvement methods are discussed in Syntheses 2, 29, and 8 (*1, 5, 6*). However, many new and innovative ground-improvement techniques have been developed during the past 20 years. Excellent reviews of these techniques are found in Synthesis 147 (*4*) and several other publications (*30, 31*). The soil-improvement techniques that appear most appropriate for bridge-approach foundations are described briefly in the following sections. For more details, the reader is referred to Synthesis 147 (*4*).

Removal and Replacement

When unsuitable material is found at or near the surface, removal and replacement with suitable borrow material is the most commonly used improvement procedure. Simple excavation methods are most effective when the unsuitable material is relatively thin [i.e., less than 10 ft (3 m) thick] and above the water table; but removal by excavation is commonly used for deposits up to 30 ft (9 m) thick. Dredging can be used below water to even greater depths; however, the disposal of dredge spoils may become a significant environmental problem.

Organic deposits and very soft clays sometimes are removed by displacement methods. Fill is placed as an advancing front to a height necessary to overstress the soft foundation soil and to displace it laterally to the sides and ahead of the fill. The filling operations must be controlled very carefully to prevent entrapment of pockets of soft material within the embankment and to control the mud wave produced in the soft deposit. Variations in the quality of the soft soil make it very difficult to maintain uniform displacement and complete removal of the unsuitable material. Because of these uncertainties, displacement methods usually are less desirable than many of the other alternatives available today. This is especially true for bridge-approach foundations, which require more stringent quality control than regular roadway embankments.

The success of all removal and replacement procedures also depends on the proper placement of borrow material. Adequate quantities of suitable material must be available, and the fill should be compacted in accordance with the criteria established for embankment construction, which will be discussed in Chapter 3. If unsuitable material is removed below water, then special procedures may be required to dewater the excavated area or to place the fill underwater. Supplemental treatment (e.g., surcharges and waiting periods) also may be necessary.

In Situ Densification

When the removal of unsuitable foundation material is impractical or uneconomical, the unsuitable material often may be improved by in situ densification. The oldest and still most commonly used densification method for cohesive soils is the consolidation process. In recent years several other relatively new densification procedures, including dynamic compaction and compaction piles, also have been used.

Precompression

Precompression is a process that consolidates and strengthens soft foundation soils before construction of critical permanent design components. For bridge approaches, the weight of the approach embankment is used to consolidate the foundation soil. Final grading and construction of the permanent roadway and bridge components must be delayed until the desired amount of compression of the foundation soil has been completed. This process can be used effectively to reduce or eliminate the post-

construction settlement caused by primary and secondary consolidation. However, the required delay times have a significant effect on construction schedules and costs. One year usually is the maximum acceptable waiting period, and frequently much shorter delays are desired. Temporary surcharges or vertical drains, which are discussed below, sometimes are used to shorten the waiting periods.

The primary considerations in the precompression design are the magnitudes and rates of primary and secondary consolidation under both the temporary and final design loads (*32*). The strength of the foundation soil and the rate of increase in strength with consolidation also must be evaluated. If the temporary loads overstress the foundation soil, massive slides and uncontrolled lateral displacements may occur.

Surcharges Temporary surcharges, which are produced by constructing the embankment to a height in excess of the final design height, commonly are considered to accelerate the rate of precompression. The larger the temporary load, the faster is the rate of precompression. In addition, surcharges can be designed to reduce long-term post-construction creep or secondary compression. The maximum feasible surcharge is controlled by stability considerations or the availability of sufficient fill. The placement and subsequent removal of a temporary surcharge increases earthwork costs. In some cases, it may be necessary to flatten the embankment slopes or install toe berms to maintain stability of the foundation soil, which also increases construction costs. Thus, for each project the costs of a temporary surcharge must be weighed carefully against the benefits of a shorter waiting period.

Vertical Drains The rate of primary consolidation can be accelerated by installing vertical drains, which shorten the drainage path in soft low-permeability foundation soils. Vertical sand columns have been used successfully for this purpose for more than 50 years (*33*). Design procedures and case histories that document the effectiveness of sand drains are readily available in the literature (*33–35*).

Today sand drains have been essentially replaced by prefabricated vertical wick drains, which consist of a plastic core encased in a synthetic geotextile jacket. More than a dozen types of prefabricated drains are commercially available in the United States. All are band-shaped with a typical width of 4 in. (100 mm) and a thickness of 0.08 to 0.30 in. (2 to 7 mm). A special machine-driven mandrel, as shown in Figure 6, is used to install these drains very rapidly in soft soils with a minimum of disturbance (*36–38*). Detailed guidelines for design and installation of prefabricated drains are presented in a recent FHWA publication (*39*). Test procedures for evaluating the relative effectiveness of various prefabricated drains have been suggested by several agencies (*39–41*).

Performance Evaluation The successful use of precompression, with or without surcharges or vertical drains, requires careful geotechnical analyses. The soil profile must be defined clearly, and the strength, compressibility, and permeability of all compressible strata must be evaluated from laboratory or in situ tests. Design computations must include stability analyses and predictions of the amounts and rates of primary and secondary consolidation.

Field instrumentation and close construction supervision are imperative. Because of the many uncertainties involved in the prediction of field settlement rates, the rate of construction or the duration of waiting periods usually must be determined by monitoring the progress of the field consolidation. On simple projects, it may be sufficient to monitor only the settlement of the embankment surface. On more complex projects, in which stability becomes a major consideration, pore pressures and both lateral and vertical movements should be monitored within the embankment and foundation soils. Recommendations of the amount and type of instrumentation may be found in Synthesis 89 (*42*) and the new AASHTO *Manual on Subsurface Investigations* (*9*).

Dynamic Compaction

Dynamic compaction is the process of densification at depth by the repeated dropping of a heavy weight on the ground surface. As of 1986, more than 200 applications of this technique have been documented in the United States (*43*). These applications are based on concepts presented by Menard and Broise in 1975 (*44*). Typically, weights ranging from 6 to 30 tons (5,400 to 27,000 kg) are dropped from heights of 30 to 75 ft (9 to 23 m), but weights as large as 100 tons (90,000 kg) and drop heights of 120 ft (36 m) have been used (*45*). The weight is dropped several times at each point on a predetermined grid pattern (see Figure 7). Sometimes, several coverages of an area are required. Dynamic compaction commonly is effective to depths of 40 ft (12 m), but densification to greater depths has been reported in some cases (*43*). The depth of densification depends primarily on the impact energy, the type of deposit being densified, and the degree of saturation. A good discussion of these factors is presented by Lukas (*45*).

Dynamic compaction has been applied to a wide range of deposits. The suitability of various deposits for improvement by dynamic compaction is summarized in Table 1. The most suitable deposits are coarse-grained pervious soils either above or below the water table. Partially saturated semipervious materials (e.g., silty soils) also can be densified with little difficulty; however, below the water table densification of these soils becomes more difficult because excess pore pressures are generated. Dynamic compaction usually is ineffective in saturated impervious materials, such as clay and organic deposits. Because of the sensitivity of bridge approaches to small post-construction differential settlements, dynamic compaction of bridge approaches should be limited to densification of loose, clean, coarse-grained deposits.

In situ testing both before and after treatment is required to evaluate the effectiveness of deep dynamic compaction. Standard penetration tests (SPT) and cone penetration tests (CPT) are the most commonly used testing methods. Dilatometers and pressuremeters also are used frequently. Cross hole seismic tests and other geophysical logging techniques are used occasionally (*30, 45, 46*).

Compaction Piles

Deep deposits of loose cohesionless soils often can be densified by vibrocompaction or vibroflotation methods. Vibration, fre-

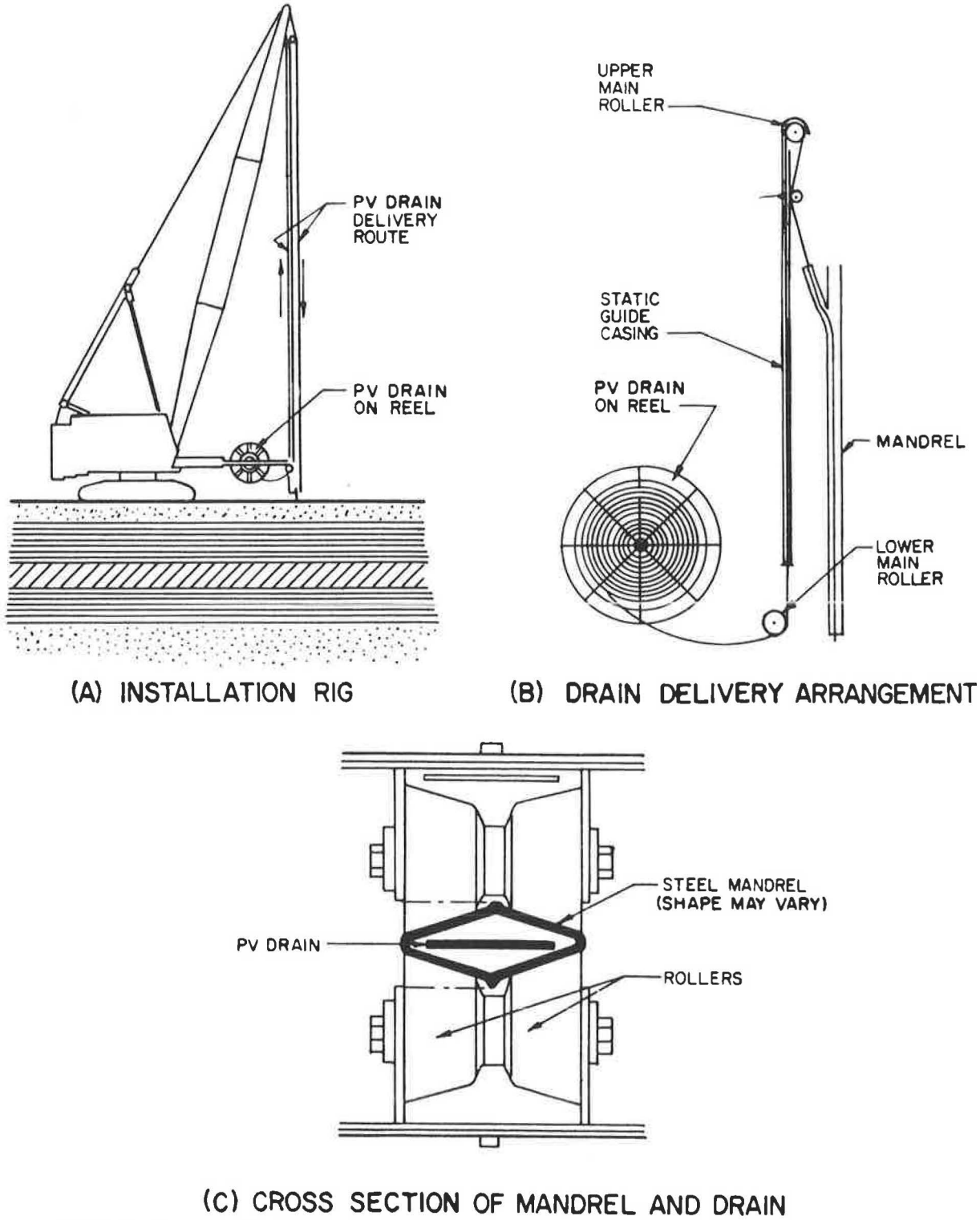


FIGURE 6 Equipment for installation of prefabricated wick drains (*39*).

quently in combination with water or air jets, is used to insert a hollow vibratory probe to the depth of desired treatment. Granular backfill is added as the probe is removed, leaving a column of compacted backfill within the in situ deposit, which has been densified by the vibration. The probe usually must be inserted at spacings of 3 to 10 ft (1 to 3 m) for effective coverage. The grain size ranges for which vibrocompaction should be considered are shown in Figure 8. Vibrocompaction is most effective in saturated cohesionless sands that contain less than 20 percent fines. For such soils, it is reported that densification can be achieved routinely to depths of 50 ft (15 m) and in some instances to depths approaching 100 ft (30 m) (*30, 43, 47*).

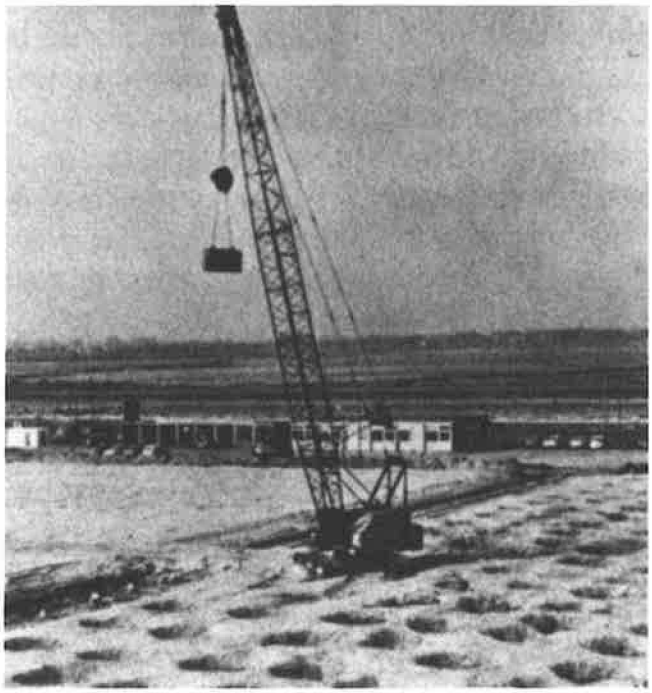


FIGURE 7 Deep dynamic compaction (*30*).

Reinforcement

Stone Columns

Stone columns are used to reinforce soft fine-grained deposits. Equipment similar to that used for vibrocompaction of cohesionless soils is used to create a cylindrical hole, which subsequently is backfilled with stone compacted by vibration. For very soft to firm soils, the hole usually is created by nondisplacement jetting methods, whereas displacement methods are more commonly used in firmer soils (*43*). The compacted stone forms a vertical column [typically 2 to 3 ft (0.6 to 0.9 m) in diameter] that is significantly stiffer and stronger than the surrounding soft soil. Stone columns function much like pile foundations. They have been used to support embankments and, in a few cases, to support bridge abutments and mechanically stabilized walls. Details on the design and construction of stone columns for highway applications are presented by Barksdale and Bachus (*48*). The performance of stone columns in the United States has been reviewed by Barksdale and Goughnour (*49*).

Lime Columns

Lime columns have been used extensively in Sweden for stabilization of soft clay embankment foundations. The lime columns,

TABLE 1
SUITABILITY OF DEPOSITS FOR DYNAMIC COMPACTION (*45*)

General Soil Type	Most Likely Fill Classification	Most Likely AASHTO Soil Type	Degree of Saturation	Suitability for Dynamic Compaction
Pervious deposits in the grain size range of boulders to sand with 0% passing the 75 μm (No. 200) sieve	Building rubble Boulders Broken concrete	A-1-a A-1-b A-3	High or low	Excellent
Pervious deposits containing not more than 35% silt	Decomposed landfills	A-1-a A-2-4 A-2-5	High	Good
			Low	Excellent
Semi-pervious soil deposits, generally silty soils containing some sand but less than 25% clay with PI < 8	Flyash Mine spoil	A-5	High	Fair
			Low	Good
Impervious soil deposits, generally clayey soils where PI > 8	Clay fill Mine spoil	A-6 A-7-5 A-7-6 A-2-6	High	Not recommended
			Low	Fair to minor improvements; water content should be less than plastic limit
Miscellaneous fill including paper, organic deposits, metal, and wood	Recent municipal landfill	None	Low	Fair; long-term settlement anticipated due to decomposition. Limit use to embankments
Highly organic deposits, peat-organic silts		None	High	Not recommended unless sufficient granular fill added and energy applied to mix granular with organic

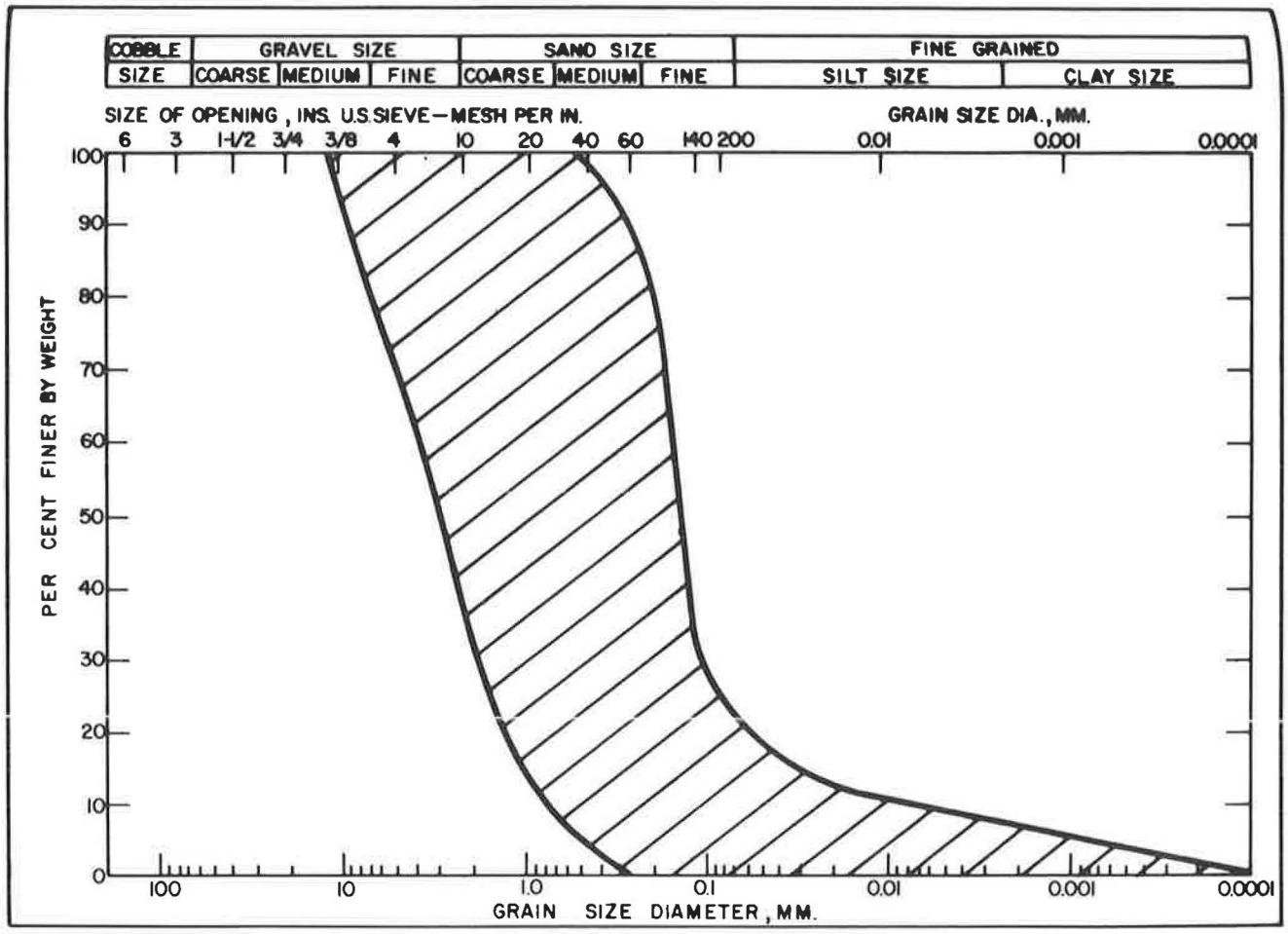


FIGURE 8 Grain size ranges suitable for vibro-densification.

which are typically 18 in. (0.5 m) in diameter and up to 50 ft (15 m) long, are constructed by in situ mixing of quicklime and clay (*50*). Special mixing equipment is required. Broms (*51*) cites a Swedish example of the use of lime columns to stabilize an approach embankment adjacent to a pile-supported abutment. Currently, the method does not appear to have been used on bridge approaches in the United States.

Embankment Piles

Embankment piles have been used in Scandinavia and Southeast Asia to transfer embankment loads through soft clay deposits. Small-diameter timber piles are most commonly used; but precast concrete piles also are used, especially when the applied loads are relatively large. The embankment load may be transferred directly to the pile by skin friction or through a pile cap. The reduction of approach embankment settlements is cited as a specific application of embankment piles (*52*). The depth of penetration is reduced with increasing distance from the abutment, thus providing a gradual transition from abutment to the roadway embankment, which is not supported on piles (see Figure 9). This technique seldom has been used in the United States, but several states currently are considering it on new projects. Design recommendations are included in Broms and Wong (*52*).

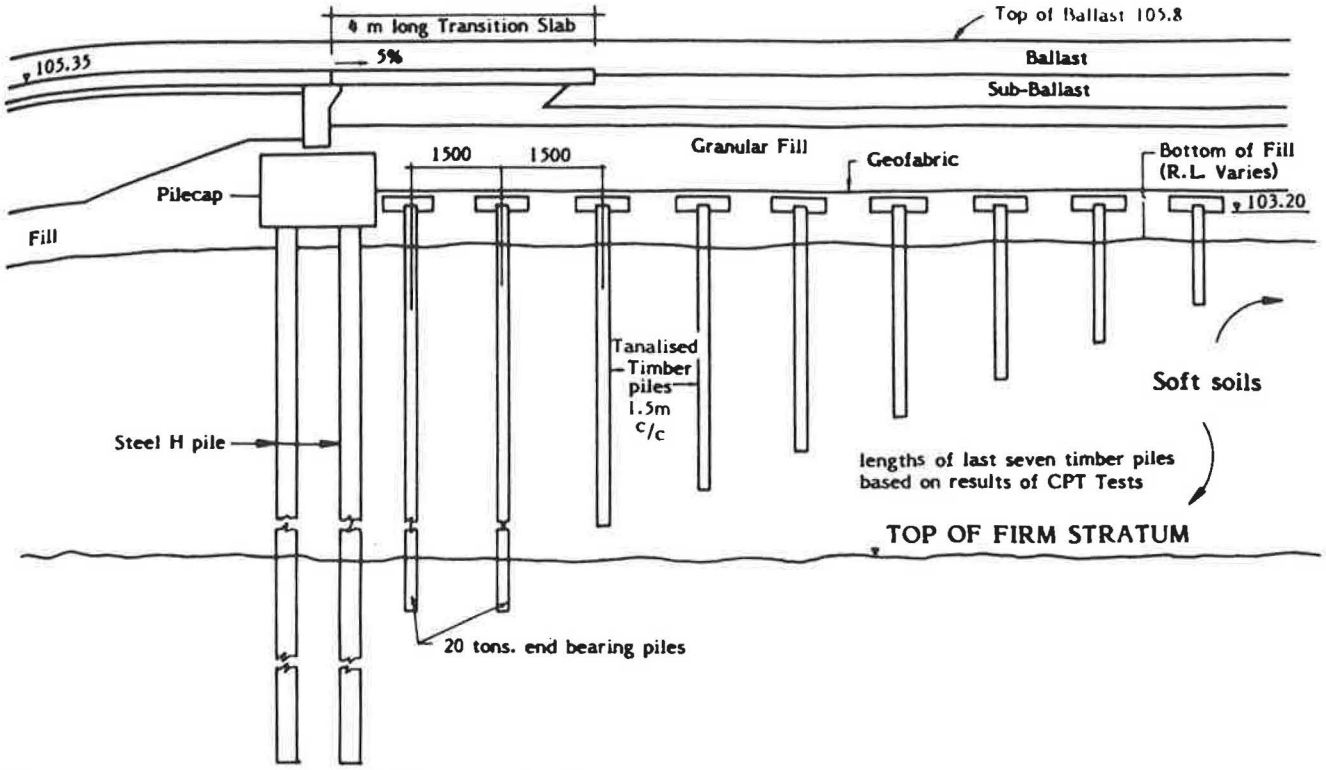


FIGURE 9 Embankment piles for bridge approach (*52*).

CHAPTER THREE

APPROACH EMBANKMENT DESIGN AND CONSTRUCTION

ROLLED EARTH EMBANKMENTS

Conventional Requirements

Most bridge-approach embankments are constructed by conventional rolled earth procedures. Materials from roadway excavation or a convenient borrow site commonly are used. However, because only small settlement can be tolerated at the abutment, it is good practice to specify select materials and increased compaction requirements, especially near the abutment. For example, California specifies fill with a maximum PI of 15 and fewer than 40 percent fines within 150 ft (45 m) of an abutment wall (Figure 10), and the required relative compaction is increased to 95 percent from 90 percent within this zone. The approach embankment typically is compacted in 6- to 24-in. (0.15- to 0.6-m) layers, depending on the type of soil and compaction equipment. The thicker lifts are used only for vibratory compaction of clean granular fills, and even for such soils thin lifts must be used adjacent to the abutment. Special material and compaction specifications for backfilling adjacent to abutments are discussed in Chapter 4.

Previous syntheses (*1, 6*) described the placement procedures and compaction requirements for construction of rolled earth embankments in the late 1960s. At that time most transportation engineers expressed general satisfaction with the quality of compacted embankments, and consequently these procedures have not changed significantly in the past 20 years. Most agencies still require 90 to 95 percent of the maximum dry density achieved in the AASHTO T 99 Compaction Test for roadway embankments and 95 to 100 percent for bridge approaches. The increased use of heavier and vibratory rollers frequently has made it possible to achieve the required compaction with fewer coverages or thicker lifts.

When fills are compacted in accordance with the current density requirements, a stable embankment with negligible compressibility usually is produced. Exceptions are noted when compressible clay fills are used, especially for embankments exceeding 40 to 50 ft (12 to 15 m) in height. For example, excessive settlement of a 22-ft- (6.7-m-) high bridge approach on a cohesionless foundation in eastern Colorado has been attributed to consolidation of the poorly graded clay fill (*3*). In the construction of high embankments, sometimes zoning of materials must be used. The higher-quality materials are placed near the bottom of the fill, where the embankment loads are the greatest. The lower-quality materials are placed in the upper parts of the embankment and capped with a zone of select material that will serve as the pavement subgrade.

When expansive soils must be used, they often are placed near the bottom of the embankment, where the overburden pressure restricts their tendency to swell. Whenever compressible cohesive materials must be used in approach embankments, especially for high fills, consolidation tests and settlement analyses should be performed to carefully evaluate the potential performance of the fill. Waiting periods may be required before the bridge abutment and superstructure can be completed.

When the approach embankment is constructed on a sloping foundation, there may be a tendency for the fill to slide down the sloping ground surface, as in Figure 11 (*14*). If the foundation soil is relatively impervious, the sliding may be accentuated by the accumulation of water along the bottom of the fill. In such cases, the foundation soil should be benched, as shown in Figure 12 (*1*). Benches should be at least 8 ft (2.4 m) wide to accommodate construction equipment. Also, granular fill and perforated pipe should be placed along the benches to prevent the accumulation of excess water in the fill.

Lightweight Fill

Lightweight fill sometimes is used to reduce the weight of an embankment over soft compressible foundation soils. The reduced embankment loading increases the stability and reduces the compression of the foundation soil. However, unless the lightweight fill also has high strength and stiffness and low compressibility, the improved foundation performance will be offset by unsatisfactory behavior of the embankment. Hartlén (*53*) lists the following requirements for a satisfactory lightweight fill:

- bulk density less than 63 pcf (1000 kg/m^3)
- high modulus of elasticity and high angle of internal friction
- good stability and resistance against crushing and chemical deterioration
- non-frost active
- noncorrosive to concrete and steel
- nonhazardous to the environment

Materials with higher bulk densities may effectively mitigate settlement and stability problems in some less severe circumstances. Availability and cost are other factors that must be considered.

A list of materials that have been used for embankments is given in Table 2, which has been modified from Hartlén (*53*). Expanded shale and industrial furnace wastes (slag, cinders, etc.) have been the most commonly used lightweight materials in the United States. However, the growing concern about the potential environmental effects of pollutants commonly found in furnace wastes is making these materials less attractive. Also, these mate-

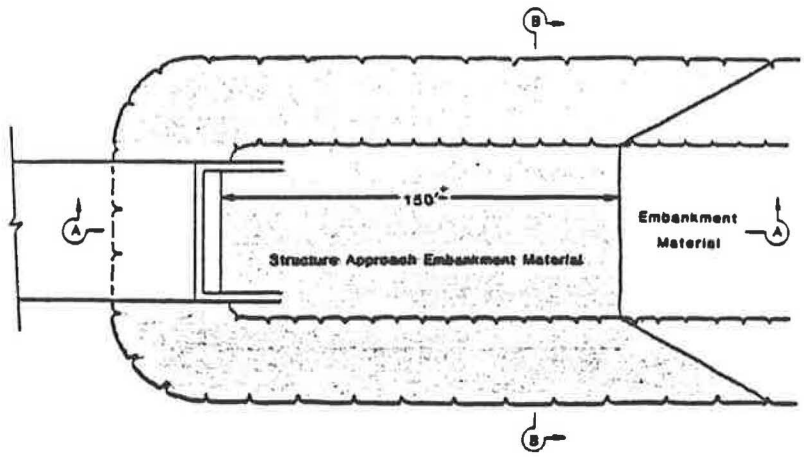


FIGURE 10 Limits of structure approach embankment material (Caltrans)

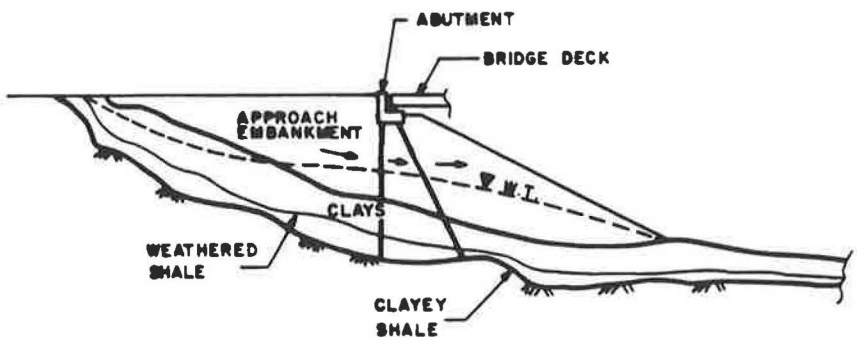


FIGURE 11 Typical cross section of side hill bridge-approach embankment where lateral movements may occur (*14*).

rials may be corrosive and thus detrimental to metallic components of drainage systems or mechanically stabilized walls and slopes. Low-density lightweight concrete is being used with greater frequency. Expanded polystyrene, which is manufactured in blocks, is very strong and light but to date rarely has been used in the United States because it is extremely expensive. This material is finding increased use in Scandinavia (*53*) and should become more common in the United States as it becomes more cost-effective.

Clamshells have been used effectively for some embankments, including bridge approaches in Louisiana (*54*). Interlocking of the shells is reported to produce high strength at compacted unit weights of 75 pcf (1200 kg/m^3).

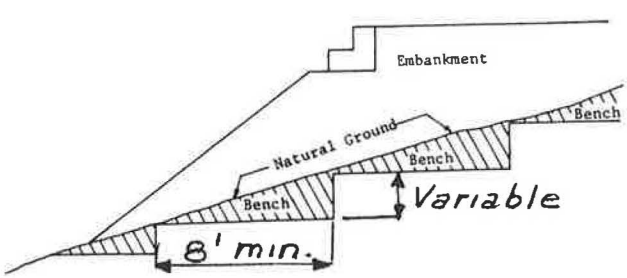


FIGURE 12 Abutment end section with natural ground benched (*1*).

Peat, bark, and sawdust have been used as lightweight fill below the water table, where they are protected from deterioration. Uncompacted sawdust was used over San Francisco Bay mud in the approach embankment for the Dumbarton Bridge (*55, 56*). This project, which also involved the use of geotextiles and wick drains, will be discussed more fully in Chapter 8.

Design Considerations

The analysis and design of conventional earth embankments involves consideration of stability and settlement. Side slopes must be selected to provide an adequate factor of safety, usually 1.25 to 1.3, along all potential sliding surfaces, including both shallow surfaces within the embankment and deep surfaces extending into the foundation soils. The stability computations are done by the classical limiting equilibrium methods, which have been described in Chapter 2.

The compression of the embankment also should be calculated by the classical methods for settlement analysis, which have been discussed in Chapter 2. The compression of the embankment then must be added to the compression of the foundation soils to obtain the predicted settlement of the embankment surface.

TABLE 2
LIGHTWEIGHT FILL MATERIALS (ADAPTED FROM *53*)

Material	Approximate Unit Weight (kN/m³)	Approximate Unit Weight (lb/ft³)	Comments
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 ground water can be reduced or eliminated by using material initially stored in water and then allowed to air dry for some months. The compacted to loose volume ratio is of 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 ground water level but has occasionally been employed for embankments that have had the side slopes sealed by asphalt or plastic sheeting.
Peat			Proved particularly useful in Ireland for repairing existing roads by replacing gravel fills with baled peat.
Air dried; 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, are generally placed at least 0.3 m (1 ft) 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 a new experimental lightweight fill material manufactured from portland cement, water, and a foaming agent with the trade names "Elastizell" and "Geocell." 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 to other techniques for constructing high-standard roads. The minimum thickness of road pavement above the expanded clay is generally in the order of 0.6 m (2 ft).
Expanded polystyrene	0.2 - 1	1.3 - 6	This is a super-lightweight material used only in the United States and Norway up to the present but where its performance has proved very satisfactory and its use is increasing. In Norway the material is used in blocks. The thickness of the pavement varies between 0.5 and 1 m (1.5 and 3 ft) 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.

For most well-compacted embankments, the settlement caused by compression of the embankment should be small. However, high embankments and most embankments composed of plastic clays may experience significant compression (*3*). The major concern will be the differential settlement between the approach embankment and the bridge abutment.

Provision for adequate drainage of the bridge-approach system is another important design consideration. Drainage provisions will be discussed in conjunction with the abutment design in Chapter 4.

End-Slope Protection

The end slope beneath a bridge structure is especially vulnerable to erosion and damage. At stream crossings, it is subjected

to erosion by flowing water. At other locations, the end slope may be damaged by drainage from the bridge and roadway. In addition, it often is difficult to maintain vegetation beneath the bridge structure. Consequently, it is good practice to provide an erosion-resistant surface for the end slope.

Materials commonly used for end-slope protection include concrete slabs and paving blocks, gravel, riprap, and heavy stone. The paving blocks and riprap may be grouted or ungrouted. In most cases, all of these materials provide satisfactory protection of end slopes. A New York study (*57*) indicates that the major cause of damage to end slopes is concentrated quantities of runoff from the structure or seepage flow emerging from the embankment. Saltwater runoff from winter salting of roadways may accelerate the deterioration of concrete end slopes. Thus it is essential that the drainage provisions for the bridge approach are designed to minimize runoff on the end slope.

TABLE 3
TYPES AND FUNCTIONS OF GEOSYNTHETICS

Type	Function
Geogrids	Reinforcement
Geotextiles	Reinforcement Separation Filtration Drainage
Geocomposites, Geonets	Drainage
Geomembranes	Isolation

MECHANICALLY STABILIZED EMBANKMENTS

The reinforcement of earth masses is one of the more important geotechnical design and construction developments of the past two decades. Metal reinforcement was used in early applications and continues to be used widely. More recently, a wide variety of synthetic polymer products, which have been given the generic name "geosynthetics," have been developed specifically for geotechnical applications. Table 3 lists the major types of geosynthetics and the functions for which these products have been developed.

The typical uses of reinforcement in embankments are illustrated in Figure 13. In Figure 13a, a single layer of reinforcement is shown at the bottom of the fill. In this case, a sheet of geotextile fabric typically is used to separate the fill from the soft foundation soils while allowing free drainage across the interface. The tensile resistance of the fabric also increases the safety against a deep slide through the fill and the foundation soil. If stability against a deep slide is the primary function of the reinforcement, then geogrids or steel mesh also could be used.

In Figure 13b, horizontal reinforcing is shown near the slopes. This reinforcement provides tensile resistance and increases the stability of the slope, thus permitting the use of steeper design slopes. The cost of the reinforcement is offset by the reduction in earthwork and right-of-way requirements. The latter factor is important in urban areas, especially for widening and improvement projects for existing bridge approaches. A variety of reinforcing systems are available, many of which are proprietary. In general, the following options are available:

- Metal or plastic strips
- Metal or plastic rectangular grids
- Continuous sheets of geotextile fabrics

Strip and sheet reinforcement rely on friction for load transfer between the reinforcing element and the soil. Well-graded granular backfill is required to assure adequate frictional resistance. For grid reinforcement systems, load transfer is developed by passive soil pressure against the transverse elements of the grid and by friction between the soil and the horizontal surfaces of the grid.

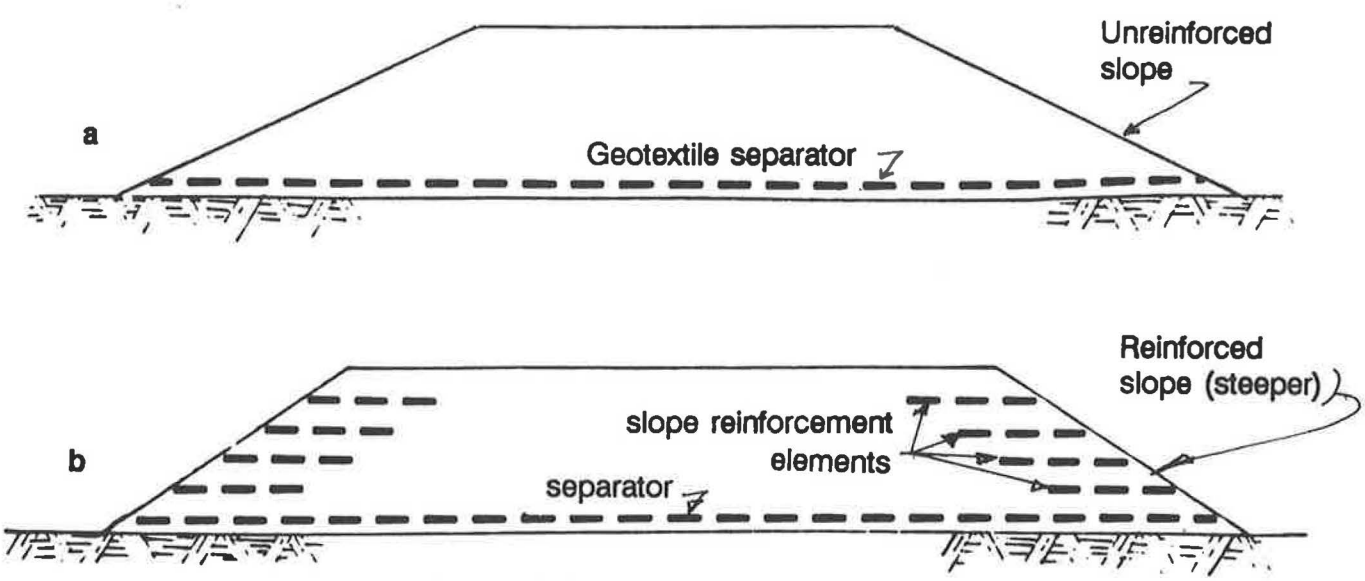


FIGURE 13 Schematic diagram of reinforced embankments.

The development of tensile resistance in the reinforcing elements requires tensile deformation of the element and consequently is related to the modulus of deformation of the reinforcing material. Steel and polymer geogrids are stiff relative to the soil and are capable of mobilizing tensile resistance at relatively small strains. On the other hand, geotextile fabrics are very ductile and require large deformations for mobilization of tensile resistance. However, the ductility of the geotextile provides safety against catastrophic failure, which is more likely to occur with overstressing of more brittle materials. Both types of reinforcement commonly are used by many transportation agencies.

In 1982 a process was developed in France to mix continuous tensile-resistant geotextile threads into a sandy soil to form a composite material, similar in concept to fiber-reinforced concrete. This fiber-reinforced soil has exhibited high bearing strength and resistance to erosion. The technique has been used in practice in France, and test embankments have been constructed in California. However, fiber-reinforced soil still must be regarded as an experimental material. The complex mixing process must be improved before it will be economical and reliable for use in general practice.

Facing elements may be attached to the outer ends of the reinforcement to produce a mechanically stabilized wall. Many types of facing elements are available, including precast concrete panels, prefabricated metal plates, welded wire mesh, and shotcrete. Examples of several wall systems are shown in Figure 14.

Mechanically stabilized walls offer many potential advantages over conventional slopes and reinforced concrete retaining walls. They are particularly effective in areas of limited right of way or difficult access. Because the facing is supported by the internal reinforcement, only a minimal footing is required. The soil-wall system is more flexible than a conventional reinforced concrete wall and thus can tolerate larger differential deformations. However, abrupt differences in settlement along the wall can damage facing elements and connections. The facing can be selected to provide aesthetic architectural effects. Construction procedures are simple and rapid and can be performed within a limited working space. The combination of these factors often makes the mechanically stabilized wall the most economical alternative. Mitchell and Villet (*58*) suggest that the cost-effectiveness of the mechanically stabilized wall increases with wall height. Christopher et al. (*59*) suggest that the use of mechanically stabilized walls instead of conventional reinforced concrete walls may produce savings of 25 to 50 percent for wall heights greater than 10 to 15 ft (3 to 4.5 m). Even greater savings may be realized if the use of the mechanically stabilized wall eliminates the need for a deep foundation. Some comparative costs for various wall types are shown in Figure 15.

Design Considerations

The design of reinforced embankments and mechanically stabilized walls must consider both internal and external stability. Internal stability considers the development of the load transfer between the soil and the reinforcing elements and the determination of the required size and spacing of the reinforcement. For walls, the pullout capacity of the reinforcing elements must be established. Well-graded clean granular backfill must be specified to ensure effective internal load transfer. The current FHWA specifications for mechanically stabilized embankments require granular fill with a maximum PI of 6 and a maximum of 15 percent finer than the 75 μm (No. 200) sieve size. The maximum allowable particle size is 4 in. (100 mm) for metallic reinforcement and $^3/_4$ in. (19 mm) for geosynthetic elements (*59*).

The evaluation of external stability is conceptually similar to the analyses of conventional slopes and retaining walls. Limiting equilibrium conditions are assumed and various potential modes of failure are examined. For reinforced slopes, the factors of safety of potential sliding surfaces are analyzed, and the resistance of any reinforcing element that crosses a potential failure surface is included in the analysis. For walls, the factors of safety against lateral sliding, overturning, and foundation bearing failure are considered. Also, a general slope failure that does not intersect the reinforced zone must be considered.

Almost all reinforcement systems are proprietary, and specific design assumptions differ with each system. The details of the analysis and design of various types of reinforced embankments and walls are presented in an excellent NCHRP Report (*58*), which includes many design examples. Additional design guidelines are provided in the 1988 FHWA report (*59*), which includes the results of full-scale tests of eight mechanically stabilized walls and four embankment slopes. Potential users of earth-reinforcing systems are referred to these informative reports.

The analysis and design of a mechanically stabilized bridge approach also must consider the potential settlement of the embankment. The methods for predicting settlement are essentially the same as for conventional unreinforced embankments. Reinforcing systems have very little effect on settlement unless the reinforcement allows a substantial reduction of the overall weight of the embankment-abutment system.

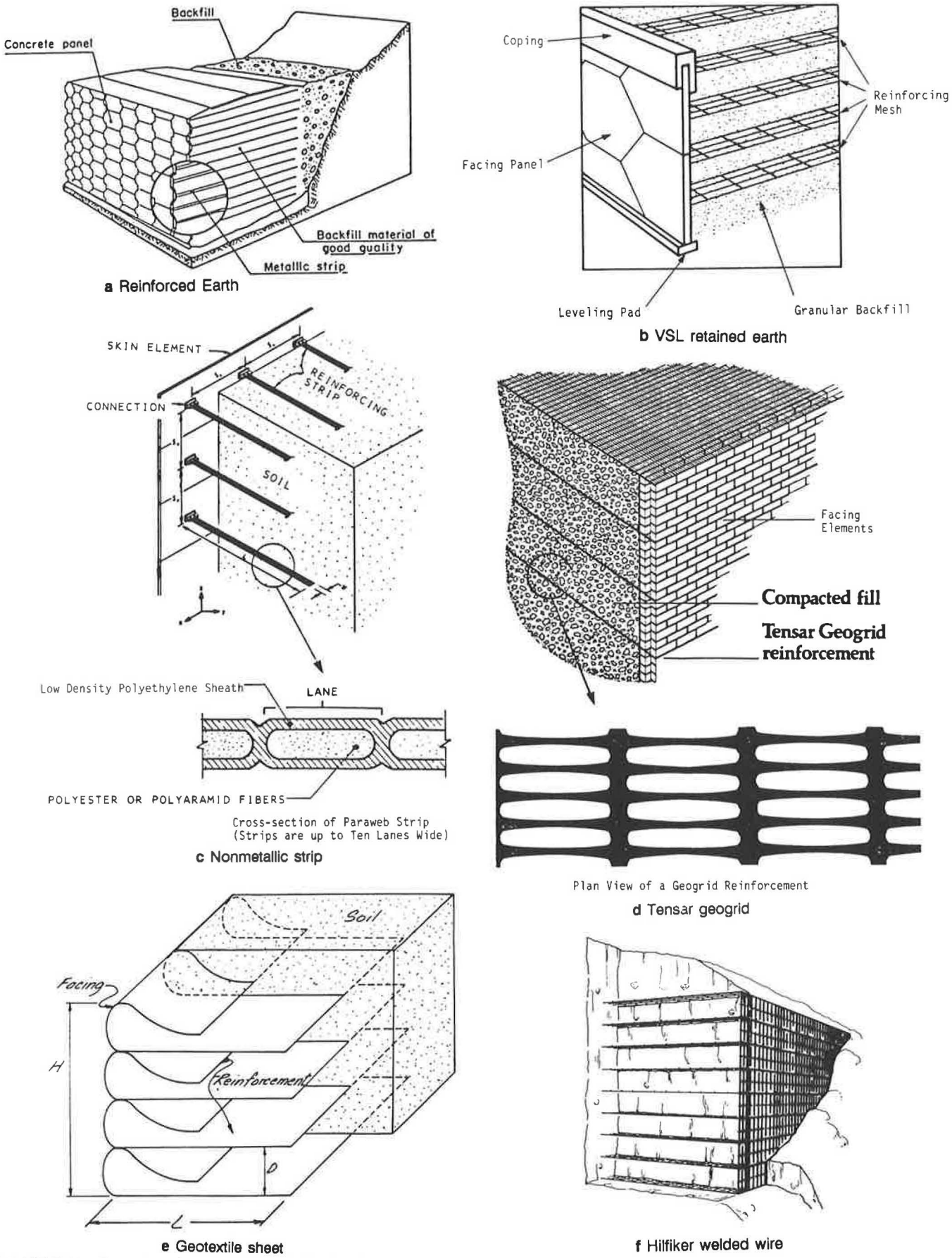
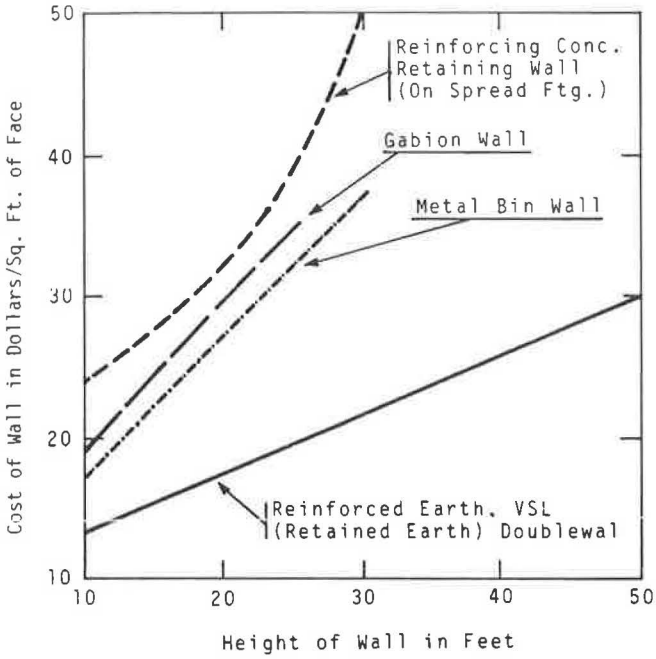


FIGURE 14 Examples of mechanically stabilized walls.



NOTES

(1) Costs shown are for wall materials and erection.

(2) Backfill and structural excavation costs are not included, except for Gabion and metal bin walls, where the costs include the cost of backfill placed inside the Gabion baskets and metal bins.

(3) For each individual project, backfill, structure and architectural treatment costs should be added to costs from the chart to make an overall cost comparison of wall types.

(4) Cost variations between RE, VSL Reinforced Earth, and Doublewal do not appear sufficient to justify separate cost curves for estimating purpose.

(5) Costs shown are intended for preliminary estimating and cost comparison purpose only.

(6) Costs shown are based on a combination of recent bid experience in FHWA Region 10.

(7) Chart was compiled by Ron Chassie, FHWA Region 10 Geotechnical Engineer, Portland, OR.

FIGURE 15 Cost comparison of six wall types, 1981 Caltrans data (*58*).

CHAPTER FOUR

ABUTMENT DESIGN AND CONSTRUCTION

Bridge abutments support the dead and live loads from the bridge superstructure and also are subjected to lateral earth pressures from the approach embankment. Several types of abutments commonly are used, and the design loads depend on the type of abutment used and the sequence of construction.

ABUTMENT TYPES

Conventional Abutments

Conventional bridge abutments provide support for the superstructure through bearings with an expansion joint that allows relative movement between the abutment and the deck. The expansion joint accommodates thermal strains in the deck and potential lateral movements of the abutment. Closed, stub, and pedestal abutments have been used in most states, and these structures have not changed significantly in recent years.

Closed Abutment

A closed abutment is a full-height wall with wing walls on each side that retains the full height of the approach embankment (Figure 16). This type of abutment minimizes the required span length of the bridge. However, there are several significant disadvantages, especially for high embankments and embankments on soft foundations. The abutment must be constructed before the adjacent embankment. Proper placement and compaction of backfill is difficult in the restricted area between the abutment and the wing walls. If heavy equipment is used to compact the backfill, the wall may be pushed laterally and out of vertical alignment. The weight of the backfill will contribute to the compression of soft foundation soils and the post-construction settlement of the embankment and abutment. If the abutment is supported on piles, the compression of the foundation soil will produce downdrag, which may overstress the piles.

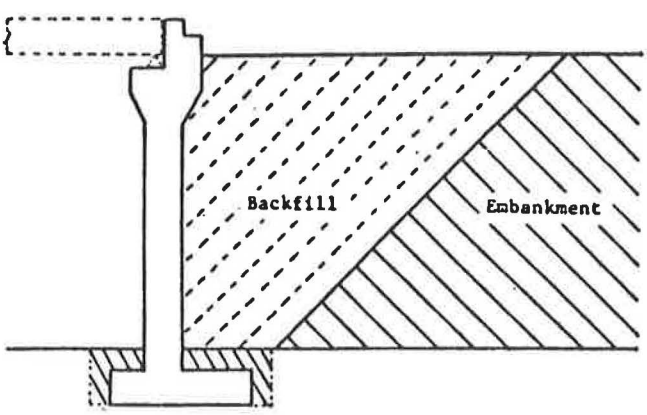


FIGURE 16 Typical full-height closed abutment.

Closed abutments must be designed for the lateral earth pressures exerted by the full height of the approach embankment. Rankine earth pressure theory or equivalent fluid pressures conventionally are used. Because the lateral force and the overturning moment about the base of the wall are proportional to the square and the cube of the wall height, respectively, the structural requirements and cost of the wall increase rapidly with height.

Stub or Perched Abutment

A stub or perched abutment is a relatively short abutment that is constructed after the embankment has been completed. The embankment can be compacted without interference from the abutment, and, if necessary, the abutment construction can be delayed until the compression of the foundation soils caused by embankment loads is completed. Thus, post-construction settlements may be minimized. The abutment may be supported on spread footings in the embankment or on piles or drilled shafts constructed through the fill, as shown in Figure 17. Because the stub abutment is relatively short and located in the upper part of the fill, the lateral earth pressure is relatively small in comparison to the pressure against a full-height closed abutment. However, longer bridge spans may be required than for a full-height abutment.

Stub abutments also are used on natural ground in cut slopes. Spread footing, pile, or drilled shaft foundations may be used.

Pedestal or Spill-through Abutment

Pedestal or spill-through abutments are short stub-type abutments supported on pedestals or columns extending to natural ground (Figure 18). As with closed abutments, the abutment must be constructed before the approach embankment. Then the fill is placed simultaneously on both sides of the pedestals to minimize the unbalanced earth pressure and the lateral movement of the abutment. However, the fill around the pedestals is very difficult to compact properly. It also is very susceptible to erosion unless the end slope is protected with paving, riprap, etc.

The lateral earth pressures against a spill-through abutment are less than for a full-height closed abutment but greater than

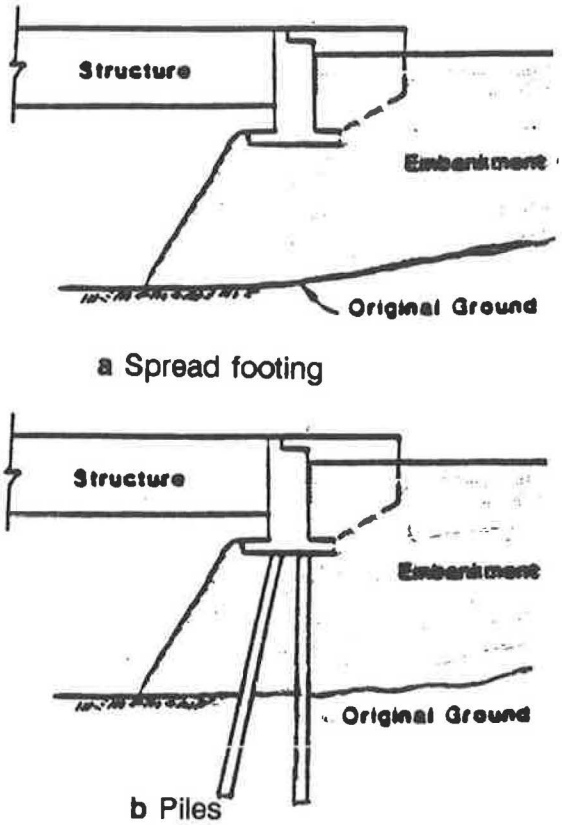


FIGURE 17 Typical perched abutment.

for a stub abutment. The weight of the fill that is placed after construction of the abutment may cause compression of soft foundation soils and the same detrimental post-construction effects that have been described for closed abutments.

Integral End Bents

Integral end bents are stub abutments that are rigidly connected to the superstructure without expansion joints. The connection is designed for full transfer of thermal stresses from the bridge deck to the abutment. The elimination of expansion joints with movable bearing systems for the deck is reported to reduce construction and maintenance costs. Various designs are used by different state agencies, but almost all require steel H-piles for support of integral abutments. A typical integral end bent is shown in Figure 19.

Integral abutments have been used in the United States for about 30 years. Early applications in Kansas, Missouri, Ohio, and Tennessee were restricted to relatively short bridges, generally less than 100 ft (30 m) long (*60*). Today integral abutments are used in more than half of the states, and the allowable lengths have increased significantly. Greimann et al. (*60*) cite the successful use of integral abutments for continuous-steel and concrete bridges with lengths of 300 ft (90 m) and 500 to 600 ft (150 to 180 m), respectively. Almost all states that use integral abutments on piles indicate that they are performing well (*2*).

Despite the apparent increasing popularity of integral abutments, some problems have been noted. These problems generally are related to cyclic lateral movements of the abutment in response to thermal stresses in the bridge deck. Cracking and buckling of approach pavements have been reported (*61, 62*). The approach pavement design must include careful consideration of the thermal effects from integral abutments. Approach slabs are discussed more fully in Chapter 6.

Because most backfill materials are not perfectly elastic, cyclic abutment movements create voids between the backfill and the abutment, which permit erosion of the backfill and progressive deterioration and settlement of the backfill. Groundwater or surface water that enters the backfill through pavement cracks accelerates the erosion process. Some states use special details to accommodate the cyclic abutment movements. For example, one North Dakota design, which is illustrated in Figure 20, provided a pressure relief system between the backfill and the abutment. Narrow vertical strips of 4-in.- (100-mm-) thick compressible material were attached at 4-ft (1.2-m) intervals along the back of the abutment. Corrugated metal sheets were placed behind the compressible strips to maintain a 4-in. (100-mm) void between the abutment and the backfill. Also, to facilitate lateral movement of the H-piles, 2-in.- (50-mm-) thick compressible material was glued to both sides of the upper 20 ft (6 m) of the

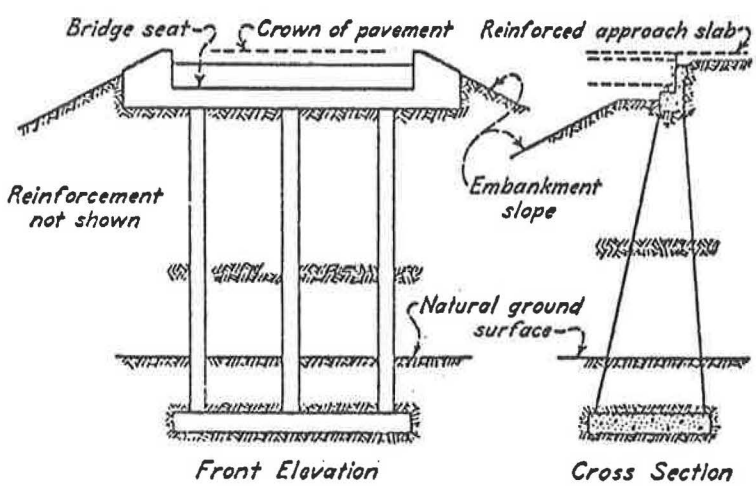


FIGURE 18 Typical spill-through abutment.

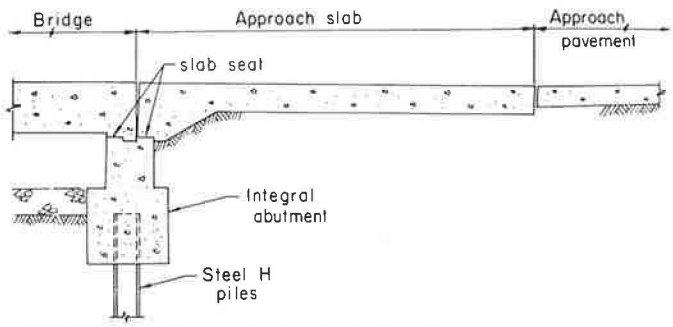


FIGURE 19 Typical integral abutment.

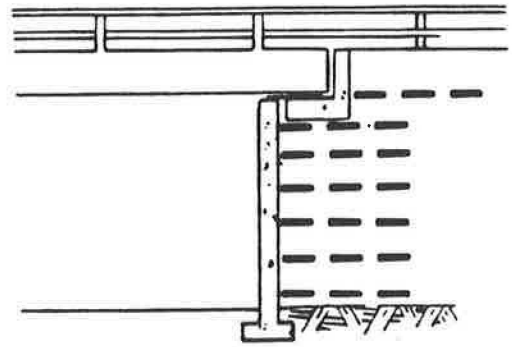


FIGURE 21 Schematic diagram of mechanically stabilized abutment.

web. An oversize hole was drilled to a depth of 20 ft (6 m), and the pile was placed in this hole and driven to a depth of approximately 110 ft (33 m). The oversize hole was backfilled with sand. The performance of this system is reported by Jorgenson (*63*).

The performance of integral abutments could be improved by the development of new compressible elastic materials that could be installed easily between the abutment and the backfill. The ideal material would have elastic properties that permit large recoverable cyclic movements and hydraulic properties that provide adequate drainage without erosion of fines from the backfill. Also, almost all agencies require steel H-piles for support of integral abutments, on the assumption that this facilitates lateral movement. Currently, there is no evidence to support this hypothesis. Significant savings could be realized if it were demonstrated that integral abutments on spread footings will perform as well as those supported on piles.

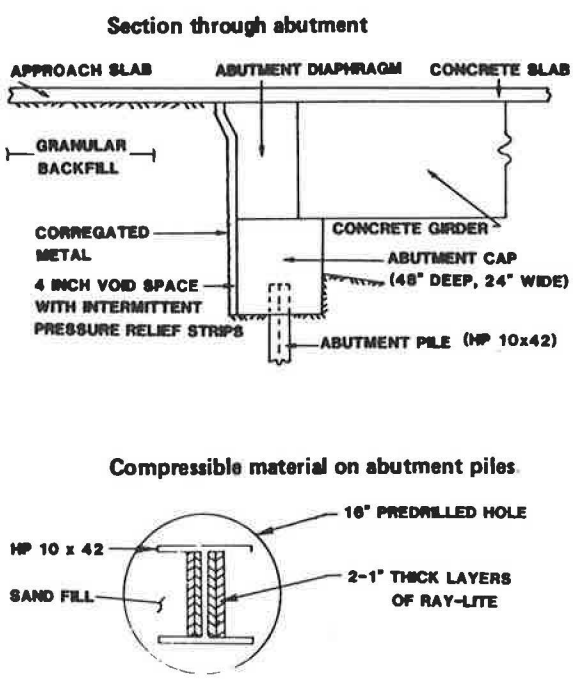


FIGURE 20 Special integral abutment design, North Dakota (*63*).

Mechanically Stabilized Abutments

Mechanically stabilized walls, which have been described in Chapter 3, also have been used to support bridge abutments. The bridge seat and footing are supported directly on the reinforced backfill, as shown schematically in Figure 21. The design concepts for the mechanically stabilized walls are applicable. The abutment loads increase the reinforcement requirements, especially in the upper part of the backfill, where the bridge loads are concentrated. However, the facing elements are not affected because the facing does not support structural loads. There are many proprietary systems that are based on different design assumptions, but generic design procedures are recommended by Mitchell and Villet (*58*) and Christopher et al. (*59*).

Because mechanically stabilized abutments can tolerate large deformations, they often are an economical alternative to deep foundations or treatment of soft foundation soils. For example, in 1972 mechanically stabilized abutments were used for a bridge across the Moselle River at Thionville, France (Figure 22) because large approach fill settlements were anticipated. The abutment was surcharged until 12 in. (0.3 m) of settlement occurred and then the superstructure was constructed. Six months after completion of the superstructure, the bridge seat was jacked 2.4 in. (60 mm) to compensate for additional settlement. Since the bridge was opened to traffic, less than 0.5 in. (12 mm) of settle-

FIGURE 22 Reinforced Earth abutments, Moselle River bridge, Thionville, France (*64*). (Courtesy The Reinforced Earth Co.)

ment has been observed (*64*). Because the abutment and approach fill settle as one unit, differential settlement between the bridge deck and the approach roadway is virtually eliminated.

The speed and ease of construction have contributed to widespread use of mechanically stabilized abutments in the United States. They are especially useful for new and rehabilitation projects in urban areas where right-of-way and work area often are restricted. For example, they have been used extensively in the Atlanta area (Figure 23). Also, tied-back abutments have been used for widening and rehabilitation projects in cuts. An example of a tied-back abutment in Virginia is given in Chapter 8.

ABUTMENT FOUNDATIONS

Bridge abutments may be supported on spread footings, piles, or drilled shafts. The selection of a safe and economical foundation requires consideration of structural loads, environmental factors, subsurface conditions, performance criteria, construction methods, and economics. The steps in a systematic foundation selection process are outlined in Figure 24, which is reproduced from Synthesis 107 (*8*). In each case the foundation must be designed to provide adequate safety with respect to bearing capacity and to limit horizontal and vertical abutment movements to tolerable levels. Both the approach fill and structural loads must be considered.

Spread Footings

Spread footings generally are the most economical foundation. However, many state agencies traditionally have been reluctant to use spread footings because of perceived uncertainties in performance predictions. Also, the potential for scour remains a major concern for bridges over waterways. In recent years, it has been demonstrated that spread footings that provide satisfactory performance can be designed and constructed at significant savings in comparison with the costs of deep foundations (*8, 65*). Spread footings have been used to support abutments on a wide variety of subsurface materials, ranging from sands and gravels to clay and rock. Commonly, the footing is constructed after the embankment fill has been placed to minimize settlement of the abutment caused by the weight of the fill.

FIGURE 23 Reinforced Earth abutments, Atlanta, Georgia. (Courtesy The Reinforced Earth Co.)

Sands and Gravels

The design of footings founded on granular soils generally is controlled by tolerable settlement criteria rather than by the bearing capacity of the soil. Thus it is essential to be able to make reliable settlement predictions. A recent FHWA study (*17*) lists more than 20 methods for predicting the settlement of footings on sand and describes 15 of them in some detail. Settlement predictions by the methods of Burland and Burbridge (*66*), D'Appolonia et al. (*67, 68*), Hough (*20*), Peck and Bazaraa (*69*), and Schmertmann (*18*) were compared with observed settlements of 21 instrumented footings for 10 bridges. It was concluded that the average difference between predicted and observed settlement was about 0.4 in. (10 mm) by each method, but larger errors occurred on individual cases by each method. The D'Appolonia method was judged to be the most accurate, but the differences among the average results for the five methods were very small. The report suggests that the reliability of the settlement prediction can be improved by using the average of predictions by two or more of the five methods.

The scour potential must be evaluated at waterways. This topic was reviewed in 1970 in Synthesis 5 (*70*). As a result of recent failures (e.g., the Schoharie Creek Thruway Bridge in New York), additional research on scour at bridges currently is a high priority. New interim guidelines for estimating scour at bridges and for designing bridges to resist scour recently have been prepared by the FHWA (*71*).

Cohesive Soils

Both bearing capacity and settlement criteria must be evaluated for footings supported on clay. The bearing capacity is determined from classical theory (*72*), using the in situ undrained strength of the clay before construction. This provides a conservative design estimate except when the footing is placed in a deep permanent excavation. Many laboratory and field procedures may be used to evaluate the undrained strength (*26*). Most settlement predictions are based on relatively conventional procedures, which in turn are based on one-dimensional compression concepts and the results of laboratory consolidation tests. These procedures may be found in all soil mechanics textbooks and are summarized in TRB Special Report 163 (*13*) and Synthesis 107 (*8*). Recently, new procedures have been developed for use with various in situ tests [e.g., the dilatometer or pressuremeter (*73*)].

When shallow footings are to be constructed over soft clay, it generally is necessary to construct the approach embankment and wait for completion of consolidation caused by this load before starting construction of the footing and the abutment. Frequently it is advantageous to construct a stub abutment over a pad of granular fill, as shown in Figure 25 (*74*).

Rock

Spread footings on rock generally are designed on the basis of presumptive allowable bearing capacities, which are based on

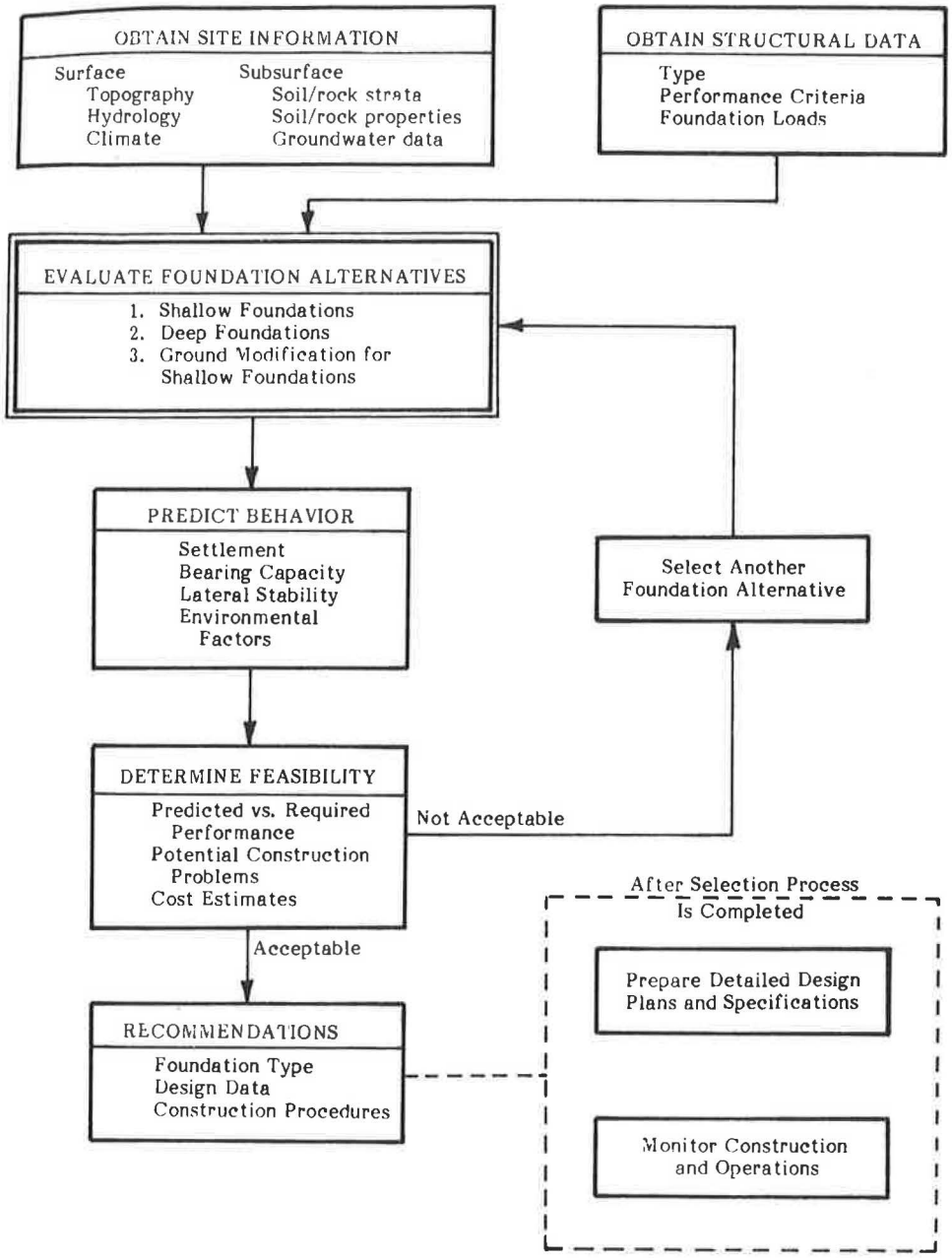
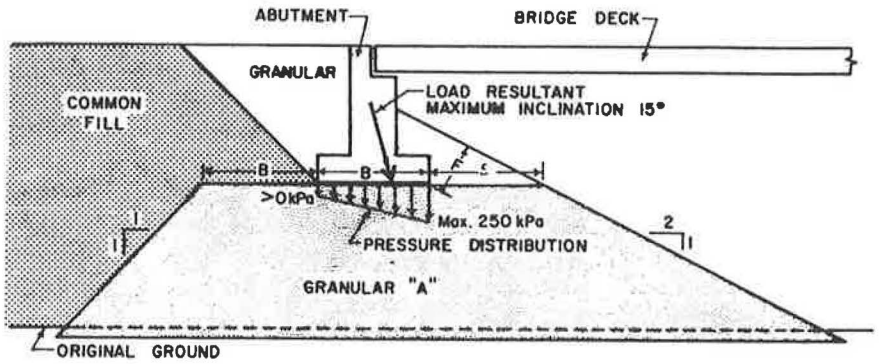


FIGURE 24 Foundation selection process (*8*).

rock type and the frequency of discontinuities in the rock mass. The rock quality designation (RQD), which was developed by Deere (*75*), commonly is used to quantify the rock quality from the frequency of fractures in core samples. Typical presumptive bearing capacities usually are very conservative but are satisfactory for most bridge foundations because the design loads are low (*17*). Furthermore, it is generally assumed that settlements of 0.5 in. (12 mm) or less should be anticipated when presumptive allowable pressures are used (*17, 76*). It is recommended that a layer of compacted gravel or crushed stone be placed between an abutment footing and the rock surface to allow a slight rotation of the abutment, which will reduce the lateral earth pressure on the abutment (*17*).

Although the preceding design procedures usually are satisfactory, there are special conditions that may adversely affect the performance of a spread footing on rock. Highly weathered rock and solution features, which are common in limestone, produce very irregular rock properties. Swelling or heave in rock excavations are common in shales. The orientation of discontinuities may be especially critical to the stability of a footing on rock slopes.

Because of these potential problems, an adequate field investigation is very important. For most bridges, core drilling that extends about 10 ft (3 m) into unweathered rock should be sufficient unless major defects are encountered. For large bridges with high bearing pressures, core drilling should be conducted



s depends on minimum frost depth F. Note: 1 kPa = 6.89 lbf/in^2.

FIGURE 25 Stub abutment on granular mat (*74*).

to a depth of twice the minimum footing dimension below the anticipated footing elevation (*17*).

Deep Foundations

When the predicted performance of spread footings appears to be unsatisfactory, deep foundations are used to transfer abutment loads through weak compressible materials to strata that provide adequate support. Both driven piles and drilled shafts are used. The design of the deep foundation must consider loads from the approach fill as well as from the structure. Also, the interaction between the approach fill and the abutment must be evaluated very carefully. Because the abutment foundation is likely to be significantly stiffer than the embankment foundation, differential settlement is likely to develop between the approach embankment and the abutment. The construction sequence, which is discussed in Chapter 5, will be an important factor in the evaluation of the predicted performance of a deep foundation. Generally it will be advantageous to construct the approach fill before the deep foundation is installed.

Piles

Piles are the most common deep foundation for bridge abutments. Piles are designed to be driven to adequate bearing strata, but occasionally piles are installed in drilled holes to overcome special problems. For example, abutment piles may be placed in holes drilled in an embankment to minimize interaction with the embankment fill; they are then driven into the foundation soils. Concrete and steel piles are used most frequently, but timber piles sometimes are used for small bridges. Concrete piles include both precast concrete and cast-in-place concrete, which is placed in a steel shell that is driven with or without the aid of a mandrel.

The bearing capacity of a pile is derived from end bearing at the pile tip, skin friction along the surface of the pile, or both. The end bearing dominates when the pile is driven to refusal on the surface of rock or hard soil. Skin friction often provides a very significant component of support in granular soils and weathered rock. However, soft cohesive soils, which may be compressing under the weight of the approach fill, may cause a downdrag force on the pile that actually decreases the design capacity of the pile.

The bearing capacity of a pile may be estimated by several methods, including classical bearing capacity theory, wave equation analyses, field load tests, and pile-driving formulas. An excellent review of pile design procedures based on theoretical bearing capacity and pile load tests is presented in Synthesis 42 (*77*). However, many designers continue to rely solely on pile-driving formulas, even though the unreliability of such formulas is well established (*78*). There does appear to be increasing use of the wave equation for analysis, design, and construction control (*79*). The computer program WEAP (Wave Equation Analysis of Piles), which was developed by Goble and Rausche for the FHWA (*80*), now is used by many transportation agencies. Also, the development of the dynamic pile analyzer, a device that measures the force and acceleration of the top of the pile under each hammer blow, has enhanced the use of wave equation analyses for construction control (*78, 81*).

The bearing capacity is required at three stages in the design and installation of pile foundations. Preliminary estimates are required at the design stage. Then the design estimate must be verified in the field, and finally a measure of bearing capacity is used for construction quality control.

Today good practice uses classical static bearing capacity theory or dynamic wave equation analyses for preliminary design estimates of pile capacity. For very large projects, field load tests may be conducted during the design stage. For small projects with light loads, pile-driving formulas may be adequate for both design and construction control. However, for larger projects and moderate to heavy loads, the estimated capacity should be verified during construction by a full-scale load test or the dynamic pile analyzer. Construction control then may be based on the driving record of the load test. For complex site conditions, the dynamic pile analyzer may be desirable for construction control.

The preceding procedures determine the axial capacity of a pile. Abutment piles also must be designed for the lateral loads from the approach fill. Frequently batter piles are used under abutments, and the horizontal component of the axial capacity is used to provide lateral support. For vertical piles, the lateral resistance must be determined by elastic methods, which also are used for drilled shafts (*82*).

Drilled Shafts

Drilled shafts are constructed by boring a cylindrical hole and filling it with fluid concrete. The minimum shaft diameter is 18

in. (0.5 m), and shafts up to 12 ft (3.6 m) in diameter can be constructed with regular drilling equipment. Also, equipment is available to underream or enlarge the bottom of the shaft to form a bell with a larger area for end bearing. Temporary or permanent steel casing or drilling slurries may be required to prevent caving of weak and cohesionless soils, especially below the water table. It is preferable to place concrete in a dry hole, but concrete also can be placed below water or a slurry with a tremie or a concrete pump. The shaft may be reinforced as necessary.

Drilled shafts have been used commonly for building foundations for many years, but recently there have been increasing applications to bridge foundations. A single large-diameter shaft generally replaces a cluster of piles, thus eliminating the need for a pile cap. Because the drilled shaft typically has greater lateral stiffness than a pile group, it is particularly effective for abutment foundations. Drilled shafts are applicable to a wide range of soil conditions, but they are especially well suited for medium stiff clays in which a shaft can be excavated easily without temporary casing.

Most drilled shafts are designed for support by end bearing, but skin friction along the shaft also should be considered. The skin friction is most important in cohesionless soils and for shafts socketed in sound rock. When skin friction is considered, the axial load capacity usually can be developed more economically by lengthening the shaft than by constructing an enlarged bell at the base. The lateral capacity of a drilled shaft is determined by elastic methods that use the load-deflection characteristics of the subsurface materials. Design procedures for determining end-bearing capacity, skin friction, settlement, and the lateral load-displacement behavior of drilled shafts in cohesive and cohesionless soils and rock have been presented by Reese and his colleagues in a series of excellent FHWA manuals (*82–84*). The most recent manual (*85*) presents the current state of the art with clear design examples.

PERFORMANCE CRITERIA

Tolerable Movements

All abutments and their foundations are likely to move. As a part of the abutment design process, the predicted movements must be compared with tolerance limits that are established to maintain the safety and function of the bridge. Significant horizontal and vertical abutment movements may seriously affect the safety and the rideability of the bridge. Also, it is necessary to consider the relative settlement of the abutment and the approach fill.

A comprehensive study of bridge movements was reported by Moulton (*86*) in 1986. Measurable movements were reported for 439, or approximately 75 percent, of the 580 abutments included in the study. The majority (357) of the observed movements were for perched abutments, but full-height and spill-through abutments also were included. Vertical movements were reported for 379 abutments and horizontal movements at 138 locations. Significant movements were reported for abutments supported on piles as well as for abutments supported on spread footings.

The Moulton report also included an assessment of which movements were regarded as tolerable and which were intolerable. The tolerability of the movement was judged qualitatively by the agency responsible for each bridge in accord with the following definition: "Movement is not tolerable if damage requires costly maintenance and/or repairs *and* a more expensive construction to avoid this would have been preferable."

Using this criterion, vertical movements that exceeded 4 in. (100 mm) and horizontal movements that exceeded 2 in. (50 mm) usually were regarded as intolerable. More distress was observed for concrete bridges than for steel bridges. Multi-span bridges were affected more often and more severely than single-span structures.

The tolerable differential settlement generally increases with span length. Moulton (*86*) suggests a tolerable angular distortion (differential settlement/span length) of 1/250 (.004) for continuous-span bridges and 1/200 (.005) for simply supported spans. These values appear reasonably consistent with criteria commonly used for tolerable settlement of buildings.

The preceding criteria do not address the differential settlement between the bridge deck and the approach pavement. A differential settlement of ½ in. (12 mm) between these components is likely to produce a "bump at the end of the bridge" that will require maintenance. However, in accord with the previous definition, such movement should not be considered intolerable unless the cost of maintenance is greater than the cost of the design modifications necessary to reduce the differential settlement.

Reliability of Movement Predictions

Comparisons of predicted movements with tolerable movement criteria must consider the reliability of prediction methods. The reliability depends on many factors, including uncertainties in the applied loads, the stress distribution within the foundation soils, and the characterization of the subsurface profile and the relevant soil parameters. For most structures, and especially for bridge abutments, the properties of the subsurface soils will be the major source of uncertainty. The degree of uncertainty will depend on the type and variability of the foundation soils and on the amount and quality of sampling and testing that is conducted.

The reliability of conventional settlement prediction methods can be estimated by reviewing case histories that include both predicted and observed settlements. One recent study (*87*) suggests that settlement predictions for shallow foundations over normally and lightly overconsolidated clays are likely to be within ± 25 percent of observed values. For dense sands and heavily overconsolidated clays, for which the observed settlement is less than 1 in. (25 mm), the percent error of the prediction may be larger but the absolute error is likely to be less than ¼ in. (6 mm). Similar results are reported in a recent FHWA study of bridge footings on sand (*17*). Using five different prediction methods, two-thirds of the predicted values were within ± ¼ in. (6 mm) of the measured settlement, and more than 80 percent were within ± ⅜ in. (9 mm). For other soils, such as loose sands, sensitive clays, residual soils, expansive and collapsible soils, and sand-clay mixtures, settlement predictions are likely to be much less reliable and errors may approach 100 percent. The study also suggests that most conventional settlement procedures tend to be conservative because loads typically are overestimated and conservative values of soil properties usually are selected.

The reliability of settlement predictions also can be studied by formulating the settlement calculations in a probabilistic format. The concepts of probabilistic error analysis are presented by

Baecher (*88*), and an example of the application of probabilistic methods for calculating the settlement of footings on sand is included in the recent FHWA study of shallow foundations (*17*). The example considers only the uncertainty in the soil parameter.

Predictions of lateral movements use the finite-element method and require elastic parameters for the soils. This method is demonstrated by Siriwardane et al. (*29*) for two perched abutments on piles. For one case the predicted horizontal movement of the abutment at the beam seat was 1 in. (25 mm) and the observed movement was 2 in. (50 mm), both toward the approach fill. For the second case the predicted movement was more than 3 in. (75 mm) toward the fill, whereas the observed movement was 2 in. (50 mm) away from the fill. These results are not surprising because of the sensitivity of finite-element analyses to the values selected for elastic parameters and other assumptions required in the model. Consequently, the current state of practice does not provide reliable predictions of lateral movements.

OTHER PROVISIONS

Backfilling Specifications

The structural backfill behind and beneath abutments requires special materials and placement procedures. The fill must be well compacted to prevent post-construction settlement of the approach fill and pavement system. The density requirements of various state agencies range from 95 percent of AASHTO T 99 maximum density to 100 percent of the AASHTO T 180 maximum density. Usually the compaction near the abutment and wing walls must be accomplished with small vibratory rollers or hand-operated vibratory plates or pneumatic tampers to prevent damage or movement of the abutment. Both stability and drainage should be considered in the selection of backfill. The use of well-graded pervious backfill with less than 5 percent finer than the 75 μm (No. 200) sieve is ideal and is strongly recommended, when available. Such materials can be compacted easily with small vibratory compactors, whereas cohesive backfills may require excessive compactive efforts to achieve the required compacted density. The pervious fill also will minimize the possibility of post-construction compression of the backfill, facilitate drainage of the backfill, and eliminate frost heave problems.

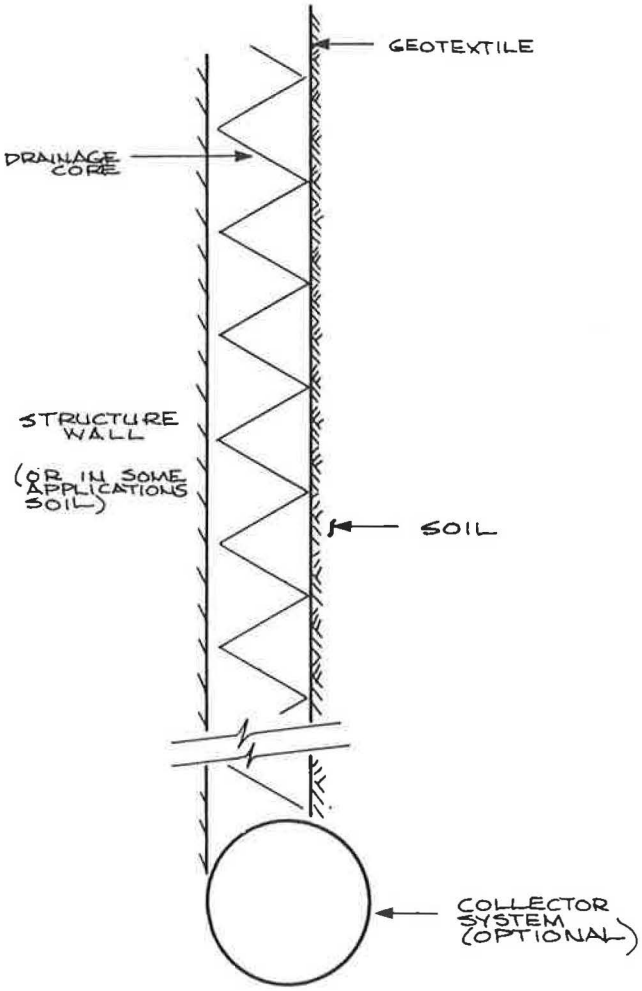
The backfill should be placed and compacted in lifts of 6 to 8 in. (150 to 200 mm). If an end slope is being constructed outside the abutment, the elevations of the interior and exterior fill should be kept approximately equal. Some agencies specify that the backfill elevation should not be raised above the elevation of the beam seat before the bridge deck girders have been constructed and the deck has been poured.

Drainage Provisions

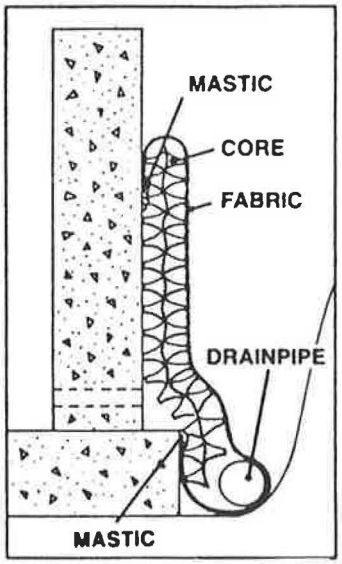
Provisions must be made to prevent the development of hydrostatic water pressure behind the abutment. Furthermore, the drainage must be handled so as not to encourage erosion of the backfill and the end slope. Surface drains, such as gutters or paved ditches, should be provided to drain surface water away from the abutment and backfill. The internal drainage provisions will depend on the type of backfill and foundation soil. If both the backfill and foundation soil are pervious, no additional drainage may be necessary. If pervious backfill is underlain by impervious material, then 4-to-6-in.- (100-to-150-mm-) diameter perforated horizontal drains are required at the bottom of the pervious fill to carry the water to the sides of the approach fill or to weepholes in the abutment. Depending on the gradation of the backfill, it also may be necessary to use a geotextile filter around the perforated pipe to prevent erosion of backfill material and clogging of the drain. Similar provisions may be necessary to drain groundwater that may enter a pervious backfill from a cut slope.

Special provisions must be made for drainage from behind the abutment. Many new prefabricated drainage systems, generically called geocomposite drains, are being developed for this purpose. The geocomposite drain consists of a geotextile fabric and a semirigid plastic core. Frequently the geocomposite is connected to a collector pipe that conducts the water to points of final discharge. A schematic drawing of a typical system is shown in Figure 26. The geotextile fabric separates the open drainage core from the backfill and serves as a hydraulic filter between the backfill and the open core. The solid portion of the drainage core supports the geotextile and maintains an open volume for free movement of water by gravity. The prefabricated geocomposite drain is manufactured in sheets that are easily installed against abutment walls.

Two important characteristics of a composite drain are the hydraulic properties of the geotextile and the compressive strength of the core. The geotextile must serve as a filter that allows entry of water while restraining the passage of soil solids. Clogging of the geotextile clearly will decrease the effectiveness of the system. The compressive strength of the core is required to prevent reduction of the open flow space in the core caused by the lateral soil pressure against the geotextile. The basic design concepts for geocomposite drains, a summary of available systems, and suggested tests for evaluating the critical properties of these systems are reviewed in a recent study sponsored by the FHWA (*89*).



a Components



b Typical installation

FIGURE 26 Geocomposite drains.

CHAPTER FIVE

SPECIAL CONSTRUCTION CONSIDERATIONS

ABUTMENT/EMBANKMENT CONSTRUCTION SEQUENCE

The safety and function of a bridge is affected by the post-construction movements of abutments and approach fills. Movements that occur before construction of the bridge superstructure do not affect performance or maintenance costs. Frequently the construction sequence and time schedule can be arranged to minimize the post-construction movements. This is particularly important for construction over soft compressible cohesive soils.

It is good practice to construct the approach fill and to wait for consolidation of the foundation soils before constructing the abutment and its foundation. Usually no waiting period will be required when the approach fill is underlain by rock, dense sands and silts, or hard clays. For soft cohesive soils, as noted in Chapter 2, temporary surcharges or prefabricated wick drains or both may be used to accelerate the consolidation process and thus reduce the required waiting period. It has been clearly demonstrated that these procedures significantly reduce the post-construction settlement of abutments (*86, 90*). For example, Moulton (*86*) reports an average settlement of 1.8 in. (45 mm) for 81 perched abutments constructed on spread footings with preloads or waiting periods compared with an average of 7.3 in. (185 mm) for 60 cases without preloads or waiting periods. The waiting periods cited by Moulton range from one to six months.

It is desirable to construct the full approach embankment, wait for consolidation, and then excavate the fill as necessary to construct the abutment and its foundation. Only minimal excavation will be required for perched abutments, but significant excavation and backfilling may be necessary for full-height or spill-through abutments. It is important that the abutment foundation be constructed after the waiting period. If a spread footing is used, the post-construction settlement of the abutment and the approach fill should be relatively uniform. If a deep foundation is used, some differential settlement should be anticipated. However, the downdrag forces on the deep foundation will be minimized. Also, there should be less post-construction lateral movement of the foundation soils and thus less lateral load on the deep foundation. A typical cross section and construction sequence for a perched abutment on piles is shown in Figure 27.

Backfilling should be completed at least to the elevation of the bridge seat before the superstructure is constructed. Some agencies require that the bridge girders and deck be installed before backfill is placed above the bridge seat.

STAGE CONSTRUCTION OF EMBANKMENTS

Design Factors

When approach fills are constructed over very soft foundation soils, the full embankment load may overstress the foundation soil and cause a slide. In such instances, the embankment may be constructed in stages with waiting periods between each stage

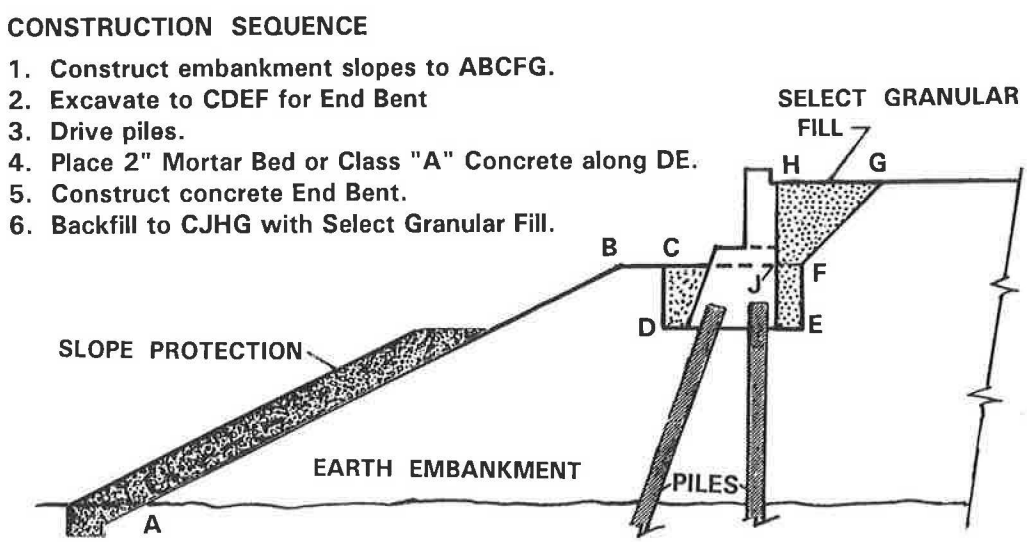


FIGURE 27 Example of recommended sequence for embankment/abutment construction (adapted from *14*).

to take advantage of the gain in shear strength that accompanies consolidation. Extensive field and laboratory investigations are necessary. Three types of analyses are required. First, stability analyses are conducted to determine the maximum allowable embankment height as a function of the undrained shear strength of the foundation soil. Second, the rate of consolidation must be estimated for each stage of embankment loading. Finally, the increase in undrained shear strength as a function of consolidation must be evaluated. The SHANSEP analysis (*27*) may be useful for prediction of the in situ undrained strength. An excellent discussion of all design aspects of stage construction is included in the 1986 Terzaghi Lecture by Ladd (*91*). An example of stage construction is presented in Chapter 8.

Field Monitoring

Field monitoring is essential for construction control when waiting periods are used with or without stage construction. Theoretical design computations usually do not provide reliable estimates of the rate of consolidation. For small projects, settlement plates for observation of vertical movements may be all that is needed. For larger and more complex projects, piezometers may be installed to monitor the buildup and dissipation of pore water pressures at several elevations within the compressible layer. Also, inclinometers may be used to measure lateral movements as a function of depth. Excellent descriptions of instrumentation and procedures for geotechnical monitoring are found in Synthesis 89 (*42*).

PROVISIONS FOR POST-CONSTRUCTION SETTLEMENT

Waiting periods should eliminate most post-construction settlement caused by primary consolidation. However, compressible cohesive soils also exhibit long-term secondary compression, which generally increases linearly with the logarithm of time. For example, if primary consolidation is completed in 110 days (0.3 yr) and 1.0 in. (25 mm) of secondary compression occurs in 3 years, then the total secondary compression will reach 2.0 in. (50 mm) after 30 years and 3.0 in. (75 mm) after 300 years.

For many cohesive soils, the secondary compression effects are small and can be neglected; however, there are examples for which secondary compression is significant (*14*). In these cases, surcharge loads and longer waiting periods may be used to eliminate part of the secondary compression during the waiting period (*33*). This is illustrated by one of the case histories in Chapter 8. Alternatively, it may be more economical to plan to use periodic maintenance for the gradual settlement caused by secondary compression.

Short-term post-construction settlement of the approach fill will be more noticeable when the abutment is supported on deep foundations. Some agencies use temporary bituminous approach pavements to eliminate the bump at the abutment when significant post-construction settlement of the fill is anticipated. The permanent approach pavement is installed when the rate of settlement decreases to a tolerable level.

Some agencies design abutments with jacking pads when large long-term post-construction settlements are anticipated. When settlement reaches the tolerance limits of the superstructure, hydraulic jacks are used to raise and shim the bridge girders. Jacking usually can be completed in a few days at a cost of several thousand dollars. Spread footings with jackable abutments have been shown to be an economical alternative to deep foundations (*65*).

The effects of post-construction settlements also may be reduced with various types of approach slabs, which are discussed in Chapter 6.

CHAPTER SIX

APPROACH SLABS

TYPES

Approach slabs are used to provide a smooth transition between the bridge deck and the roadway pavement. The slab usually is designed to withstand some embankment settlement and a reduction of subgrade support near the abutment. Joints must be provided to accommodate cyclic thermal movements of the bridge deck, abutment, and roadway pavement.

Most state transportation agencies currently use reinforced concrete approach slabs for some applications (*2*). Some agencies use approach slabs that are tied to the abutment, and others provide an expansion joint between the bridge deck and the approach slab. Most of these agencies believe that rigid approach slabs are successful. However, some feel that they merely move the bump at the end of the bridge to the approach slab-roadway interface, and others report unsatisfactory performance. Many agencies use bituminous approach pavements, which can be maintained easily with overlays, but some are reluctant to use bituminous approaches with portland cement concrete roadways.

Most reinforced concrete approach slabs are 20 to 40 ft (6 to 12 m) long. California employs a combination of a 30-ft (9-m) approach slab and a sleeper slab with a minimum length of 15 ft (4.5 m). Significantly longer slabs are used by a few agencies (*2*). For example, Illinois uses a 100-ft (30-m) continuously reinforced slab with a thickened section for the first 20 ft from the abutment. The top of the slab is constructed 1.5 in. (42 mm) below finish grade and brought to grade with a 1.5 in. (42 mm) bituminous surface. When excessive approach embankment settlements are anticipated, Louisiana uses 80- to 120-ft. (24- to 36-m) continuous approach slabs, which are pile supported. Some typical examples of approach slabs are shown in Figure 28 (*2, 62*).

Most approach slabs are of uniform thickness, but some sections are tapered or haunched to provide more flexural resistance near the abutment. The slab width typically is the same as the bridge deck. The amount of reinforcement varies with the design loads and slab length. Most slabs are reinforced at top and bottom and designed as one-way slabs without intermediate support.

All approach slabs are supported at one end on the bridge abutment. The remainder of the slab should be underlain by a permeable base and the approach embankment. The roadway end of the approach slab may be supported directly on the subgrade or on a sleeper slab, which provides an additional transition to the roadway pavement, as shown in Figure 28. The sleeper slab is a hidden slab that underlies and supports the ends of the approach slab and the adjacent roadway pavement and thus minimizes the possibility of differential settlement at the approach slab-roadway interface. Pile-supported approach slabs were reported only in Louisiana (*2*). Continuous slabs are supported at 10-ft (3-m) intervals by timber friction piles, which are driven 10 ft into natural ground and which decrease in length with increasing distance from the abutment (Figure 29). In some instances, a one-way slab is designed between transverse grade beams at the pile rows, and sometimes a two-way slab is designed between small caps at each pile location. Illinois abandoned the use of pile-supported approach slabs in the early 1970s because of erratic performance.

A permeable base or longitudinal edge drains or both are recommended to ensure adequate drainage of the approach slab support system. Several new geotextile products may be applicable.

INTERFACE JOINTS

Details of the joints at the ends of approach slabs are extremely important to satisfactory performance of the slab. The joints must transfer traffic loads and be able to accommodate thermal expansion and contraction of the pavement system. In addition, the joints must be sealed to prevent entry of water and debris. A good discussion of approach slab joints, with special emphasis on integral end bents, is presented by Burke (*62*). Burke stresses the need for cycle-control (expansion) joints that may endure cyclic expansion and contraction without deterioration. Special care must be taken with approach slabs for integral bridges without joints because the cyclic thermal effects are transferred from the bridge deck to the approach slab. Burke also demonstrates that very large compressive stresses and buckling may develop when pavement movements are fully restrained.

Some typical joints at integral and nonintegral abutments are shown in Figure 30. In most cases, the joint permits relative movement of the approach slab and bridge deck. However, in a few examples, the approach slab is tied to the abutment so that no relative movement may occur. For such cases, a cycle-control joint must be provided at the roadway end of the approach slab. Where movement is permitted, the joints are filled with various materials, including elastomers, asphalt cement, and roofing felt, with varying degrees of success. All agencies appear to recognize the importance of maintaining watertight joints. However, no universally satisfactory joint filler is yet available.

The type of joint required at the approach slab-roadway interface depends on the type of abutment and the joint design at the abutment. Several of the approach slab examples in Figure 28 show cycle-control (expansion) joints at the roadway end of the approach slab. Several additional examples of roadway-approach

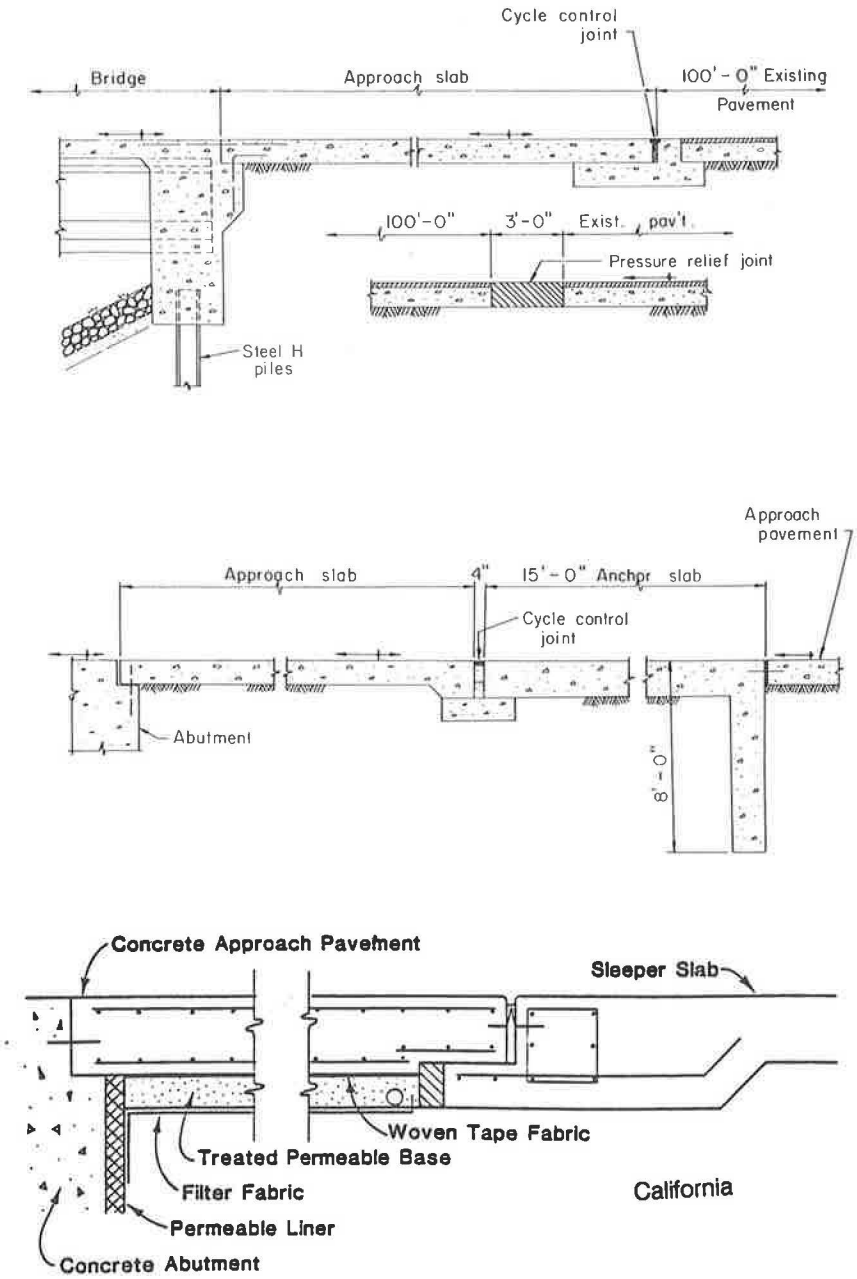


FIGURE 28 Examples of approach slabs (*2, 62*).

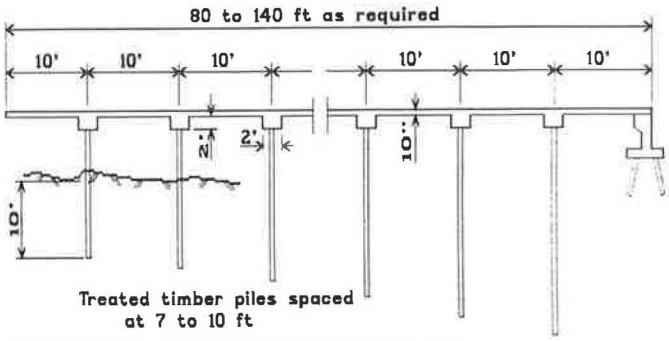


FIGURE 29 Pile-supported approach slab.

slab joints are shown in Figure 31. One of the approach slab examples in Figure 28 uses a 30-ft (1-m) bituminous pressure-relief section in the roadway pavement to reduce the compressive stresses that may be transferred to the approach slab and abutment because of thermal pavement growth. A pressure-relief joint with a sleeper slab, as used in Pennsylvania with concrete pavements, is shown in Figure 32. This design is used when there is an expansion joint at the abutment.

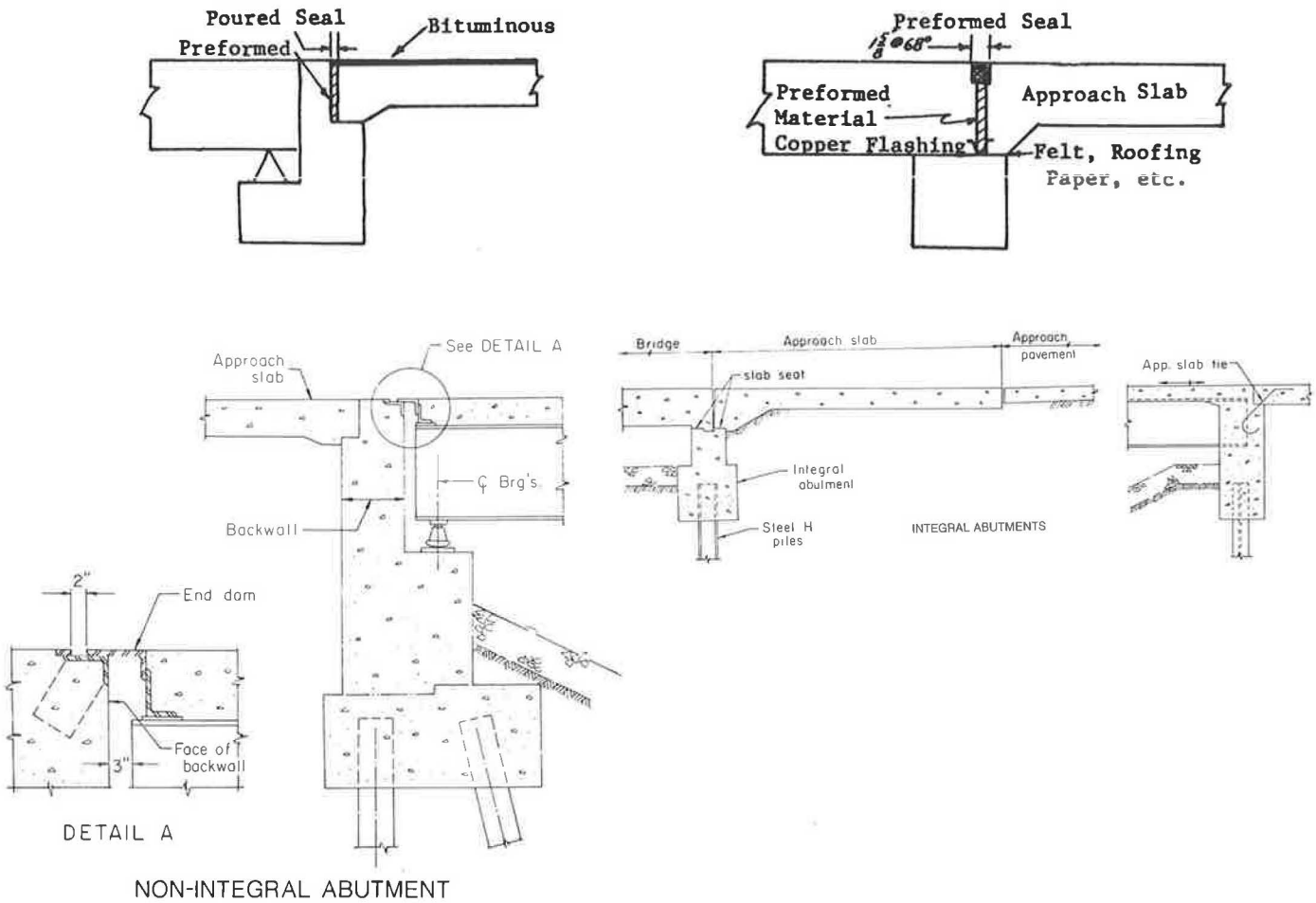


FIGURE 30 Approach slab/abutment joints (*62*).

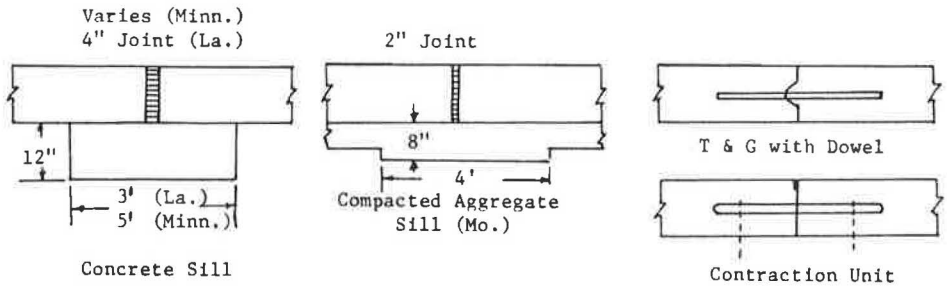


FIGURE 31 Approach slab/roadway joints (*1, 62*).

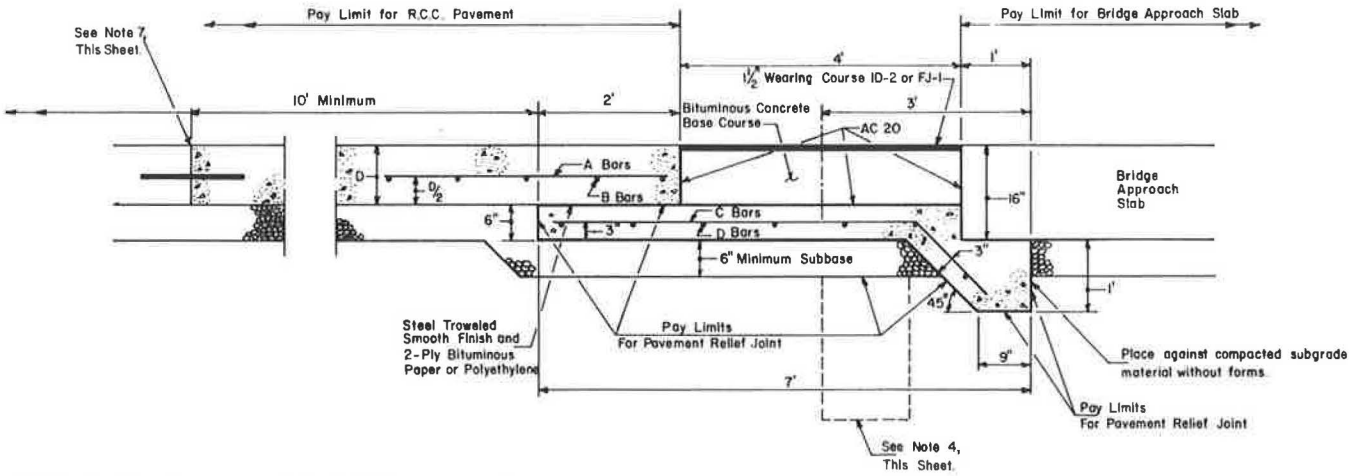


FIGURE 32 Pressure-relief joint (Pennsylvania).

CHAPTER SEVEN

MAINTENANCE AND REHABILITATION

The role of maintenance often is overlooked in the design of bridge-approach systems. Potential maintenance costs, as well as initial construction costs, should be considered in the selection of the most cost-effective bridge and approach fill designs. However, factors other than cost also must be recognized. For example, the impact of maintenance operations on traffic should be assessed. Maintenance that requires temporary closing of one or more traffic lanes will inconvenience the public and may be unacceptable in congested urban areas, especially during periods of high traffic volume.

Rehabilitation implies major renovations to restore or upgrade the safety or capacity of a transportation facility. The bridge replacement programs that are under way in most states require careful consideration of the approach embankment requirements.

APPROACH PAVEMENTS

Relatively small settlement of the approach embankment may require periodic minor maintenance, especially when bituminous approaches are used instead of rigid approach slabs. For bituminous approaches, minor leveling at the abutment usually can be accomplished with little difficulty. More widespread irregular settlement may require machine-placed overlays or leveling with a heater-planer, which may require closing one or more traffic lanes for several days.

Minor settlement of reinforced concrete approach slabs may be repaired with bituminous or epoxy overlays. Most agencies also use slab-jacking to level concrete approach slabs. Slurry or grout may be pumped through holes drilled in the pavement or horizontal pipes inserted beneath the pavement. The lifting of the slab probably can be controlled more easily with holes through the pavement, but this technique will interrupt traffic for longer periods of time. Care must be taken to avoid blowouts at joints, cracks, or the edges of the slab. Also, there is a danger that the slurry or grout may clog a permeable base or otherwise disrupt the subdrainage system for the slab. Consequently, drainage provisions must be reviewed carefully when slab-jacking is considered.

More serious problems arise if failures occur in the base, subbase, or subgrade of either bituminous or portland cement approach pavements. Such problems may be caused by design errors, improper construction practices, or unanticipated traffic loads. Frequently, base or subgrade failures are the result of the accumulation of excess moisture in the pavement system caused by inadequate provision for drainage. Sometimes the unsatisfactory conditions can be alleviated by the installation of additional subdrains or edge drains, but often complete removal and replacement of the base and pavement will be required, together with provisions for additional drainage. These major repairs will interrupt traffic for long periods and clearly are undesirable. Usually these conditions can be avoided by proper design and construction practices during the original construction of the bridge-approach system.

EMBANKMENTS

Unsatisfactory embankment performance usually is limited to surficial erosion of the slopes or unanticipated post-construction settlement. Surficial erosion can be corrected by providing additional surface or internal drainage to keep water off the slopes. Also, vegetation or riprap can be used to make the slopes more resistant to erosion.

Excessive unanticipated settlement is a much more serious problem that may be very difficult to correct. A detailed investigation would be needed to determine whether the settlement is caused by improper compaction of the embankment or by continuing vertical and horizontal displacement of the foundation soils. Depending on the cause, several in situ soil-reinforcement systems could be considered. For example, reticulated root pile systems have been used to stabilize embankments and foundations. The embankment could be reconstructed with lightweight fill or mechanically stabilized walls to reduce the weight of the fill acting on the foundation soils. If a feasible embankment modification can be found, it most certainly will be very expensive and require major interruptions to the use of the entire facility. This again emphasizes the importance of thorough initial investigations and analyses for bridge-approach projects.

ABUTMENTS

Small movements of abutments should be expected but should not affect the performance of the bridge and approach system. However, as noted in Chapter 4, vertical movements of 4 in. (100 mm) and horizontal movements of 2 in. (50 mm) generally will affect performance and be unacceptable. Horizontal movements are particularly critical because they may cause damage to bearings and jamming of beams or girders against the abutment. When excessive abutment movements occur, major maintenance usually is required.

If an abutment settles excessively, it may be possible to jack and shim the superstructure to its original position. This is relatively simple if the abutment was constructed with jacking pads; otherwise it will be difficult and expensive. Usually it is not

feasible to raise the entire abutment. Modification of the abutment foundation may be required to prevent additional settlement.

If the abutment is pushed outward by the approach fill, the abutment cannot be returned to its original position unless the backfill behind the wall is removed. Remedial work generally is directed to preventing further movement. Lateral soil or rock anchors can be installed to provide additional lateral resistance that will restrain further outward movement. In South Dakota, a number of old abutments on piles are being rehabilitated with tie-backs anchored to drilled shafts, which serve as deadmen. If water in the backfill is producing hydrostatic pressure against the abutment, additional drainage should be provided, such as horizontal drains, weepholes, or geocomposite drains behind the abutment wall.

Large abutment movements toward the approach fill usually are caused by consolidation or lateral movements of the foundation soils beneath the embankment and abutment. Corrections of this condition usually will involve expensive modifications of the abutment foundation.

CHAPTER EIGHT

CASE HISTORIES

Three cases have been selected to illustrate some of the innovative technological developments that have been applied to the design and construction of bridge approaches. All three projects replace or upgrade existing facilities. Two cases are bridge-approach construction along new alignments; the first is unaffected by the existing roadway, whereas the second is severely constrained by existing facilities. The third case involves a new grade separation to upgrade an existing intersection without altering the current alignment.

DUMBARTON BRIDGE, CALIFORNIA

Approach embankments for the new 1.6-mile (2.6-km) Dumbarton Bridge over South San Francisco Bay were constructed between 1979 and 1982. The new high-level bridge is a replacement for a 50-year-old low-level lift span, which was located approximately 90 ft (27 m) south of the new bridge. The new approach embankments traverse salt ponds, which were up to 15 ft (4.6 m) deep on the west side but only 2.5 ft (0.8 m) deep on the east side. The east embankment is 2.4 miles (3.9 km) long and the west embankment is 1.6 miles (2.6 km) long. The entire area is underlain by 10 to 40 ft (3 to 12 m) of very soft San Francisco Bay mud.

Originally, it was planned to construct the required 10-to-12-foot- (3-to-3.7-m-) high embankments by conventional methods and to allow four years for consolidation settlement before placing the pavement. However, it was determined that mud waves and shear failures would occur at an embankment height of approximately 8 ft (2.4 m). Also, it was estimated that up to 6 ft (1.8 m) of consolidation settlement would occur over a period of 50 years (*55, 56*). Consequently, an innovative design was developed using several new construction technologies.

The new design used a combination of geosynthetics, lightweight fill, and prefabricated wick drains. A typical embankment cross section is shown in Figure 33. A low earth-containment dike was constructed over a layer of reinforcing fabric to the north of the proposed embankment alignment. Then the area between the dike and the existing roadway for the old bridge was dewatered, and reinforcing fabric was placed on top of the bay mud in the dewatered area. Uncompacted sawdust fill was placed over the fabric to reduce the embankment loading in areas of the main embankment where stability of the bay mud would be marginal with conventional fill. The sawdust was used only below elevation +5 ft (1.5 m) so that it will be saturated permanently after consolidation of the bay mud. A permeable drainage layer, encased in filter fabric at top and bottom, was placed over the sawdust fill and covered with a thin layer of earth fill. Then prefabricated wick drains were installed through the bay mud, and the compacted earth fill was constructed at a rate not exceeding 1 ft/week (0.3 m/week) to elevation +10 ft (3 m) and 1 ft/2 weeks (0.3m/2 weeks) above elevation +10 ft (3 m). Waiting periods for consolidation of the bay muds were determined by monitoring settlement and pore pressures in the bay mud. Two to 7 ft (0.3 to 2.1 m) of consolidation settlement was measured in approximately 1 year, compared with the 50-year estimate of 6 ft (1.8 m) without the wick drains.

Details of the design and construction features and the results of field measurements for this project are reported in the literature (*55, 56, 92*). The California Department of Transportation estimates that these features saved more than $5 million on this project compared with conventional earthwork construction on bay muds. In addition, the construction time was shortened significantly, and long-term maintenance cost should be reduced.

RAINIER AVENUE EMBANKMENT, SEATTLE, WASHINGTON

A reconstruction project in downtown Seattle, Washington, will upgrade State Route 90, a major east-west freeway, to Interstate standards. Three bridges are to be constructed over Rainier Avenue as a part of this project. A plan view of the site is shown in Figure 34. The current case describes only the construction of the approach embankments for the westbound bridge (*93*).

The design requirements and construction constraints for the westbound embankment were very severe. The design elevation of the roadway requires an embankment height of up to 36 ft (11 m). In order to use abutments on spread footings, the design criteria limited the embankment settlement to less than ½ in. (12 mm) in the abutment area for the 40-year design life. Outside the abutment areas, the 40-year settlement was limited to less than 2 in. (50 mm). The design also minimized the length of the pile-supported bridge section in order to minimize overall project costs. This required the approach embankment to be constructed very close to existing roadways, which remained open during construction. Furthermore, there was concern that the new embankment would create new downdrag forces on the existing piles that support the nearby elevated Lakeways Freeway.

The site is underlain by two compressible lacustrine deposits. An organic deposit was 0 to 12 ft (0 to 3.7 m) thick, and a soft clay was 0 to 40 ft (0 to 12 m) thick. Analyses predicted as much as 2 ft (0.6 m) of settlement caused by primary consolidation of these layers and a significant additional amount from secondary compression. Consequently, preload and surcharging criteria were developed that would complete the primary consolidation and most of the secondary compression before construction of the abutments. A maximum temporary fill height of 57 ft (17 m)

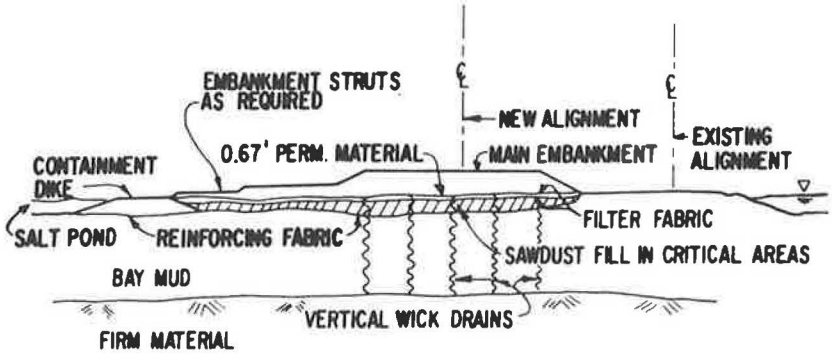


FIGURE 33 Cross section showing special construction features used at Dumbarton (*55*).

was required to satisfy the design criteria for post-construction settlement.

Prefabricated wick drains were installed in areas where the clay layer was more than 15 ft (4.5 m) thick to accelerate primary consolidation. Approximately 165,000 lin ft (50,000 m) of wick drains were installed at a spacing of 8 ft (2.4 m) in a triangular pattern. The drain system was designed to provide an estimated time of 75 days for completion of primary consolidation. The actual time required to complete primary consolidation ranged from 65 to 170 days. The difference between predicted and observed time was attributed to clay smear along the clay-drain interface. Because 210 days actually were available for primary consolidation, the wick drain performance did not delay the project.

Because of the areal constraints on the placement of the fill, six temporary retaining walls with heights up to 57 ft (17 m) were required to contain the fill. The positions of these walls are shown in Figure 34, and a typical wall section is shown in Figure 35. The design documents included four alternative wall systems: a temporary Reinforced Earth wall, a fabric wall, a geogrid wall, and a Hilfiker welded-wire-mesh wall. The use of the fabric and geogrid walls was limited to areas with a maximum height of 30 ft (9 m). On the basis of competitive bidding, the Hilfiker welded-wire system was selected. A photograph of a 50-ft- (15-m-) high

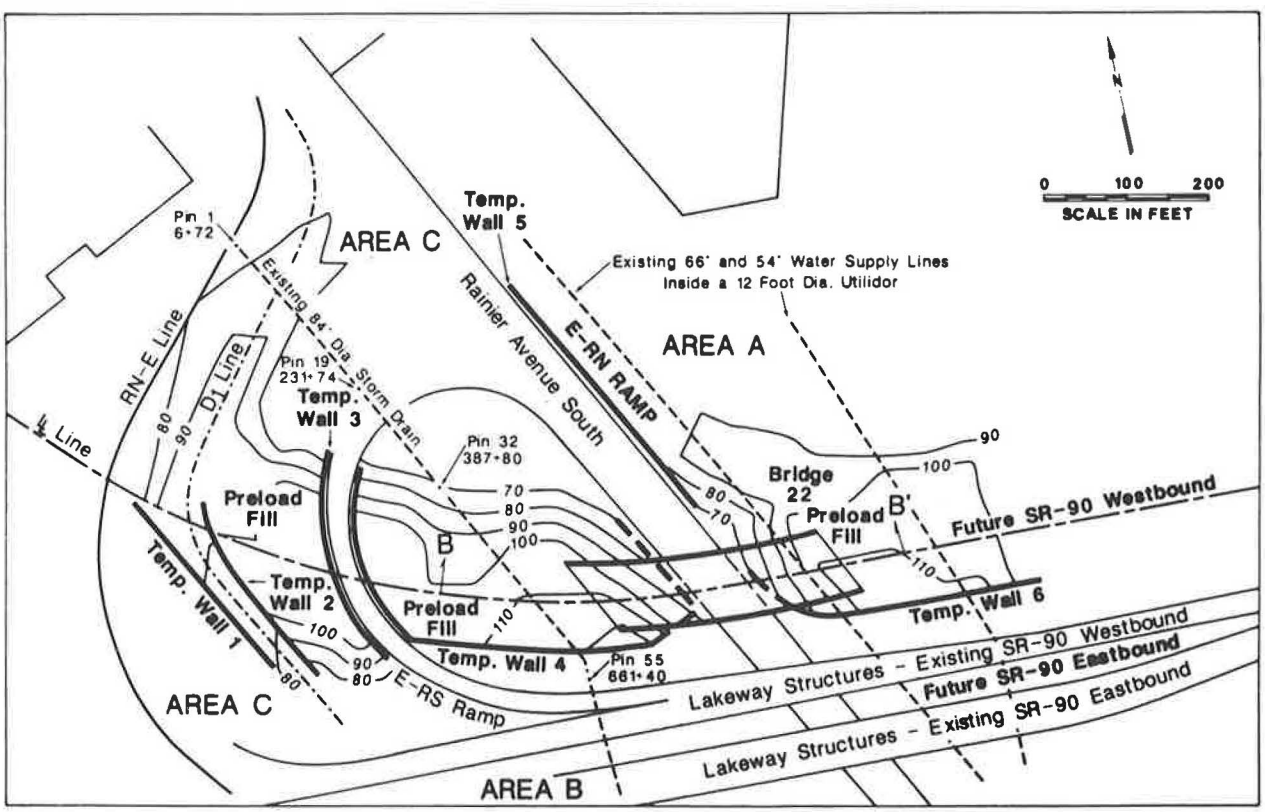


FIGURE 34 Plan of westbound embankment, Rainier Avenue (*93*).

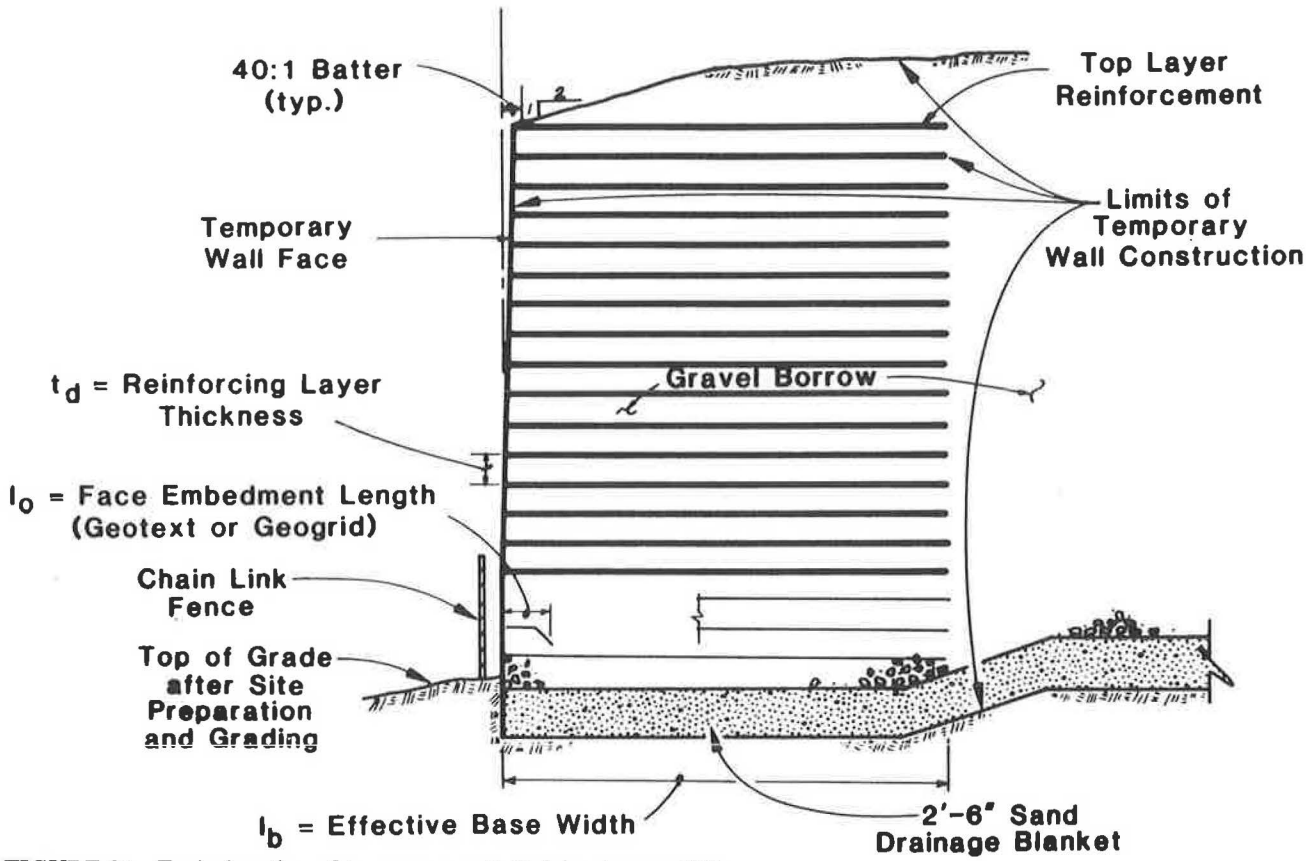


FIGURE 35 Typical section of temporary wall, Rainier Avenue (*93*).

section of the wall is shown in Figure 36. The walls, which were instrumented, performed very well. Details of the performance of the Hilfiker wall are reported by Anderson et al. (*94*).

Finally, it should be noted that the innovative concepts incorporated in the final design resulted in significant cost savings in comparison with the preliminary design developed in 1981. The preliminary design specified augercast sand drains, Reinforced Earth walls, and pile-supported abutments. The use of wick drains saved $349,550, the Hilfiker wall saved $626,800, and the elimination of piles by surcharging saved $300,000. The total savings on these three items represent a 60 percent reduction of the original 1981 estimates.

GALLOWS ROAD GRADE SEPARATION, FAIRFAX, VIRGINIA

In 1984 the Virginia DOT developed plans for a grade separation at the congested intersection of Gallows Road and Route 50 in suburban Washington, D.C. The project involved a 25-ft (7.5-m) excavation to depress the grade of Route 50 and a bridge to carry Gallows Road over Route 50 at the existing grade (*95*). In addition, permanent access ramps were required at the intersection, and both roads were widened from four to six lanes. Construction had to be done in restricted rights-of-way while maintaining traffic flow.

A proprietary system of tied-back walls and abutments was used to achieve the project objectives. The tie-back retaining wall system was constructed by driving soldier piles at 8-to-10-ft (2.4-to-3-m) centers and installing horizontal lagging between the piles as the excavation proceeded. At depth intervals of 8 ft (2.4 m), the soldier piles were anchored with steel tendons that were placed in augered holes and grouted into weathered gneiss. Finally, a concrete facing was attached to the soldier piles for aesthetic purposes. The entire section is less than 2 ft (0.6 m) thick. Figure 37 shows the wall before placement of the concrete overlay. The bridge abutments were constructed in a similar manner except that bearing piles were driven to support the vertical bridge loads and deadman anchors were used for the lateral impact loads on the bridge seats.

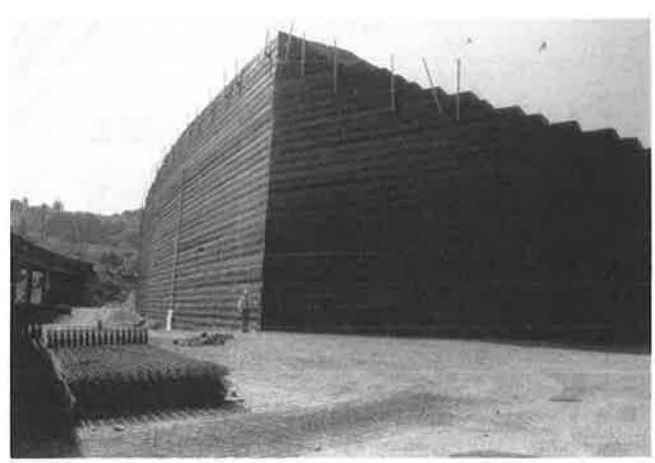


FIGURE 36 Fifty-ft-high welded-wire wall, Rainier Avenue (*94*).

The primary advantage of the tied-back walls and abutments was that no excavation was required behind the walls. By comparison, conventional retaining walls would have required excavation of a zone as wide as 22 ft (6.7 m) behind the wall (Figure 38). Because excavation was not needed behind the walls, traffic could continue on the access ramps directly adjacent to the wall construction, as shown in Figure 39. This minimized traffic delays and detours caused by construction, and the cost of the project was reduced by the elimination of temporary roads. In addition, construction time was reduced by five months.

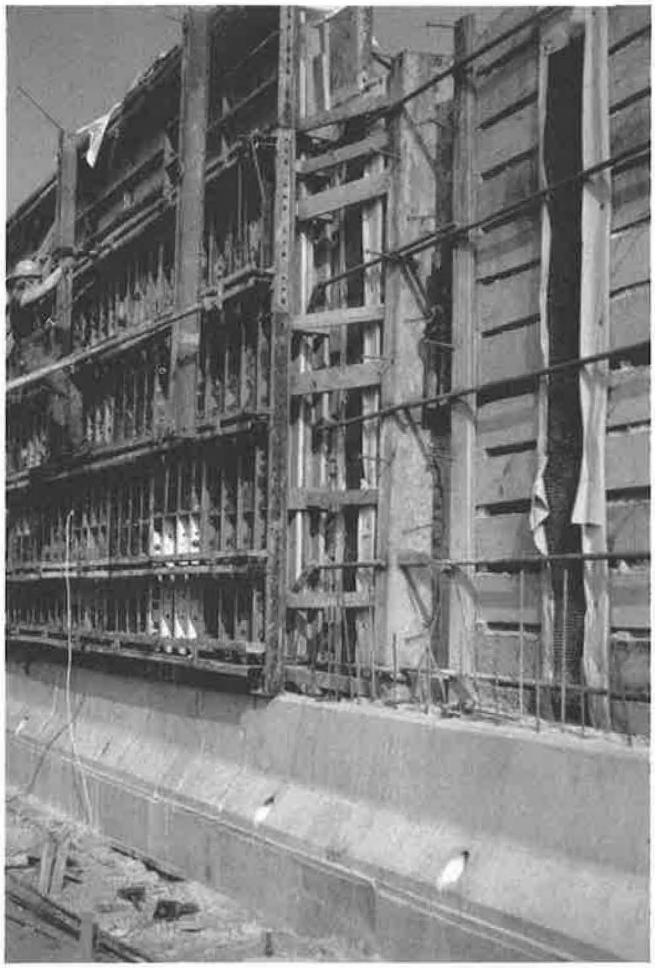


FIGURE 37 Photo of tied-back wall construction. (Courtesy of Schnabel Foundation Co.)

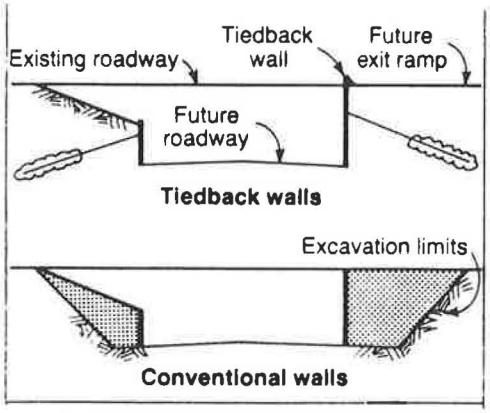


FIGURE 38 Comparison of excavated zones for tied-back and conventional walls (*95*).

FIGURE 39 Installation of tied-back walls between Route 50 and access ramps, Gallows Road grade separation. (Courtesy of Schnabel Foundation Co.)

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