NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 107

SHALLOW FOUNDATIONS FOR HIGHWAY STRUCTURES

TRANSPORTATION RESEARCH BOARD
National Research Council

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

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

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The Transportation Research Board evolved from the 54-year-old Highway Research Board. The TRB incorporates all former HRB activities and also performs additional functions under a broader scope involving all modes of transportation and the interactions of transportation with society.

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PREFACE

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

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

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of special interest to foundation designers, bridge engineers, geotechnical specialists, and others seeking information on the use of shallow foundations for transportation structures. Detailed information is presented on selection, design, construction, and maintenance of shallow foundations.

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.

Foundations represent a considerable portion of the construction costs for structures such as bridges, walls, culverts, and sign supports. Where suitable for the site, shallow foundations such as spread footings, mats, or rafts are typically less costly than deep foundations such as piles or caissons. Nevertheless, procedures used by many agencies do not allow for adequate consideration of shallow foundations. This report of the Transportation Research Board is intended to facilitate proper consideration and greater application of shallow foundations. Current practices and performance criteria are discussed, and case histories are presented to demonstrate cost comparisons between different types of foundations.

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

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

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

SHALLOW FOUNDATIONS FOR HIGHWAY STRUCTURES

SUMMARY

The type of foundation selected for a highway structure will be a major factor in both the construction and the maintenance costs of the structure. Foundation alternatives include spread footings at a relatively shallow depth; deep foundations, such as piles or caissons; or ground modification, such as densification, reinforcement, or removal and replacement.

Selection of the foundation should be based on information about the proposed structure and the site conditions. Investigations and analyses of site conditions should be carried out by a geotechnical engineer (or engineering geologist) who understands the requirements of various types of structures and the foundation alternatives. Normally, shallow foundations should be evaluated first, as they are usually the most economical alternative. The report prepared by the geotechnical engineer for the structural designer should include recommendations as to foundation type, design criteria, special construction methods, and monitoring (if needed).

Adequate evaluation of site conditions is essential for selection of foundation type. Investigation should include a review of surface conditions and should provide detailed information on soil strata, rock, and groundwater evaluation.

The evaluation of a proposed shallow foundation requires predictions of the behavior of the foundation in response to loads and environmental factors. Allowable foundation pressures are established on the basis of (a) ultimate bearing capacity of the bearing strata and (b) tolerable settlement of the foundation. Bearing capacity is based on the shear strength of the bearing strata. Settlement is based on consolidation properties of the foundation soils.

The reliability of settlement predictions depends primarily on how accurately the subsurface soil properties were determined; in turn, this depends not only on the type and variability of the soil, but on the magnitude and quality of the sampling and testing program. Soil properties estimated on the basis of general soil classifications will be much less reliable than those obtained from quality laboratory or in-situ tests. Most settlement calculation procedures are conservative; thus, settlements are more likely to be overestimated than underestimated.

The tolerable movement of a structure is difficult to define. Differential movements may induce stresses in a structure and thus are more significant than absolute movements. The acceptable magnitude of highway structure foundation movements depends on how the function of the structure is affected. For example, movements that cause only minor cracking will not be of concern. Nor will lateral movement of a retaining wall cause concern unless a roadway is supported on the backfill. However, movements that cause poor ridability, damage superstructure elements, or require costly main-

tenance or repairs will be of concern. Tolerable movements are affected by other factors including type, material, span length, and total length of the bridge. There are currently no widely accepted criteria for tolerable movements of bridge foundations.

Among the findings of this synthesis are:

- Criteria for tolerable movement currently in use could be relaxed without affecting safety or performance of highway bridges.
- A thorough geotechnical investigation, including field exploration, laboratory testing, and analysis of foundation response, is required for adequate consideration of shallow foundations.
- Ground improvement techniques are now available to make in-situ conditions more suitable for shallow foundations.
- Relatively inexpensive modifications of a bridge superstructure to make it more tolerant of movement may provide substantial savings in foundation costs.
- Research needs include improved methods for site characterization and for evaluation of engineering properties of soil and rock, field studies of selected bridges on shallow foundations, and improved design concepts and procedures.

CHAPTER ONE

INTRODUCTION

Modern highway systems require a variety of structures, particularly in urban areas and regions of rugged terrain. Land and water bridges, retaining walls, culverts, and support systems for major signs are a part of almost all highway projects, and these structures often represent a significant share of highway construction costs. Furthermore, every highway structure requires a foundation. Because the costs of various types of foundations vary significantly, the type of foundation selected will be a major factor in the total cost of a highway structure.

The geotechnical or foundation engineer must recommend the most appropriate foundation to the structural engineer. The foundation must enable the structure to perform its design function with safety and without costly maintenance. Thus, the selection and design of a foundation requires consideration of the design loads and performance requirements of the structure. In addition, the nature of the subsurface materials and their responses to the design loads and the relative costs of various types of foundations must be evaluated. Both construction and maintenance costs should be considered.

The various types of foundation alternatives that may be considered are as follows:

- I. Shallow Foundations
  - A. Spread Footings
  - B. Mats or Rafts
- II. Deep Foundations
  - A. Piles
    - 1. Timber
    - 2. Steel
      - a. H-Sections
      - b. Pipe
    - 3. Concrete
      - a. Precast
      - b. Cast-in-Place
  - B. Drilled Piers
  - C. Caissons
- III. Ground Modification (for Shallow Foundations)
  - A. Densification
    - 1. Compaction
    - 2. Preloading
  - B. Cementation
    - 1. Grouting
    - 2. Admixtures
  - C. Reinforcement
    - 1. Soil Reinforcement
    - 2. Geotextiles
    - 3. Stone Columns
  - D. Removal and Replacement

The simplest and commonly most economical foundation is a reinforced concrete spread footing founded at relatively shallow depth. When the surface soils appear to be weak or large settlements are anticipated with spread footings, deep foundations, such as piles, piers, or caissons, may be used to transmit the design loads through unsuitable soils to deeper soils or rock in which adequate support can be developed by skin friction and/or end bearing. Deep foundations typically are more costly than shallow foundations and, thus, are usually recommended only when it is believed that shallow foundations will not perform satisfactorily. Alternatively, the weak surface soils may be removed and replaced by stronger material or improved by densification, reinforcement, or additives to permit shallow foundations to be used. The cost and reliability of feasible ground modification techniques then must be compared with that of deep foundations. Many ground modification techniques are proprietary and require the use of specific firms that provide both design and construction services.

In some states, shallow foundations on soil have been used successfully for highway bridges. For example, in Washington between 1965 and 1980, more than 500 highway bridges have been constructed with one or more abutments or piers supported on spread footings (*1*). For 180 of these bridges, one or both abutments are supported on spread footings in the approach fills. An example is illustrated in Figure 1. Similar examples can be cited in other states, particularly in the northeast.

Despite the successful experiences of a relatively few states, recent surveys by FHWA (*2*) indicate that in most states spread footings rarely are used for support of highway bridges. In some states, spread footings are not considered unless they can be founded on rock. This policy may be encouraged by the current AASHTO bridge specifications, which state that "piling shall be considered when footings cannot, at reasonable expense, be founded on rock or other solid foundation material" (*3*). If spread footings on soil are not considered, then the choice of an appropriate foundation is limited to the selection of the most suitable type of deep foundation, which usually will be significantly more expensive than a shallow spread footing.

The failure to consider bridge footings on competent soil is an extremely conservative policy, which is much more restrictive than the policies used for selection of building foundations. The limited consideration of spread footings for bridges may be attributed to two factors. First, the performance criteria designated by many bridge engineers appear to be very conservative. Second, the predicted behavior of a footing on soil frequently is assumed to be of questionable reliability, often because of inadequate investigations and analyses of the subsurface materials and their response to design loads.

The purpose of this synthesis is to encourage proper consid-

FIGURE 1 Evergreen Parkway Undercrossing, Washington, a six-span concrete box girder, has both abutments and one pier on spread footings (*1*).

eration and, as a consequence, greater utilization of shallow foundations for highway structures. Current practices of transportation agencies that make significant use of shallow foundations are examined. In addition, some procedures that are not commonly used by transportation agencies but are regarded within the current state of the art of foundation engineering are identified. Practices related to bridge foundations are emphasized. Performance criteria for bridge foundations are reviewed in the light of recent studies of performance in relation to foundation movements and compared with criteria for buildings. Several case histories are presented to illustrate the potential cost differential between shallow and deep foundations.

CHAPTER TWO

FOUNDATION SELECTION PROCESS

The rational selection of a safe and economical foundation involves a systematic process of evaluation of many factors, including structural design loads, environmental effects, subsurface conditions, performance requirements, construction methods and economics. A suggested sequence of steps in this process is outlined in Figure 2 and discussed briefly below. Additional discussions of the various phases of the process are presented in subsequent chapters of this synthesis.

The foundation selection should be based on information about the proposed structure and the site conditions. Ideally, a preliminary evaluation of the subsurface conditions and potential foundation problems should be included in preliminary site-location studies. However, foundation conditions frequently are overlooked in site selection. Similarly, the type of structure usually is established before the foundation investigations. Thus, the type and size of structure, the foundation design loads, and the required performance criteria often are specified by the structural engineers with little or no geotechnical input. This can lead to problems when foundation conditions are later found to be incompatible with the structural design.

The field and laboratory geotechnical investigations should be planned by a geotechnical engineer or engineering geologist who understands the type of information that will be needed in the foundation-selection studies. This individual must recognize the requirements of various types of structures, the foundation alternatives that may be considered, and the types of analyses that will be required to make a rational selection among these alternatives.

The analysis of the behavior of various potential foundation systems in response to design loads and environmental factors is the responsibility of the geotechnical or foundation engineer. The predicted behavior of each alternative then is compared with the performance requirements established by the structural engineers. For foundations that appear to provide satisfactory performance, potential construction problems and costs are considered. Maintenance costs also should be considered. Finally,

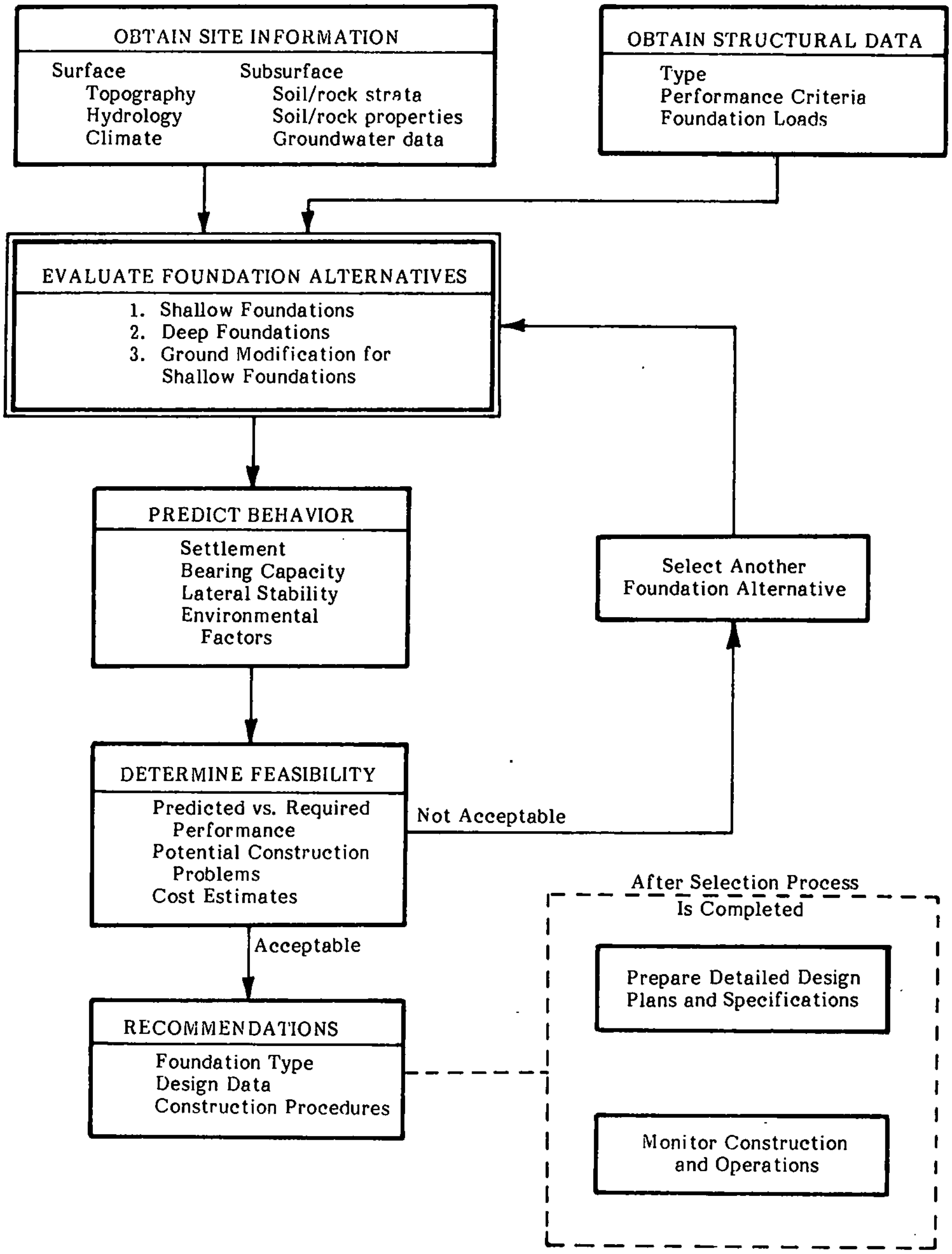


FIGURE 2 Foundation selection process.

the foundation system that will provide satisfactory performance at the lowest cost is recommended.

As noted in Figure 2, shallow foundations normally should be evaluated first. If shallow foundations will perform satisfactorily, they usually will be the most economical alternative. If the response of a shallow foundation appears to be unsatisfactory or marginal, other alternatives must be considered. Various types of deep foundations and/or ground modification techniques may be evaluated. Ideally, modification of the primary structure to reduce performance criteria also should be considered. However, this option is seldom used in current practice.

The foundation investigations and recommendations are presented in a foundation report, which is prepared by the geotechnical engineer. The report should include:

- Site description
- Boring logs and subsurface profiles
- Results of laboratory and field tests for identification, classification, and relevant engineering properties of strata
- Review of design loads
- Analyses of behavior of each foundation alternative
- Evaluation of predicted performance in relation to performance requirements
- Discussion of potential construction problems (excavation, dewatering, etc.)
- Discussion of relative costs
- Recommendations
    - Foundation type

- Foundation design criteria (allowable loads, depths, etc.)
- Special construction methods
- Construction monitoring where required.

The geotechnical engineer's recommendations are submitted to the structural designers, who ultimately must approve the design recommendations and prepare the detailed structural design and the construction plans and specifications for the foundation. The geotechnical engineer should be available to discuss the recommendations with the structural designers. Finally, the geotechnical engineer must be prepared to monitor foundation construction to detect and respond to problems that may develop. Because of the inherent variability of subsurface conditions, it is not uncommon to encounter unanticipated conditions that may significantly affect the foundation design. Minor and occasionally major design revisions may be necessary to accommodate the unforeseen conditions. The geotechnical report may recommend monitoring of field conditions during and/or after construction where experience (or lack thereof) with subsurface variability or structure sensitivity warrants.

In conclusion, the selection, design, and construction of an adequate cost-effective foundation requires coordination among geotechnical engineers, geologists, structural designers, and construction engineers. It is desirable for the agency to be organized in a manner that permits the geotechnical engineer to be directly involved in all phases of the foundation work from preliminary planning through construction and maintenance.

CHAPTER THREE

TYPES OF STRUCTURES AND DESIGN LOADS

BRIDGES

Highway bridges range in size and type from small single-span beam and girder bridges to huge multi-span suspension bridges. They are constructed primarily of steel, reinforced concrete, or prestressed concrete, but masonry and timber are still used in some cases. For single-span structures, the superstructure is supported on two abutments. For multi-span structures, one or more intermediate piers are required in addition to the abutments.

The loads transmitted to the substructure depend on the size and type of superstructure. Foundations and design loads must be identified for both piers and abutments.

Piers

The foundation loads for piers include the dead load of the substructure and superstructure, traffic live loads, wind loads, longitudinal forces, and earthquake loads. For stream crossings, forces from ice and stream current must also be included. Recommended magnitudes for these loads are given in the AASHTO Standard Specifications for Highway Bridges (*3*).

Abutments

Foundations for bridge abutments are subjected to the same loads as those listed for bridge piers. In addition, the abutment foundation will be affected by the loads of the approach fill. The interaction of approach fills and abutments has been discussed more fully in NCHRP Synthesis 2 (*4*). Three general types of abutments are commonly used, and each is affected differently by the embankment loads.

Stub or Perched Abutment

A stub or perched abutment is constructed after the embankment has essentially been completed. Because of its position and low height, the lateral earth pressures against the back of the abutment are relatively low. If the stub abutment is supported on a spread footing, the footing will tend to move or "float" with the embankment movement. Therefore, it is important to consider the postconstruction settlement and lateral movement of the approach embankment as part of the design analysis. If the stub abutment is supported on piles, lateral earth pressures against the piles become significant and may cause rotation of the piles and the abutment. Differential settlement may develop between the embankment and the abutment, creating downdrag on the piles.

Most of the preceding problems can be minimized by stage construction in which the major settlement and lateral movement occur before construction of the abutment. The primary advantage of the stub abutment is that the embankment can be constructed and properly compacted without interference from the abutment. Also, if spread footings are used, the differential settlement between the abutment and the approach embankment will be minimized. Stub abutments can also be used in cut slopes where they can be founded on natural ground. Usually the toe of an abutment footing must be located a sufficient distance from the embankment or cut slope to provide adequate bearing capacity and frost protection for the footing. This may require a longer bridge span than is required with a closed full-height abutment.

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. Its major advantage is in limiting the length of bridge spans. The major disadvantage is that the embankment fill near the abutment can not be placed until after the abutment has been constructed. The backfill between the abutment and the wing walls is difficult to place and to compact properly, and the weight of this backfill will contribute to postconstruction settlement if compressible cohesive soils are beneath the abutment. Furthermore, large lateral earth pressures will be developed against the walls. These pressures normally are calculated by Rankine active pressure theory or equivalent fluid pressure concepts. Thus, the evaluation of foundation requirements for closed abutments must include the lateral forces and overturning moments produced by these lateral earth pressures as well as the effects of the embankment loads on the foundation soils beneath the abutment.

Spill-through or Pedestal Abutment

Spill-through or pedestal abutments are essentially short stub abutments supported on columns or pedestals that extend to natural ground. The concept is to allow the approach embankment to spill through the spaces between the pedestals, thus minimizing the lateral earth pressure against the abutment. The abutment is constructed and then the fill usually is placed simultaneously on both sides of the abutment to minimize unbalanced earth pressures. As with closed abutments, it is difficult to obtain adequate compaction around the pedestals. Because the fill is not retained by a closed wall, poorly compacted soil is more susceptible to erosion, and settlement of the embankment surface is likely to occur. Lateral pressures act against the pedestals. However, the lateral forces and overturning moments that act on the foundation will be much less than for a closed abutment. If compressible soils exist beneath the abutment, the response of the foundation soils to the weight of the backfill also may be a significant factor.

RETAINING WALLS

Several types of conventional retaining walls are commonly used. Low walls [less than 20-ft (6-m) high] usually are gravity, semi-gravity, or cantilever types. For higher walls, counterforts or buttresses may be necessary for economical structural design.

The foundation loads for all types of retaining walls include the dead load of the wall and the backfill, surcharge loads applied to the backfill, seismic loads, and the effects of the lateral earth pressure and any hydrostatic water pressures that may develop behind the wall. Drainage should be provided, however, to prevent the buildup of hydrostatic pressures. The earth and water pressures impose both lateral forces and overturning moments on the foundation. The earth pressures behind retaining walls usually are computed from Rankine theory for active earth pressure conditions. For low walls, an equivalent fluid weight (mass) of 30 lb/ft^3 (480 kg/m^3) sometimes is used. This value may be satisfactory under some conditions, such as for a free-draining backfill with a horizontal surface.

Consideration also must be given to the direct effect of the backfill on the foundation soils. Where a retaining wall is founded on a highly compressible soil, the weight of the backfill may produce significant compression of the foundation soil beneath the wall and thus contribute to settlement of the wall.

CULVERTS

Culverts range from small diameter concrete and metal pipes to large span steel and reinforced or prestressed concrete box culverts. The primary function of a culvert is to provide a conduit for water; thus the primary design considerations involve selection of size and shape on the basis of hydraulics and hydrology.

The foundation loads for culverts depend on the dead load of the culvert, the overburden pressure on the top of the culvert, and live loads for shallow embedments. However, the overburden pressure on a culvert is known to be a function of the flexibility of the culvert. For a flexible culvert, the weight of the backfill arches over the culvert, and the vertical pressure on the culvert is lower than for a stiffer culvert. Thus, the foundation loads are affected by this arching phenomenon. Further consideration of the evaluation of loads on buried culverts is considered beyond the scope of this synthesis.

SIGN AND NOISE BARRIER SUPPORTS

Foundation loads for sign or noise barrier supports include dead loads and wind loads. The dead load usually will be nominal, and the major loads are the lateral forces and overturning moments generated by wind forces against the sign or noise barrier and its supports.

CHAPTER FOUR

SITE CONDITIONS

Adequate evaluation of surface and subsurface site conditions is essential to every geotechnical project. The type and magnitude of the site investigation depend on the size and function of the proposed structure and the complexity of the site. Also, the types of foundations to be considered will affect the information that is required. For example, consideration of spread footings will require reliable estimates of the compressibility and bearing capacity of shallow soil strata whereas investigations for deep foundations will require similar data for deeper bearing strata.

SURFACE INVESTIGATIONS

Site investigations should begin with the review of surface conditions. Aerial photographs; geologic, topographic, and agricultural soils maps; water well logs; and stream-gage records may provide useful preliminary information. Visual observations on the site may provide additional information regarding:

- Nature of surficial soils
- Nature and extent, if any, of rock in outcrops or cuts
- Presence and condition of old fills
- Evidence of erosion or deposition along stream banks
- Evidence of and potential for undermining or scour
- Evidence of previous flooding
- Stability of existing natural and cut slopes
- Indications of previous slides or creep of natural slopes
- Proximity and performance of existing structures in similar geologic and environmental conditions at or near the site

Transportation agencies generally have good knowledge of the geologic conditions that are likely to be found within their jurisdictions. Surface conditions usually are identified adequately. Sometimes, however, visual evidence of previous problems may be overlooked, particularly if the site observations are made by inexperienced or untrained personnel. It is a good practice for field reconnaissance to be conducted by experienced geotechnical engineers and/or geologists.

SUBSURFACE INVESTIGATIONS

Subsurface investigations should provide all information that may be required for analyses of various foundation alternatives. Relevant information may include some or all of the following items:

I. Soil strata
   A. Depth, thickness, variability
   B. Identification and classification data
   C. Relevant engineering properties
      1. Shear strength
      2. Compressibility
      3. Stiffness
      4. Permeability
      5. Expansion or collapse potential
      6. Frost susceptibility
II. Rock
   A. Depth to rock
   B. Quality (soundness, hardness, jointing, resistance to weathering if exposed, etc.)
   C. Compressive strength
   D. Expansion potential
III. Ground water elevations

The observations and evaluations required for a specific project must be selected by the geotechnical engineer on the basis of the size and complexity of the project and the types of analyses that are anticipated. For example, quantitative engineering properties may not be required for a soft organic layer that is judged to be unsuitable on the basis of identification and classification data. Similarly, information on bedrock may not be needed for a small, lightweight structure (e.g., a sign support or noise barrier). Guidelines for subsurface investigations are presented in the AASHTO "Manual on Foundation Investigations" (*5*). This manual, which was published in 1967, currently is being revised extensively and the new edition should be available in 1984.

Transportation agency practices for the acquisition and presentation of subsurface information were summarized in 1976 in NCHRP Synthesis 33 (*6*). At the same time, West Virginia University (*7*) conducted a survey of geotechnical practices related to abutments for short-span bridges. A 1981 FHWA (*2*) summary of foundation practices for highway structures indicates that the 1976 reports still provide a good summary of the current subsurface investigation practices of transportation agencies. Therefore, these practices will be summarized only briefly, and the reader is referred to Synthesis 33 and the AASHTO Manual for more details. In addition, some recent trends and advances in in-situ testing techniques will be discussed.

Transportation agencies generally have the capability for conducting soil borings with disturbed and undisturbed sampling or may obtain them by contract drilling when necessary. For structural foundations, auger borings with disturbed split-barrel sampling and standard penetration tests are performed routinely by most agencies. Undisturbed samples of cohesive soils usually are obtained by agencies that consider shallow foundations. Most undisturbed sampling is done with thin-walled Shelby tubes rather than with piston samplers because of the high cost of piston sampling, although the latter gives less-disturbed samples. Rock coring is performed routinely by most agencies. Seismic refraction methods occasionally are used to determine the depth to rock.

The type of soil sample required depends on the purpose for which the sample is to be used. Auger cuttings may be sufficient for visual classification. Disturbed samples (e.g., split-barrel samples) can be used for classification tests. However, undisturbed samples are required for laboratory tests of engineering properties, which are very sensitive to sample disturbance. Because such properties are required for analyses of settlement and stability, undisturbed samples of cohesive soils usually are essential to the consideration of shallow foundations. The sample requirements for specific laboratory tests are listed in Table 1.

The number and depth of borings vary with the type and size of the proposed structure. Most agencies require at least one boring for each substructure unit. For major bridges, two to four borings per substructure unit may be more typical. For retaining structures, borings may be spaced at 50- to 100-ft (15- to 30-m) intervals along the length of the wall with consideration given to the topographic features of the site. For lightly loaded sign supports or noise barriers, structural borings often are omitted, and the foundation design is based on nearby roadway borings.

Many agencies do not have specific requirements for the minimum depth of structural borings, but all appear to extend investigations to adequate depths. Borings typically extend through soft deposits and 10 to 30 ft (3 to 9 m) into hard or dense soil. When deep foundations are to be considered, rock coring is common. It is good practice to extend borings to the depth at which the stress increase caused by the structure does not exceed 10 percent of the original effective overburden pressure.

TABLE 1

SAMPLE REQUIREMENTS FOR VARIOUS LABORATORY TESTS.

Disturbed Samples May Be Used	Undisturbed Samples Required
Visual Classification	Unit weight
Moisture Content	Permeability
Specific Gravity	Consolidation
Atterberg Limits	Direct shear
Grain size distribution	Unconfined compression
Compaction	Triaxial compression

EVALUATION OF SOIL PROPERTIES

Simple soil identification and classification tests are performed on almost all highway projects. Generally, field densities, moisture contents, and Atterberg limits are determined for cohesive soils, and gradation data and relative densities are reported for cohesionless soils. These data enable agencies to classify soil strata in accordance with the AASHTO or Unified Soil Classification Systems.

Most agencies measure the dynamic penetration resistance, in blows/ft, during the driving of split-barrel samplers and use this value to provide an estimate of the consistency of cohesive soils and the relative density of cohesionless soils. Correlations based on the standard penetration test, which is used by most agencies, are given in Table 2. A few agencies (e.g., in New York and Texas) use other penetration tests, and in such cases, other correlations must be developed.

Analyses for consideration of shallow foundations require the strength and deformation properties of soil strata. Compressibility, elastic modulus, and shear or compressive strength are the properties of primary importance. Reliable values of these properties are obtained by direct measurement of the desired property either in the field or in the laboratory. For cohesive soils, the most common practice is to perform laboratory tests on undisturbed samples. Consolidation tests are used to evaluate compressibility, and unconfined compression or undrained triaxial compression tests are conducted to determine the strength of cohesive soils. A few agencies use in-situ vane shear tests to measure the undrained shear strength of soft clays and organic soils from which good quality samples cannot be obtained economically.

Laboratory and field strength tests are conducted to evaluate the in-situ strength under current or potential future field loading and environmental conditions. Because the strength of a clay is related to stress history and age, these factors must be considered in selecting the strength-testing procedures. Furthermore, in addition to effects of sample disturbance, it also is recognized that measured values of shear strength are affected by the size of sample, the type of test, and the rate of strain at which it is conducted. A good discussion of test procedures and factors that affect the evaluation of the undrained and the effective strengths of clays has been presented by Wahls (*8*) and by Raymond (*9*).

For small projects or preliminary studies for larger projects, correlations between engineering properties and various classification or index properties frequently are used. For example, Table 2 includes an estimate of the unconfined compressive strength that is based on the blow count from the standard penetration tests, and Figure 3 shows a correlation between plasticity index and the effective angle of internal friction. Figure 4 illustrates the statistical correlation among cohesion, effective overburden, and plasticity index. A correlation between compressibility and natural water content is given in Figure 5. Several other potential correlations that may be found in the literature are listed in Table 3. Such correlations can provide reasonable estimates of engineering properties, particularly when they are based on experience with local soils. However, they also may be very unreliable when based on data for soils from other geographic regions, geologic origins, or environmental conditions.

For cohesionless soils, undisturbed samples rarely are obtained. In almost all cases, the in-situ standard penetration test is the only source of information for estimates of the engineering properties of sands and gravels. Sometimes the penetration resistance, in blows/ft, is used to estimate relative density or angle

TABLE 2

PENETRATION RESISTANCE AND SOIL PROPERTIES ON BASIS OF THE STANDARD PENETRATION TEST [AFTER (*13*)].

Sands (Fairly Reliable)		Clays (Rather Unreliable)		
Number of Blows per ft, N	Relative Density	Number of Blows per ft, N	Consistency	Unconfined Compressive Strength, Kips/ft^2
		Below 2	Very soft	0-0.5
0–4	Very loose	2–4	Soft	.5-1
4–10	Loose	4–8	Medium	1-2
10–30	Medium	8–15	Stiff	2-4
30–50	Dense	15–30	Very stiff	4-8
Over 50	Very dense	Over 30	Hard	8

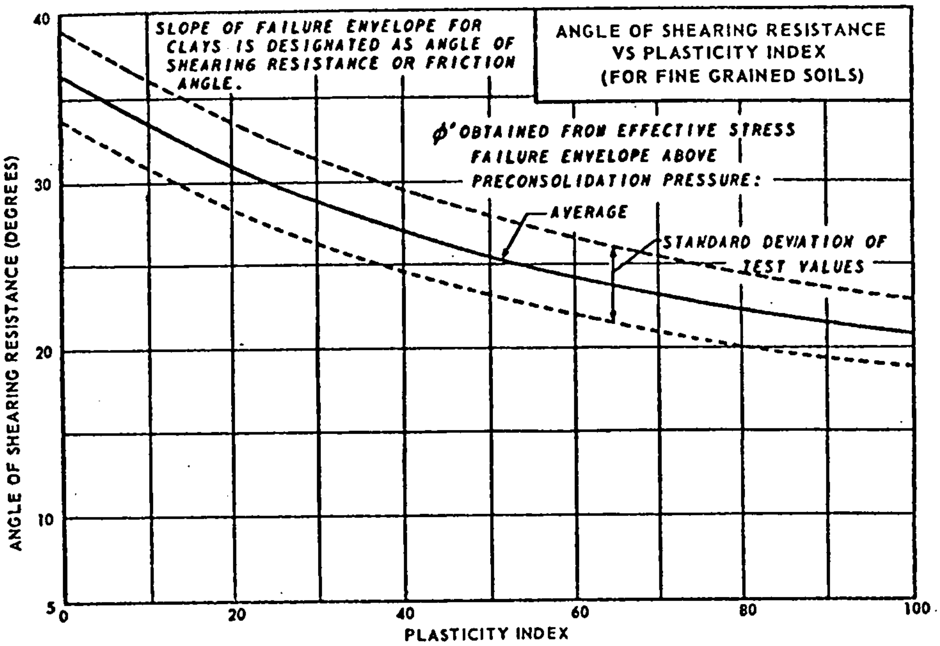


FIGURE 3 Correlation between effective friction angle and plasticity index for fine-grained soils (*10*).

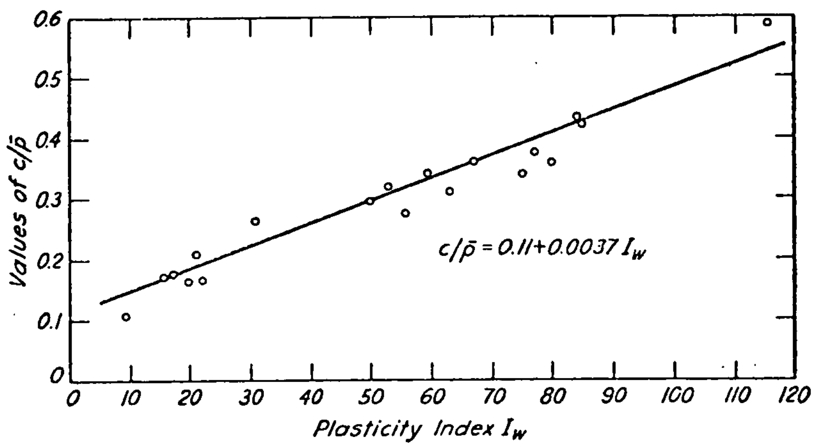


FIGURE 4 Relation between c/p̄ ratio and plasticity index (*11*).

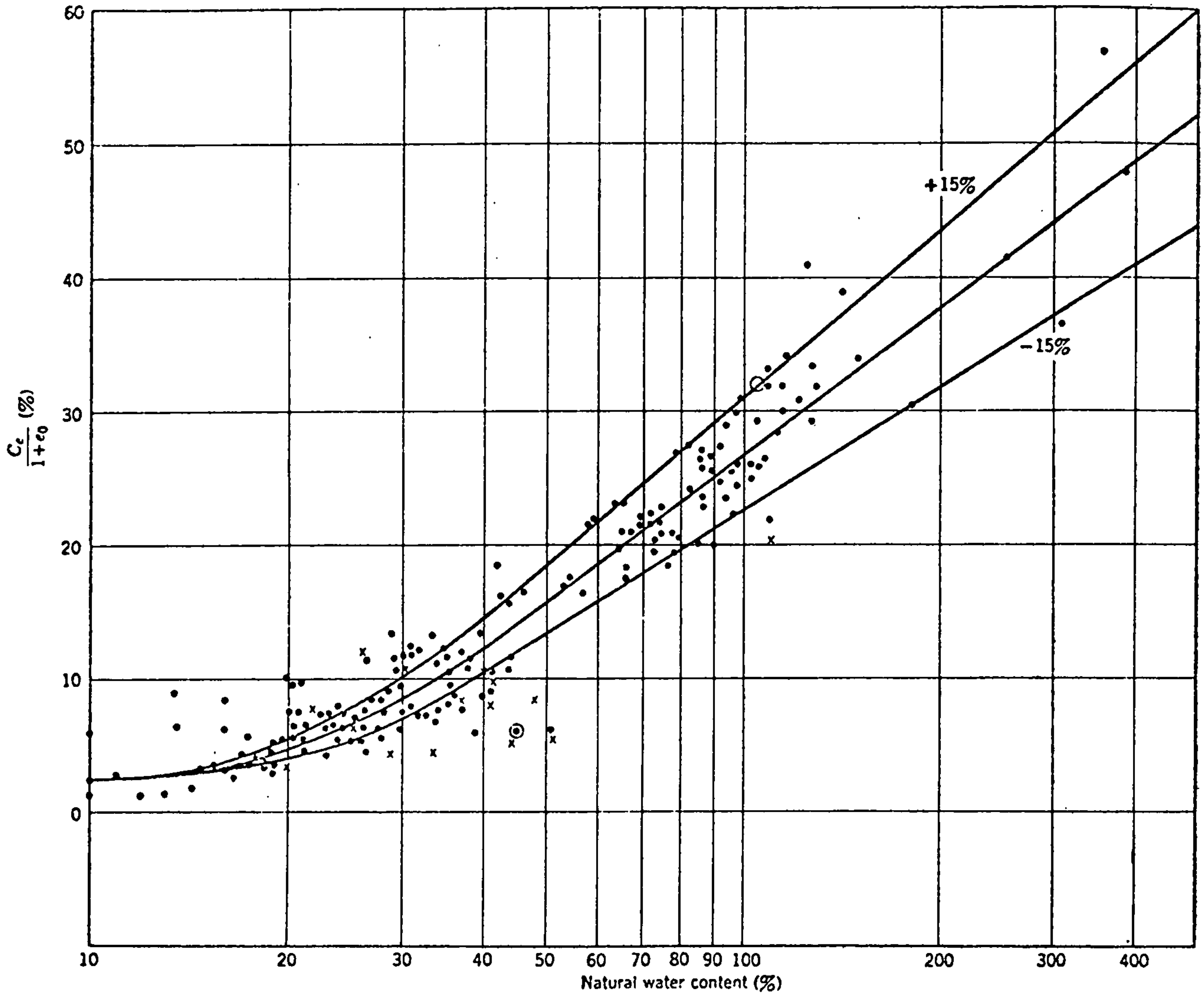


FIGURE 5 Relation between compression ratio and natural water content (*12*).

TABLE 3

CORRELATION OF TEST DATA (*21*).

Simple Test	Possible Correlation
Water content	Shear strength of clay. Compression index of clay.
Grain size (D_{10}, D_{15}, Cu)	Permeability, strength, and drainability of cohesionless soils.
Liquid limit, LL	Compressibility.
Plastic index	Swell-shrink, ϕ_D, of clays.
Water plasticity ratio, R_w	Potential swell-shrink; preconsolidation load.
Void ratio, e, unit weight, γ	Compressibility, shear strength.
Relative density, D_r	Strength, compressibility of cohesionless soil.
Seismic velocity, V	Modulus of elasticity; strength of soil, rock.
Electrical resistivity, ρ	Water, clay, organic, and salt content.
Penetration resistance, static and dynamic	Shear strength, relative density, modulus of compressibility.

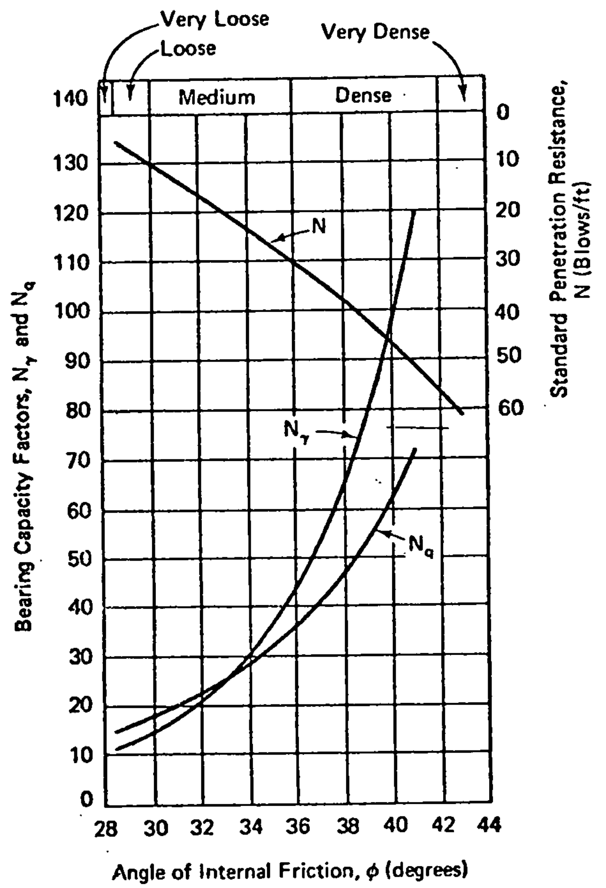


FIGURE 6 Curves showing the relationship between bearing capacity factors and θ, as determined by theory, and rough empirical relationship between bearing capacity factors or θ and values of corrected standard penetration resistance N [after Peck et al. (*13*)].

of internal friction, as illustrated in Figure 6 or compressibility, as in Figure 7. The measured blow counts often are adjusted for the influence of effective overburden pressure by use of a correction factor as shown in Figure 8. The correction factors in Figure 8 are regarded to be more reliable than previously published factors; however, they require a preliminary estimate of relative density. In other instances, allowable design loads are related directly to corrected blow counts without an intermediate evaluation of specific strength or compressibility properties. Examples of this approach will be presented in a subsequent chapter.

The reliance on penetration resistance for evaluation of cohesionless soils is typical not only for transportation agencies but also for most geotechnical engineering practice in the United States. However, in recent years there has been a significant trend toward the use of other in-situ testing methods, particularly for soils that are difficult to sample. (*16*) These methods include vane shear, static cone penetrometer, pressuremeter, and, more recently, dilatometer tests. All of these test methods originated in Europe and practices for their use are more firmly established there.

The vane shear test measures the undrained shear strength of soft cohesive and organic soils. It is not applicable to cohesionless materials. The cone penetrometer measures the force required to advance the cone into undisturbed soil. (*17*) Usually, a friction sleeve is used, which allows the point resistance and side friction to be evaluated separately. Both mechanical and electrical tips are used. Electrical cones have a higher initial cost, but they appear to provide more accurate and repeatable results, faster testing, and the potential for automatic logging and data acquisition. The cone resistance then is used either to estimate stiffness and/or shear strength parameters or directly as a measure of allowable design loads. Recent innovations in cone penetrometer techniques include the piezocone, which uses a piezometer to monitor pore water pressures generated during the test. Another experimental device includes a microphone attached to the cone to monitor the acoustic emission from the soil-cone interface. Both the pore pressure and the acoustic response change with soil texture and thus are useful when soil samples are not obtained for visual identification. The cone penetrometer may be used in either cohesive or cohesionless soils. However, the primary current use in the United States appears to be for predominantly cohesionless deposits.

Pressuremeters measure the pressure required to expand a cylindrical cavity in a soil. The pressure-volume change relation

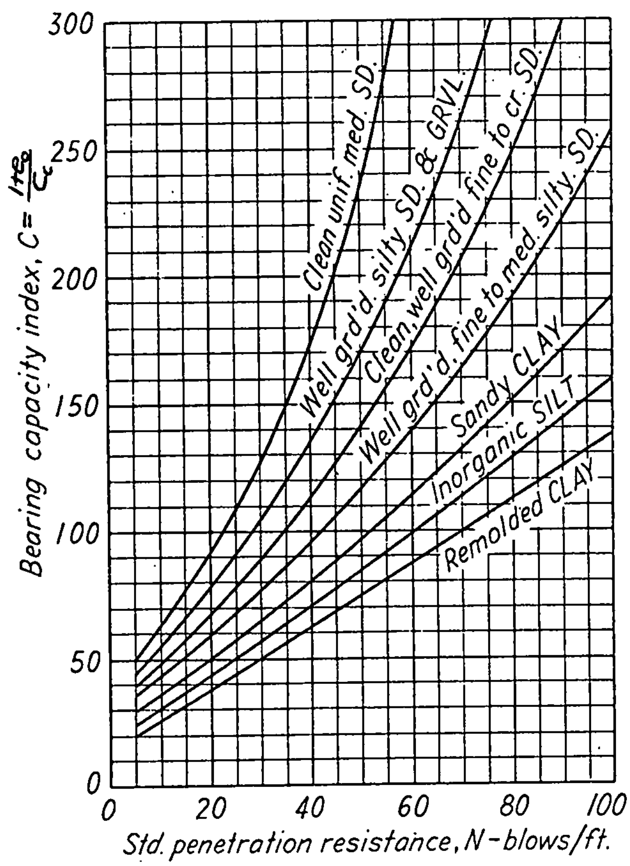


FIGURE 7 Bearing capacity index for compressibility vs. corrected standard penetration resistance (*14*).

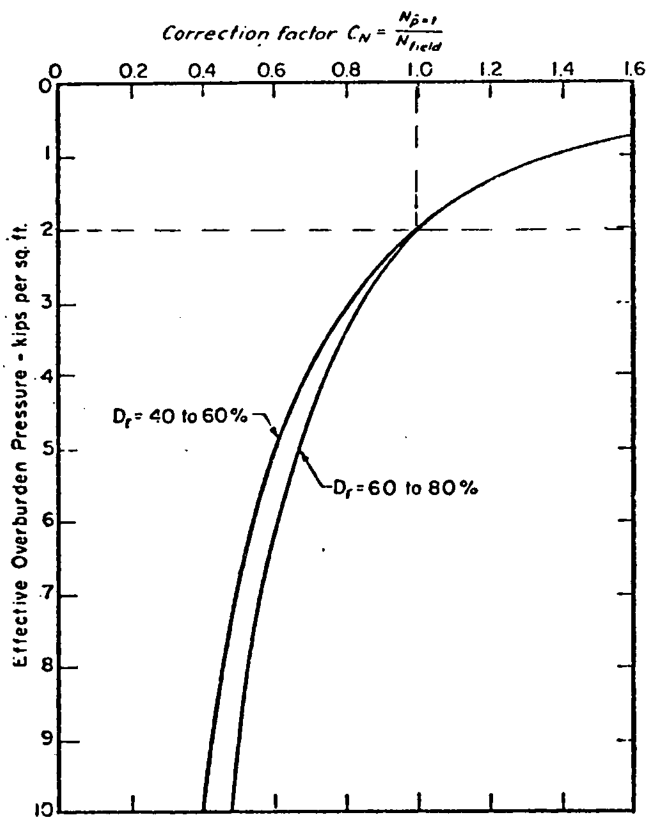


FIGURE 8 Correction factors for influence of overburden pressure on N-values in sand based on W.E.S. tests (*15*).

is used to estimate stiffness and strength parameters and also is used as a direct predictor of design loads. (*18*) The most commonly used Menard-type pressuremeter is driven into granular soils or lowered into a drilled hole in cohesive soils. Self-boring pressuremeters have been developed in England and France, but they appear to have been used only on a research basis in the United States. The Menard-type probes are being used in practice by consulting firms for evaluation of deposits that are difficult to sample (e.g., partly saturated residual clayey sands in the southeastern states).

The dilatometer is another new and simple in-situ device, which was developed by Marchetti (*19*) in Italy and very recently was introduced in the United States. A small flexible steel membrane is mounted on the side of a flat plate. After the plate has been pushed into undisturbed soil, the membrane is deflected out laterally by means of an applied internal pressure. The pressure at initial soil contact and the pressure required to deflect the membrane 1 mm (0.04 in.) into the soil are recorded. Correlations have been developed between these two pressures and soil type, undrained strength, constrained modulus, overconsolidation ratio, and coefficient of lateral earth pressure at rest. The device is appealing because of the ease and speed of operation as well as for the wide range of parameters that may be estimated.

Several transportation agencies are experimenting with one or more of these in-situ testing techniques. It is likely that the next decade will see a significant increase in the use of these devices in practice. This should provide improved evaluations of the engineering properties of soft cohesive soils and cohesionless soils, which are most difficult to sample and test today.

CHAPTER FIVE

ANALYSES OF SHALLOW FOUNDATIONS

The evaluation of a proposed shallow foundation requires predictions of the behavior of the foundation in response to structural loads and environmental factors. The imposed loads, environmental conditions, and the relevant properties of the subsurface materials are used to consider the following types of analyses.

ENVIRONMENTAL FACTORS

Scour

At stream crossings, the potential depth of scour should be estimated, and the foundation must be placed below this depth. The amount of scour will depend on many factors including the erodibility of the soil, the geometry of the stream bed and bank, and the estimated stream velocities during floods. Detailed discussion of scour is found in NCHRP Synthesis 5 (*20*). AASHTO specifications require footings for stream piers and arch abutments to be founded at least 6 ft (1.8 m) below stream bed. Footings for other structures, except culverts, that are exposed to stream currents must be at least 4 ft (1.2 m) below the stream bed. Because of the unpredictability of scour, many agencies require deep foundations at all stream crossings with erodible soils.

Frost

Shallow foundations should be founded below the maximum depth of frost penetration to minimize the potential for movements that may accompany seasonal freezing and thawing of many soils. The maximum depth of frost penetration generally is established by local experience or from maps, such as Figure 9. Frost problems are likely to be most severe for lightweight structures on frost-susceptible soils (e.g., silts).

Expansion/Collapse Potential

In some parts of the United States, primarily the arid and semi-arid regions of the West and Southwest, expansive or collapsible soils may be encountered. High-plasticity clays and clay shales undergo very large volume changes with wetting and drying cycles. During wet periods, extremely high swell pressures [e.g., 8–10 ksf (380–480 kPa)] may be generated and structures with light to moderate bearing pressures may be lifted and damaged. On the other hand, some partly saturated silts and lightly cemented sands may collapse when inundated. These phenomena will have a significant impact on the performance of structures on shallow foundations. In areas where these problems are encountered, the expansion or collapse potential should always be considered during the investigations of site conditions. The swell potential can be evaluated on the basis of natural moisture contents, Atterberg limits, soil suction, and modified consolidation tests. Similarly, collapse potential can be identified with consolidation tests. It should be noted that expansive soils will have a significant impact on the design of deep foundations as well as shallow foundations.

LATERAL STABILITY

When lateral loads are involved, the safety against overturning and horizontal sliding of the foundation must be evaluated. Conventional analyses of this type use shear strength data to consider safety against catastrophic failure but do not predict magnitudes of lateral movements. Lateral earth pressures usually are based on classic Rankine theory or equivalent fluid pressure concepts. Typically, factors of safety of 1.5 to 2 are desired. Prediction of lateral movements requires complex analytical methods (e.g., finite-element analyses) and detailed knowledge of the stress-strain relations for the backfill soils. The reliability of these methods is extremely sensitive to the soil properties, and thus careful and extensive soil testing is necessary when such analyses are anticipated. Consequently, lateral movement predictions are rarely made in routine practice because of the complexity, high cost, and low reliability of current evaluation procedures.

ALLOWABLE DESIGN LOADS

Allowable foundation pressures are established on the basis of two criteria: (a) ultimate bearing capacity or rupture analysis of the bearing strata and (b) the tolerable settlement of the foundation. The ultimate bearing capacity of the soil is computed on the basis of the estimated shear strengths of the bearing strata and a factor of safety of 2.5 to 3 is used to establish the upper limit of the allowable foundation pressure. The settlement of the foundation is computed for the foundation design loads and the proposed foundation pressure. The consolidation properties of the foundation soils are used to compute both the magnitude and rate of long-term settlement. Elastic properties may be used to calculate immediate settlement. However, the elastic settlement occurs during construction and often is small in relation to consolidation settlements. The predicted settlements must be within tolerance limits established for the structure, which will be discussed fully in Chapter 6.

In most practical cases, the allowable settlement is the governing factor. In the following sections, typical practices of transportation agencies for establishing allowable footing pres-

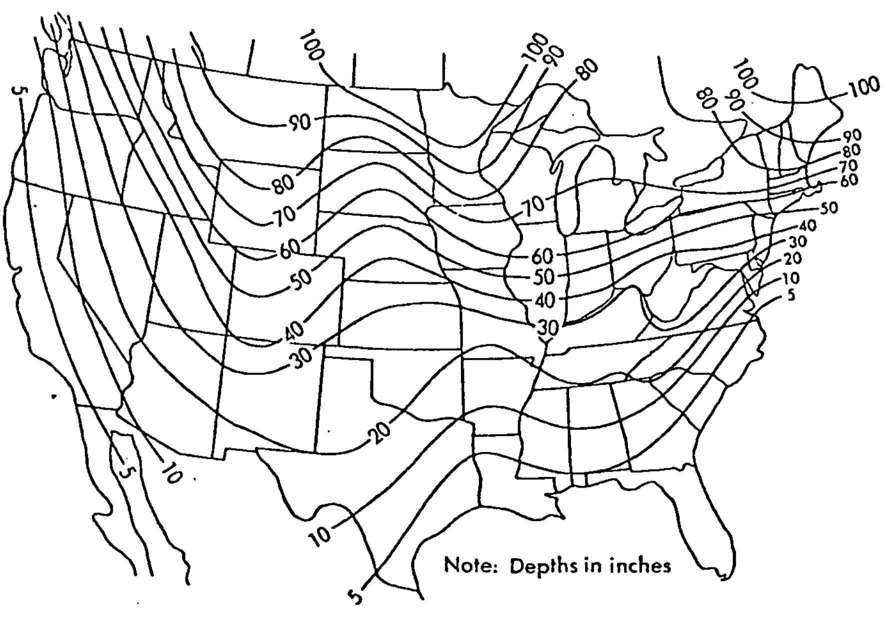


FIGURE 9 Maximum depth of frost penetration in the United States [after Sowers (*21*)].

sures are reviewed for cohesive, cohesionless, and compacted soils and for rock.

Cohesive Soils

The ultimate bearing capacity of cohesive soils usually is based on the undrained strength at the start of construction. This is a reasonable and conservative approach for most conditions because the soil should become stronger as it consolidates with time. However, the use of initial undrained strength may not be conservative in special cases in which the cohesive soil is unloaded permanently during construction. For example, the soil beneath a footing at the base of a deep permanent cut may lose strength with time if the stress release caused by the excavation is greater than the stress increase produced by the footing loads. In such cases, the drained or effective shear strength must be used to estimate the long-term bearing capacity.

Classic bearing capacity theory suggests that the ultimate bearing capacity, q_o, of a cohesive soil for undrained conditions is

$$q_o = 5.14 s_u F_1 + D_f \gamma$$

where

s_u = undrained shear strength,
D_f = depth of embedment,
γ = unit weight of soil above the footing,
$F_1 = 1 + 0.2 \frac{B}{L}$ = shape factor,
B = footing width, and
L = footing length ($B \leq L$).

The net allowable bearing pressure, q_a, is q_o minus the overburden pressure at the footing level and divided by a factor of safety, $F_s = 2.5 - 3$. Thus,

$$q_a = \frac{5.14 s_u F_1}{F_s} \simeq 2 s_u = Q_u$$

where Q_u = unconfined compression strength. As noted in the preceding equation, the common assumption that q_a is equal to the unconfined compression strength provides a simple and conservative approximation for the analysis of cohesive soils for undrained conditions and horizontal surfaces. Near slopes, lower values of q_a must be used (*22*).

The net allowable bearing pressure from bearing capacity considerations provides the upper limit for the allowable footing pressure. The settlement caused by this pressure then must be computed and compared with the tolerable settlement criteria for the structure. If the estimated settlements are too large, then the allowable footing pressure must be reduced to a value that will produce acceptable predicted settlements.

Relatively conventional practices exist for computation of the compression of cohesive soils. The vertical stress increase, Δp, caused by the footing loads is calculated as a function of depth. Usually Boussinesq theory is used. In some instances, Δp is estimated by spreading the footing load uniformly over an area that increases with depth. This approximation is satisfactory when only the settlement of the center of an isolated footing is desired. The Boussinesq method is preferred when differential settlement between the center and edge of a footing is required or the interaction among several footings is considered. The preconsolidation pressure and the compression indices for the recompression and the virgin compression ranges are obtained from consolidation tests and used to compute the settlement caused by primary consolidation. The secondary compression characteristics of the soils should be reviewed, but usually secondary effects will be negligible for soils capable of supporting shallow foundations. Procedures for computing consolidation settlements are presented in Transportation Research Board Special Report 163 (*23*).

The settlement of a bridge abutment is affected by the load of the approach embankments as well as that of the bridge superstructure. Because of the large areal extent of the embankments, significant stress increases are produced at great depths [e.g., 50–100 ft (15–30 m)] beneath the abutment. If a soft cohesive layer is located at such depths, the settlement of the abutment footing may be governed primarily by the embankment loading rather than the footing loads. This interaction between the embankment and abutment loads must be recognized and considered in the analysis of shallow foundations for abutments. Similarly, backfill loads must be considered in the analysis of settlement of a retaining wall footing.

Consolidation test data also may be used to predict rates of settlement. Such computations are of interest when the feasibility of certain construction procedures, such as surcharging or stage construction, is considered. Estimates of settlement rates generally are less reliable than the estimates of final settlement magnitude. If the rate of settlement is critical to construction schedules, the settlement should be monitored in the field during construction.

Some transportation agencies do not consider shallow foundations on cohesive soils because of the concern for excessive settlement. This is a costly policy that is much more conservative than that used in selecting foundations for buildings. It prohibits the use of shallow foundations on some cohesive soils that can be demonstrated to provide satisfactory support for highway structures.

Cohesionless Soils

Classic bearing capacity theory indicates that the ultimate bearing capacity of cohesionless soils increases linearly with footing width and also increases rapidly with depth of embedment. On the other hand, the footing pressure that will produce a given amount of settlement decreases as the footing width increases. Thus, as illustrated conceptually in Figure 10, the allowable footing pressure will be governed by the tolerable settlement, except for very small footings at or near the surface.

In most current U.S. practice, the contact pressure-settlement relation is based on the blow counts from standard penetration tests. Usually this is the only quantitative information available for cohesionless soils. Most transportation agencies appear to use empirical curves proposed by Peck et al. (*13*) and shown in Figure 11. These curves relate the contact pressure, q_1, that will cause 1 in. (25 mm) of settlement to the footing width and the adjusted blows/ft from a standard penetration test. Also, Figure 11 assumes that the depth to the water table is greater than the

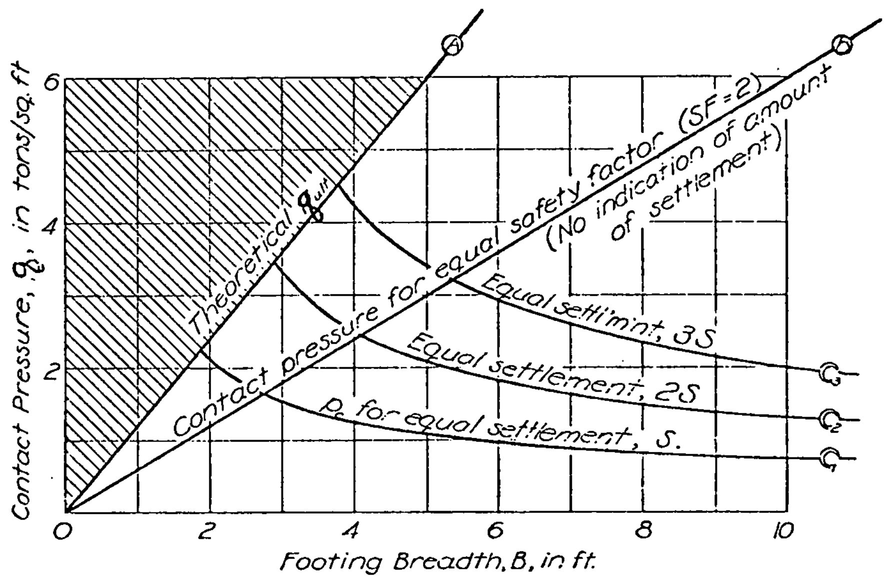


FIGURE 10 Relation of allowable pressure, q_a, to settlement, s, and ultimate bearing capacity, q_{ULT}, for cohensionless soils [after Hough (*14*)].

footing width, that is, the water table is below the zone of influence of the footing. For higher positions of the water table or for settlements other than 1 in., the allowable pressure, q_a, is estimated as

$$q_a = q_1 \frac{C_w}{s}$$

where

q_1 = allowable pressure obtained directly from Fig. 11
s = tolerable settlement, in inches,
$C_w = 0.5 + 0.5 \frac{D_w}{D_f + B}$ = water table correction ($C_w \leq 1$)
D_w = depth to water, and
D_f = depth of embedment.

Recent studies [e.g., (*24*)] have indicated that the curves of Figure 11 are conservative and likely to overestimate settlement by as much as 50 percent.

Several agencies estimate settlements in granular soils by the same procedures used for cohesive soils, that is,

$$s = H_0 \left(\frac{C_c}{1 + e_0}\right) \log \frac{p_0 + \Delta p}{p_0}$$

where

s = settlement,
H_0 = thickness of compressible zone,
p_0 = initial effective overburden pressure,
Δp = stress increase at mid depth of compressible zone owing to footing pressure, q_a,
C_c = compression index, and
e_0 = initial void ratio.

However, for cohesionless soils, a compressibility factor $C = (1 + e_0)/C_c$ is estimated from the adjusted blow count from standard penetration tests using the curves proposed by Hough (Fig. 7). Also, the stress increase, Δp, is likely to be computed by the approximate method of spreading the footing load uniformly over an area that increases with depth, that is,

$$\Delta p = q \left(\frac{B}{B + Z'}\right)^n$$

where

q = footing contact pressure
B = footing width
Z' = depth below footing for which Δp is to be computed, and
n = 1 for strip footings and 2 for square footings.

Numerous other methods have been proposed for estimating settlements of footings on cohesionless soils (*24*). The Schmertmann (*17, 25*) procedure uses strain influence factors, which have been developed from finite-element analyses of stresses in granular media, and static cone penetration test data. The subsoil beneath the foundation is divided into sublayers, and the settlement, s, is computed as

$$s = C_1 C_2 \, \Delta p \sum_{i=1}^{n} \left(\frac{I_z}{E_z} \Delta z\right)_i$$

where

Δp = net foundation pressure,
Δz = thickness of sublayer
I_z = average strain influence factor (Fig. 12) for the sublayer,

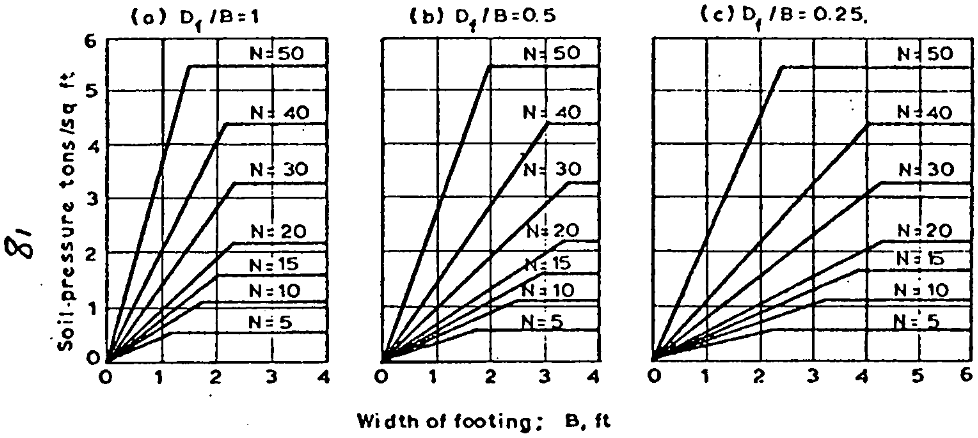


FIGURE 11 Allowable soil pressure, q, for limiting settlement of 1 in. in sands. N=blows/ft from SPT corrected for effective overburden pressure [after Peck et al. (*12*)].

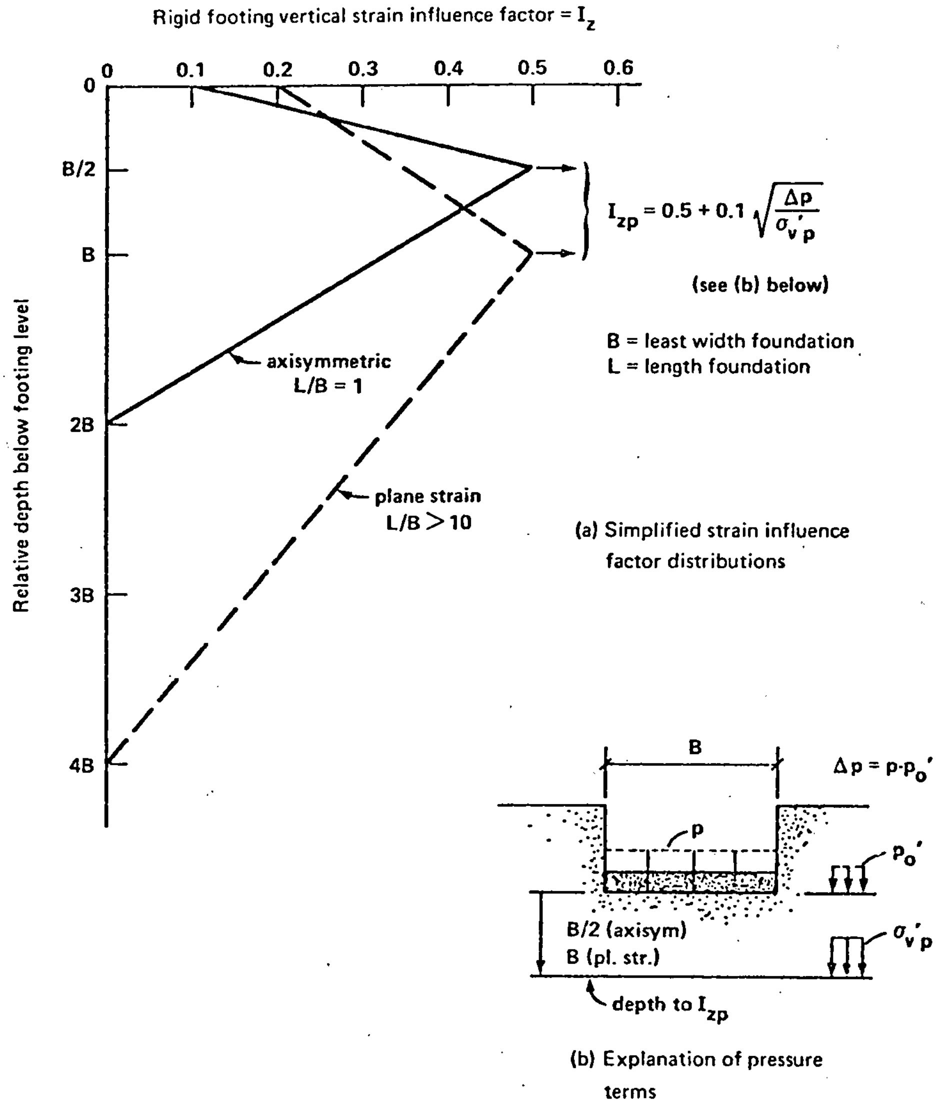


FIGURE 12 Modified strain influence factor diagrams for use in Schmertmann method for estimating settlement over sand [after Schmertmann (*17*)].

E_z = average elastic modulus for the sublayer = 2.5 to 3.5 q_c,
q_c = static cone penetration resistance,
C_1 = correction factor for embedment, and
C_2 = creep factor.

Schmertmann and others have proposed correlations between cone resistance and the blow counts from standard penetration tests so that the method can also be used with blow counts. Some of the correlations are shown in Figure 13. This method appears to provide reasonable estimates of settlement and it is likely to be used more commonly in the future, particularly as cone penetration tests become more common.

Most agencies also have tables of presumptive allowable pressures that are used when no test data are available. Examples are given in Table 4. It should be noted that presumptive values usually are very conservative and may eliminate the consideration of shallow foundations in some cases for which more detailed investigations may show that shallow foundations are feasible.

Compacted Soils

Allowable contact pressures on compacted fills may be evaluated in the same manner as for natural soils. However, the strength and compressibility properties are obtained from laboratory tests of compacted samples. Usually the compacted samples are soaked before testing to account for the critical effects of increased moisture. Of course, the density of the compacted sample should be representative of the compacted densities to be required in the field.

Normally, fills for support of structural foundations are compacted to at least 95 percent of the maximum dry density from an AASHTO T 180 Compaction Test (some agencies require 100 percent). This degree of compaction produces a good quality material that typically will behave as an overconsolidated soil. Compression of the fill may be analyzed by conventional consolidation methods, but usually it will be negligible except for very high fills. Perhaps more important is the analysis of the effect of the weight of the fill on underlying natural cohesive soils, which has been discussed previously.

Many agencies do not permit the construction of footings on compacted fills because of uncertainties regarding the quality and uniformity of fills. Usually these uncertainties arise from bad experiences with poorly compacted fills. However, with proper quality control, compacted fills may actually outperform natural soils. In some cases, good quality fills may spread foundation loads and reduce the unit pressures on underlying soft materials so that shallow foundations may become feasible. For example, Connecticut and Washington commonly use compacted granular glacial stream deposits for controlled fills over

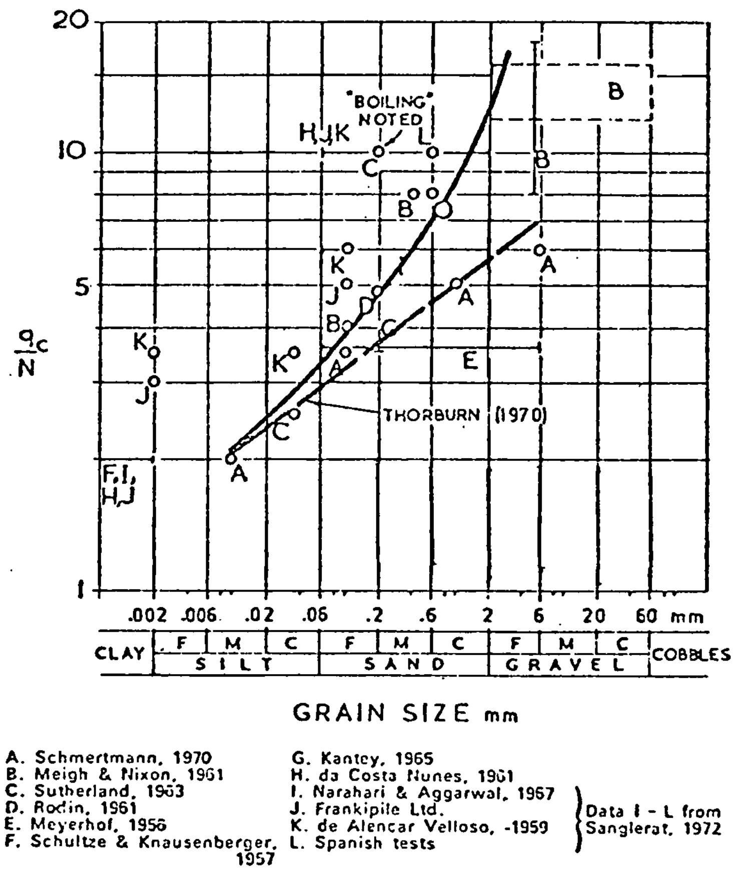


FIGURE 13 Correlation of q_c/N with grain size [after Jorden (*24*)].

TABLE 4

PRESUMPTIVE BEARING CAPACITIES FROM AASHTO AND BUILDING CODES, KSF.

Soil description	*Chicago, 1966*	*Natl. Board of Fire Underwriters, 1967*	*Atlanta, 1950*	*New York City, 1949*	*Uniform Bldg. Code, 1964*	AASHTO, 1978
Alluvial soils						1-2
Clay, very soft	.5					2-8
Clay, soft	1.5	3.0	2.0	2.0	1.5	
Clay, ordinary	2.5		4.0			
Clay, medium stiff	3.5	5.0			2.5	
Clay, stiff	4.5		4.0	3.0		
Clay, hard	6.0				8.0	
Sand, compact and clean	5.0					2-8*
Sand, compact and silty	3.0					
Inorganic silt, compact	2.5					
Sand, loose and fine		4.0		4.0	1.5	
Sand, loose and coarse, or sand-gravel mixture, or compact and fine		6.0		8.0	2.5	2-8*
Gravel, loose, and compact coarse sand		8.0	8.0	8.0	8.0	4-8
Sand-gravel, compact		12.0	12.0	12.0	8.0	
Hardpan, cemented sand, cemented gravel	12.0	20.0	20.0	24.0		10-20
Soft rock				16.0		> 10
Sedimentary layered rock (hard shale, sandstone, siltstone)		30.0	30.0			
Bedrock	200.0	200.0	200.0	120.0		

*For sand that is confined.

weaker natural soils. Allowable bearing pressures of 4 to 8 ksf (190 to 380 kPa) are used on the basis of experience.

Rock

Allowable bearing pressures for various types of rock usually are presumptive values based on local experience. The rock type and its quality are evaluated qualitatively from core samples. Rock quality may be quantified on the basis of the rock quality designation (RQD), which is the ratio of the total length of intact core recovered to the length of the core drilled (*13*). Only hard and sound pieces of core that are at least 4-in. (100-mm) long are considered. An empirical correlation between allowable bearing pressure and RQD is given in Table 5.

In a few instances, the allowable pressure is based on actual laboratory compression tests of core samples. Allowable pressures on rock incorporate large factors of safety with respect to the strength of intact rock because of the potential effects of various localized defects (e.g., joints, fissures, or slickensides). For example, the allowable bearing pressure, q_a, may be expressed as

$$q_a = K_{sp} q_u$$

where

q_u = unconfined compressive strength of a core sample, and
K_{sp} = coefficient, which depends on the size and spacing of discontinuities in relation to the footing width.

Typical values of K_{sp} range between 0.1 and 0.4. These conservative values usually are sufficient for the contact pressures required for spread footings, which generally will have a minimum size based on other considerations, such as the geometry of the structure to be supported.

Settlement of shallow foundations on rock typically is not a problem and is not investigated. Soft shales and some other highly weathered rocks may be exceptions that require consideration of settlement. In such cases, the materials are treated and analyzed as cohesive or cohesionless soils.

RELIABILITY OF SETTLEMENT PREDICTIONS

The reliability of settlement predictions depends on many factors including uncertainties in the applied loads, the relevant soil parameters, and the stress distribution within the foundation soils. For most structures, and particularly for highway bridges, the properties of the subsurface soils will be the major source of uncertainty. The precision of the soil property evaluation depends not only on the type and variability of the soils but also on the magnitude and quality of the sampling and testing program. Properties that are estimated on the basis of general soil classifications or index properties (e.g., natural water content, grain size, or plasticity index) will be much less reliable than values obtained from good quality laboratory or in-situ strength/deformation tests.

Assuming good quality sampling and testing, the predicted settlement for a shallow foundation on a uniform, normally consolidated clay of low sensitivity is likely to be within ± 25

TABLE 5

ALLOWABLE CONTACT PRESSURE, q_a, ON JOINTED ROCK [AFTER PECK ET AL. (*13*)].

RQD	q_a[a] (kips/sq ft)	q_a[a] (lb/sq in)
100	600	4170
90	400	2780
75	240	1660
50	130	970
25	60	410
0	20	140

[a] If tabulated value of q_a exceeds unconfined compressive strength q_u of intact samples of the rock, as it might in the case of some clay shales, for instance, take $q_a = q_u$.

percent of observed values. For stiff clays and dense sands, for which the predicted settlement is likely to be small [e.g., less than 1 in. (25 mm)], the percent error of the prediction may be larger but the absolute magnitude of the error is likely to be acceptable. Predictions will be much less reliable in loose sands, sensitive clays, partly saturated soils, residual soils, sand-clay mixtures, highly stratified deposits, expansive clays, and collapsible soils. In such materials, the error of settlement predictions may exceed 100 percent.

Experience has indicated that most established settlement calculation procedures are conservative. Loads are overestimated and conservative values of soil properties are selected. As a result, the settlement is more likely to be overestimated than underestimated. A recent comparison (*26*) of predicted and observed settlements for 148 shallow foundations on various types of soil illustrated this point. Eight of the nine prediction methods used in the study tended to overestimate settlement. The only exception was a method that is not used in the United States.

CHAPTER SIX

PERFORMANCE CRITERIA

The predicted behavior of the foundation system must be evaluated in relation to its impact on the performance of the structure that it supports. In general, the safety, function, and appearance of the structure must be considered. Also, maintenance costs should be weighed against initial construction costs.

A bearing capacity failure or the undermining of a footing owing to scour is very likely to produce a catastrophic collapse of all or a major portion of the structure. Loss of life, long-term loss of function, and high replacement costs are likely to result. Clearly, the foundation system must be extremely safe against these types of failure, and high factors of safety with respect to bearing capacity and scour are necessary. As noted previously, normal practice requires a factor of safety of 2.5 to 3 against bearing capacity failure.

The tolerable movement of a structural foundation is much more difficult to define. Differential movements between various points of a structure may induce stresses in structural members and thus generally are more significant than absolute movements. Very small differential movements may induce minor cracking in concrete components that affects appearance but not function or safety. Larger movements may interfere with the function of the structure and still larger movements may produce stresses that ultimately cause a major structural failure. Large but uniform movements will not cause cracking of concrete or overstressing of a structure but are likely to create discontinuities and disruptions of connections between the structure and its surroundings.

The acceptable magnitude of foundation movement depends on the nature, size, and function of a structure. The cracking of architectural elements (e.g., glass panels, plaster, etc.) or the functioning of windows, doors, or elevator shafts govern the tolerable movements for most residential, office, and commercial buildings. Such buildings are likely to be more sensitive to differential movements than are warehouses, mill buildings, or most highway structures where function and structural integrity are likely to govern.

For most highway structures, tolerable movements will be governed by their influence on the function of the structure. Thus, movements that produce minor cracking of concrete but do not affect function may not be of concern. For example, lateral movements of retaining structures do not affect a wall's ability to support the backfill and thus are not of concern unless roadways or other structures are supported on the backfill. Then, the tolerable movement of the roadway or structure will control the allowable movement of the wall. Differential movements of culverts are unimportant unless they are sufficiently large to disrupt flow gradients within the culvert or to induce settlement of pavement surfaces above the culvert. (However, the latter problem is more commonly caused by improper compaction in the backfill above the culvert rather than settlement of the culvert.) Similarly, movements usually are not critical to the foundations for signs.

Vertical and horizontal movements of bridge foundations may seriously affect the function of the bridge and thus are properly of major concern to bridge engineers. The rideability of the bridge deck may become poor and even dangerous at high speeds, and damage to abutments, superstructure, decks, sidewalks, and railings may result. Bearings may become misaligned or jammed. However, these defects develop gradually with foundation movements and it has been suggested that foundation movements become unacceptable only when the resulting defects become severe enough to require costly maintenance or repairs. TRB Committee A2K03 has proposed that intolerable movement be defined as follows: "Movement is *not* tolerable if damage requires costly maintenance and/or repairs *and* a more expensive construction to avoid this would have been preferable" (*33*). This approach to defining tolerable movements requires qualitative judgments, which will vary among transportation agencies. It also is recognized that tolerable movements are affected by many other factors, including the type, material, span length, and total length of the bridge. Thus, there currently are no widely accepted criteria for tolerable movements for bridge foundations.

TOLERABLE MOVEMENT CRITERIA

The AASHTO Bridge Specifications (*3*) do not specify tolerable movements for bridge piers and abutments supported on spread footings. However, the allowable load on piling is specified as 50 percent of the load that will produce ¼ in. (6 mm) of net settlement, and some agencies infer that the allowable movement of a footing should be of the same order of magnitude. This is an extremely conservative requirement, which would prohibit the use of spread footings on most soils. In practice, most transportation agencies will consider 1 in. (25 mm) of differential settlement as acceptable, and a few agencies will accept 2 in. (50 mm) as satisfactory. A recent summary of British practice (*27*) reports 1 to 2 in. as the typical limit of acceptable total settlement.

The British report also suggests that both continuous and simply supported decks may be designed for differential settlements of 1 in 800 (1 in./65 ft span) and notes that 1 in 200 (1 in./16 ft span) has been used occasionally for simply supported decks. The differential settlement is defined in relation to span length. This ratio commonly is referred to as the "angular distortion." For comparison, some typical allowable settlements, in terms of angular distortion, are shown for other types of structures in Table 6. Note that the value of 1/800 proposed for bridge decks is more stringent than any value given in Table 6, which is further evidence of the conservatism of the criteria for bridges.

FIELD OBSERVATIONS

Until recently very little field data have been available on which to establish more rational tolerable movement criteria for bridges. However, in 1978 a series of papers dealing with tolerable movements of bridges was published in Transportation Research Record 678. Most of the data in these reports were obtained from a 1975 survey conducted by TRB Committee A2K03. Keene (*29*) reviewed factors that influence tolerable movements and summarized seven case histories from Connecticut, which included examples of postconstruction settlements of 2 to 3 in. (50 to 75 mm) without detrimental effects. Keene also emphasized the importance of stage construction, that is, allowing the abutment to settle under the loads of the approach embankment before construction of the superstructure is started. Walkinshaw (*30*), on the basis of data from 35 bridges in ten western states, stated that poor riding quality was reported when settlement exceeded 2.5 in. (64 mm) although larger vertical movements could be tolerated without structural distress. He also stated that structural distress was reported where 2 in. of horizontal movement occurred. Grover (*31*) presented data from 79 bridges in Ohio and concluded that abutment settlements of 1 in. (25 mm) or less are tolerable; settlements of 2 to 3 in. will be noticeable to drivers but will cause only minor damage, if any, to structures; and settlements in excess of 4 in. (100 mm) will be objectionable to drivers and likely to cause damage to the bridge abutments and superstructure. He also suggested that maintenance would be necessary for settlements in excess of 4 in. and desirable for settlements of 3 to 4 in. Bozozuk (*32*) attempted to summarize the 1975 survey data and to suggest criteria for tolerable movements. Bozozuk's findings are illustrated in Figures 14 and 15, which indicate horizontal and vertical movements and whether the movements were reported as tolerable or intolerable. It should be noted that very large movements were reported for bridges supported on piles as well as for those on spread footings. Horizontal movements,

TABLE 6

LIMITING ANGULAR DISTORTION (*28*).

CATEGORY OF POTENTIAL DAMAGE	$\beta = \delta/\ell$
Danger to machinery sensitive to settlement	1/750
Danger to frames with diagonals	1/600
*Safe limit for no cracking of buildings	1/500
First cracking of panel walls	1/300
Difficulties with overhead cranes	1/300
Tilting of high rigid buildings becomes visible	1/250
Considerable cracking of panel and brick walls	1/150
Danger of structural damage to general buildings	1/150
*Safe limit for flexible brick walls, L/H > 4	1/150

*Safe limits include a factor of safety.

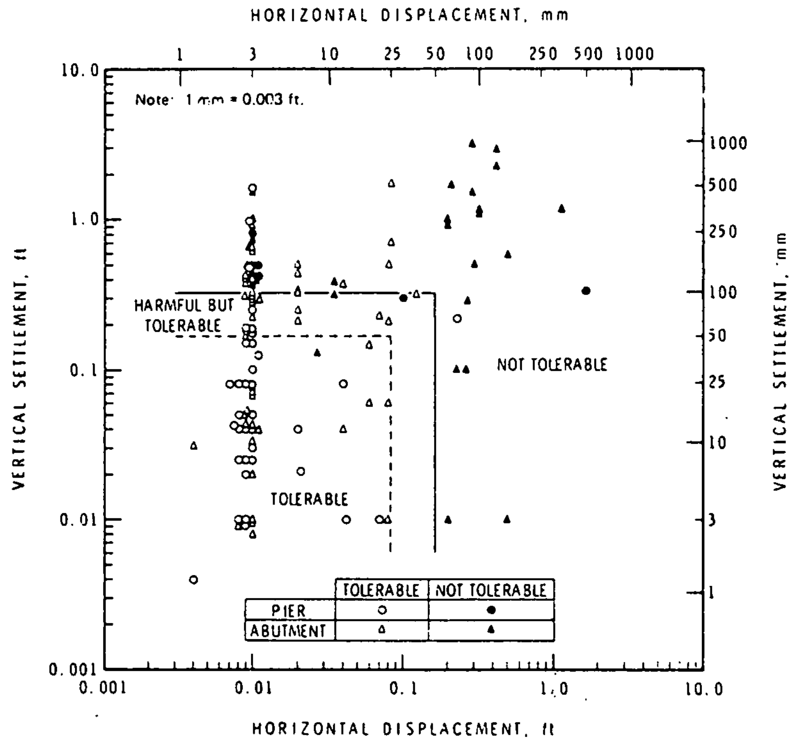


FIGURE 14 Engineering performance of bridge abutments and piers on spread footings (*32*).

he concluded, were more critical than vertical movements. He proposed that settlements of less than 2 in. are tolerable, from 2 to 4 in. are harmful but tolerable, and in excess of 4 in. are not tolerable. For horizontal movements, he proposed that less than 1 in. are tolerable, 1 to 2 in. are harmful but tolerable, and in excess of 2 in. are not tolerable.

Most recently a very extensive study of bridge movements has been conducted by West Virginia University under the sponsorship of FHWA (*33*). Data on movements and damage, if any, for 204 bridges on both spread footings and pilings have been reviewed. The bridge movement data that were reported in Transportation Research Record 678 have been re-examined in more detail and additional case histories have been obtained. Several different analyses were attempted. First, movements were studied in relation to substructure and superstructure variables. The effects of soil type, abutment type, foundation type, approach embankment height, span type, and structural material were considered. Second, the types of structural damage associated with various magnitudes of horizontal and vertical movements were assessed. Third, the tolerance of various bridges to movements was studied in relation to the type of structure and foundation. Finally, analytical studies were conducted to evaluate the effect of differential vertical movements on stresses in continuous two- and four-span concrete and steel bridges of various span lengths.

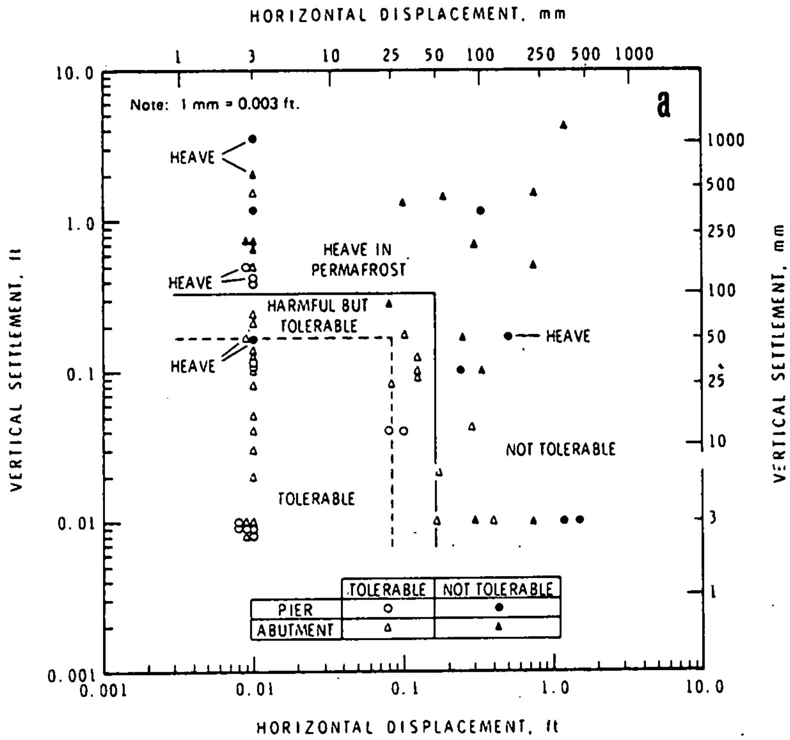
Some of the results of this study are summarized in Tables 7–10. Table 7 indicates the number of abutments and piers for which vertical and/or horizontal movements were observed. Table 8 shows the range, average, and standard deviation of the movements observed for each category of substructure and foundation listed in Table 7. Some extremely large movements are reported. The average vertical movement of abutments was in excess of 4 in. (100 mm), regardless of whether the abutment was supported on spread footings or piles, and the average lateral movement was greater than 2.5 in. (60 mm). For piers, the average vertical movements are slightly smaller than for abutments, whereas the lateral movements are larger.

Damage surveys were reported for 171 bridges. Twenty-five experienced no structural damage, 83 experienced damage that was considered tolerable by the agency responsible for the bridge, and 63 experienced damage that was intolerable and required repairs. The most commonly reported tolerable damage included relatively minor cracking of concrete, opening or clos-

TABLE 7

SUMMARY OF OBSERVED MOVEMENTS, WEST VIRGINIA UNIVERSITY STUDY (*33*).

Substructure and Foundation Type	No. Observed	Number that moved			
		Total	Vert.	Horiz.	H. & V.
Abutments					
Spread footings	190	162	150	38	26
Piles	172	114	71	76	33
Total	362	276	221	114	59
Piers					
Spread footings	242	104	94	17	7
Piles	456	90	69	29	8
Total	698	194	163	46	15



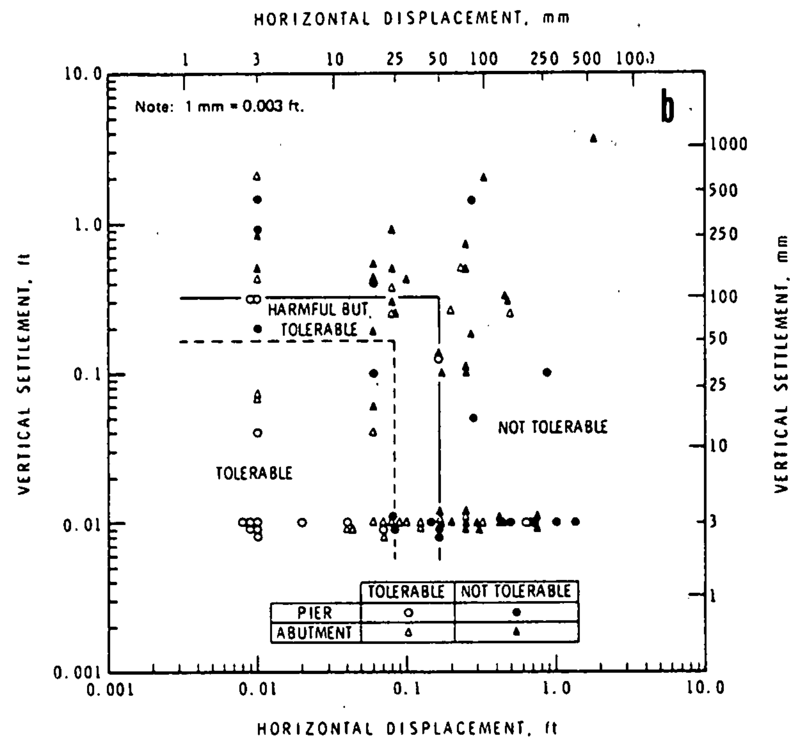


FIGURE 15 Engineering performance of bridge abutments and piers on (a) friction and (b) end-bearing piles (*32*).

TABLE 8

MAGNITUDES OF OBSERVED MOVEMENTS, WEST VIRGINIA UNIVERSITY STUDY (*33*).

Substructure Element	Foundation	Movement	No.	Range (in.)	Average (in.)	Std. Dev. (in.)
Abutment	Spread Footings	Vertical	150	0.1 - 35.0	4.2	5.6
		Horizontal	38	0.1 - 8.8	2.5	2.1
		Vertical &	26	0.1 - 35.0	9.3	9.8
		Horizontal		0.1 - 8.0	2.4	2.0
	Piles	Vertical	71	0.1 - 50.4	5.2	8.1
		Horizontal	76	0.5 - 14.4	2.9	2.4
		Vertical &	33	0.3 - 50.4	6.4	10.7
		Horizontal		0.5 - 14.4	3.1	2.9
Pier	Spread Footings	Vertical	94	0.1 - 42.0	2.1	4.9
		Horizontal	17	0.5 - 20.0	3.3	5.0
		Vertical &	7	0.8 - 9.0	3.8	2.6
		Horizontal		0.6 - 20.0	4.9	7.3
	Piles	Vertical[a]	69	0.1 - 14.0	3.8	3.0
		Horizontal	29	0.1 - 16.0	3.1	3.9
		Vertical &	8	0.3 - 13.7	3.0	4.6
		Horizontal		0.6 - 4.0	1.6	1.2

[a]Number of piers on piles with movement includes 7 piers that raised vertically. These are not included for vertical movements.

ing of construction joints in abutments, and cracking or spalling of concrete decks. Intolerable damage included inward movement of abutments, jamming of beams or girders against abutments, closing of expansion joints in the deck, and damage to bearings. Surprisingly, poor riding quality was reported for only 12 of the bridges, but 11 of these were severe enough to be regarded as intolerable. These 11 bridges were subjected to very large movements and angular distortions, which exceeded 1/130 in all cases and averaged 1/50 (1 in./4 ft of span).

Table 9 shows the range of movements for which bridge damage was judged tolerable or intolerable. Table 10 shows the range of angular distortions for simple and continuous span bridges. Several significant observations can be drawn from these tables:

1. For bridges experiencing only vertical movements, (a) only tolerable damage, if any, was observed in 61 of 64 cases for which differential settlement was less than 4 in. (100 mm), and (b) intolerable damage was observed in 10 of 17 cases for which differential settlement exceeded 4 in.

2. For bridges experiencing only horizontal movements, (a) only tolerable damage, if any, was observed in 5 of 6 cases for which the horizontal movement was less than 2 in. (50 mm), and (b) intolerable damage was experienced in 15 of 19 cases for which the horizontal movement exceeded 2 in.

3. For bridges experiencing both horizontal and vertical movements, intolerable damage occurs more frequently for significantly smaller movements than for cases in which only vertical or horizontal movements were reported. The critical

TABLE 9

RANGE OF MOVEMENT MAGNITUDES CONSIDERED TOLERABLE OR INTOLERABLE, WEST VIRGINIA UNIVERSITY (*33*).

Interval[a] in Inches	Number of Bridges With the Given Type of Movement							
	Vertical Only		Horizontal Only		Vertical and Horizontal			
					Vertical Component		Horizontal Component	
	Tolerable	Intolerable	Tolerable	Intolerable	Tolerable	Intolerable	Tolerable	Intolerable
0.0 - 0.9	30	0	2	0	7	0	7	0
1.0 - 1.9	21	0	5	1	6	3	6	7
2.0 - 3.9	10	3	1	7	6	4	5	8
4.0 - 5.9	1	1	2	0	2	4	0	7
6.0 - 7.9	3	2	1	3	0	2	0	1
8.0 - 9.9	0	4	0	3	0	1	0	2
10.0 - 14.9	2	1	0	2	0	4	0	2
15.0 - 19.9	1	2	0	0	0	2	0	1
20.0 - 60.0	0	0	0	0	0	4	1	0
Total	68	13	11	16	21	24	19	28

[a]For vertical movements, magnitudes refer to maximum differential vertical movement. For horizontal movements, magnitudes refer to maximum horizontal movement of a single foundation element.

TABLE 10

RANGES OF MAGNITUDES OF LONGITUDINAL ANGULAR DISTORTION CONSIDERED TOLERABLE OR INTOLERABLE, WEST VIRGINIA UNIVERSITY STUDY (*33*).

Angular Distortion Interval ($\times 10^{-3}$)	Number of Bridges of the Given Type and Tolerance					
			Span Type			
	All Bridges		Simple		Continuous	
	Tolerable	Intolerable	Tolerable	Intolerable	Tolerable	Intolerable
0 - 0.99	30	1	15	1	12	0
1.0 - 1.99	18	2	7	0	9	1
2.0 - 2.99	10	0	4	0	2	0
3.0 - 3.99	7	0	5	0	1	0
4.0 - 4.99	4	2	2	0	0	2
5.0 - 5.99	0	2	0	1	0	1
6.0 - 7.99	2	4	1	2	1	2
8.0 - 9.99	1	1	0	1	1	0
10.0 - 19.99	3	9	2	3	1	3
20.0 - 39.9	1	5	1	4	0	1
40.0 - 59.9	0	0	0	0	0	0
60.0 - 79.9	0	2	0	1	0	1
Total	76	28	37	13	27	11

movements appear to be approximately half of the values reported for one-directional movement.

4. The observations from the West Virginia study tend to support the tolerance criteria suggested by Bozozuk.

The angular distortion data in Table 10 suggest that values less than 1/200 (0.005) and 1/250 (0.004) will be tolerable for simple and continuous spans, respectively. These values are significantly larger than the British values, which were quoted earlier. However, they appear very consistent with the suggested values for buildings, which are given in Table 6.

In summary, it is very clear that the tolerable settlement criteria currently used by most transportation agencies are extremely conservative and are needlessly restricting the use of spread footings for bridge foundations on many soils. Angular distortions of 1/250 and differential vertical movements of 2 to 4 in. (50 to 100 mm), depending on span length, appear to be acceptable, assuming that approach slabs are used or other provisions are made to minimize the effects of any differential movements between abutments and approach embankments. Finally, horizontal movements in excess of 2 in. appear likely to cause structural distress. The potential for horizontal movements of abutments and piers should be considered more carefully than is done in current practice. This will require development of new methods for prediction of lateral movements.

CHAPTER SEVEN

GROUND MODIFICATION

When analyses indicate that spread footings will not perform satisfactorily on natural ground, most transportation agencies immediately elect to use deep foundations. However, sometimes it will be more economical to improve the natural soil so that spread footings can be used. The basic approaches to soil improvement have been recognized for many years and include:

- Removal and replacement
- Drainage
- Densification
- Cementation
- Reinforcement
- Drying or desiccation
- Heating

Although the basic concepts are very old, many new and innovative construction techniques have been developed to implement these concepts. Examples include deep dynamic compaction of cohesionless soils, soil reinforcement, and the utilization of geotextiles. Mitchell (*34*) has presented an excellent state-of-the-art review of current soil improvement techniques and their potential applications in civil engineering.

Transportation agencies traditionally have used soil improvement methods primarily for roadway and embankment construction but less commonly in conjunction with structural foundations. However, when these procedures are used for construction of approach embankments, they will affect the performance and hence the feasibility of spread footings for bridge abutments. Therefore, some of the more significant procedures for treatment of foundation soils will be reviewed.

Undercutting of unsuitable materials and replacement with controlled compacted fill is common on roadway cuts and fills. Undercutting also may be used for relatively shallow zones beneath proposed footings. The availability of good quality fill usually will control whether it is more economical to lower the footing to sound bearing material or to replace the unsuitable material with compacted fill. In Connecticut, where high quality granular deposits are prevalent, clay layers may be undercut partially or completely and replaced with controlled granular fill to provide support for footings for bridge abutments (*29*). Also, stub abutment footings may be placed on granular embankments rather than extending them to natural ground. The granular fills beneath footings must be well graded and contain only minimal fines, usually less than 10 percent minus 74 μm (No. 200) sieve material. Densities equivalent to 95 to 100 percent of Standard Proctor density are required. Compaction usually is achieved most efficiently with vibratory compactors using loose lifts of 12 to 18 in. (300 to 450 mm). Similar procedures also have been used extensively in Washington (*1*).

Several methods are available for in-place densification of deep loose cohesionless deposits (*34*). All utilize vibratory forces for densification. Heavy vibratory rollers may provide effective compaction to depths of 4 to 6 ft (1.2 to 1.8 m). For deeper deposits, vibrocompaction methods may be used. A hollow vibratory probe is driven to the desired depth of treatment usually with the aid of water or air jets. Granular backfill is added as the vibrating probe is removed so that a compacted cylinder of sand remains. Because the radius of influence of the process is relatively small, the probe usually must be used at spacings of 3 to 10 ft (1 to 3 m). Vibrocompaction has been shown to be effective routinely to depths of 60 ft (18 m), and in a few instances compaction has been achieved to depths of 100 ft (30 m). However, the method generally is ineffective for deposits that contain more than 20 to 25 percent fines. Dynamic compaction is a relatively new method in which weight of 2 to 200 tons (1.8 to 180 Mg) is raised as high as 100 ft (30 m) and dropped on the surface. Several blows are used at points spaced 6 to 10 ft (1.8 to 3 m) apart. The depth of influence depends primarily on the impact energy of the tamper. Treatments to depths in excess of 100 ft have been reported. In recent years this method has been introduced in the United States and it has now been used on highway projects in several states, including Arkansas, Colorado, Montana, and New Mexico.

Preloading with or without sand drains is a well-established procedure for the treatment of soft cohesive soils. The method is particularly well suited for use in conjunction with embankment construction and has been used by many transportation agencies. The embankment is constructed and a waiting period is established to allow consolidation of the soft foundation soils. Vertical drains may be installed in the foundation soil to accelerate the rate of consolidation. When the consolidation is completed, final grading of the embankment is started. More details on preloading and sand drains are found in NCHRP Synthesis 29 (*35*). In recent years, a variety of small prefabricated drains have been developed. These drains typically are 4-in. (100-mm) wide by ¼-in. (6-mm) thick and consist of a plastic core surrounded by cardboard, fibrous fabric, or porous plastic. They can be installed rapidly to depths in excess of 100 ft (30 m) with the aid of special mandrels. Specific drains and the related installation equipment are manufactured by several firms. A review of consolidation with prefabricated drains has been presented by Hansbo (*36*). Experience in Europe, Japan, and, more recently, the United States indicates that prefabricated drains are rapidly replacing sand drains.

The use of precompression of soft soils under approach embankments is of considerable importance to the consideration of spread footings for abutments. As noted previously, settlement of an abutment footing is affected significantly by the embankment loads. If this settlement can be removed by loading before construction of the abutment and superstructure, the postconstruction settlements are more likely to be tolerable. Keene (*29*) cities several examples of the application of this concept in Connecticut in which settlements ranging from 2 to 14 in. (50 to 350 mm) were eliminated by the preloading process.

Soil reinforcement techniques utilize tensile reinforcing members embedded in granular fill to form a retaining structure. The basic concepts of soil reinforcement systems were developed by Vidal in France. The key components of a ground reinforcement system are the facing, reinforcement, and backfill, which are shown schematically in Figure 16. The facing, which may be metal or precast concrete, is supported by thin galvanized steel reinforcing strips that extend horizontally into the backfill. Typically, the length of the reinforcing strips is 0.7 to 0.8 times the height of a retaining structure and proportionally greater for an abutment, which is subject to high surcharge loads. Granular backfill is required, and the reinforcing strips are placed in the backfill as it is constructed. The frictional resistance between the reinforcing strips and the backfill is used to develop the tensile force in the strips that is required for lateral support of the facing. Specific design procedures are available to determine the tensile forces in the reinforcing strips and the size, spacing, and length of the strips. Also the overall stability of the composite system must be considered. Design services are provided by the manufacturers and constructors of various proprietary soil reinforcement systems.

An earth-reinforced structure is flexible and can withstand deformations without distress. Thus, it can be used at sites where relatively large movements are anticipated. Because the wall does not require a footing, it can be constructed at the property line, which may be an advantage when right-of-way is limited. These characteristics often will make construction of an earth-reinforced structure more economical than a conventional retaining wall, particularly at difficult or restricted sites. Also, when a soil reinforcement system is used to support a bridge abutment, differential settlement between the bridge deck and the approach fill is eliminated or minimized.

The principal applications of soil reinforcement systems are for moderate to high highway retaining structures in regions of difficult terrain or restricted right-of-way. However, a significant number of bridge abutments supported on soil reinforcement systems have been constructed in the United States and elsewhere. The original Vidal system, which is constructed by the Reinforced Earth Company, has been used on at least 40 abutments in the United States. A typical Reinforced Earth abutment design used in New York is illustrated in Figure 17. The subsurface profile at one site where this design was used is shown in Figure 18.

Variations of the Vidal-type wall are available from other manufacturers. Fabric, wire mesh, and metal facings are used in combination with plastic, fiber-glass, geotextile, or wire-mesh reinforcing. Although the basic design concepts are similar for these systems, details will vary with the stiffness and durability

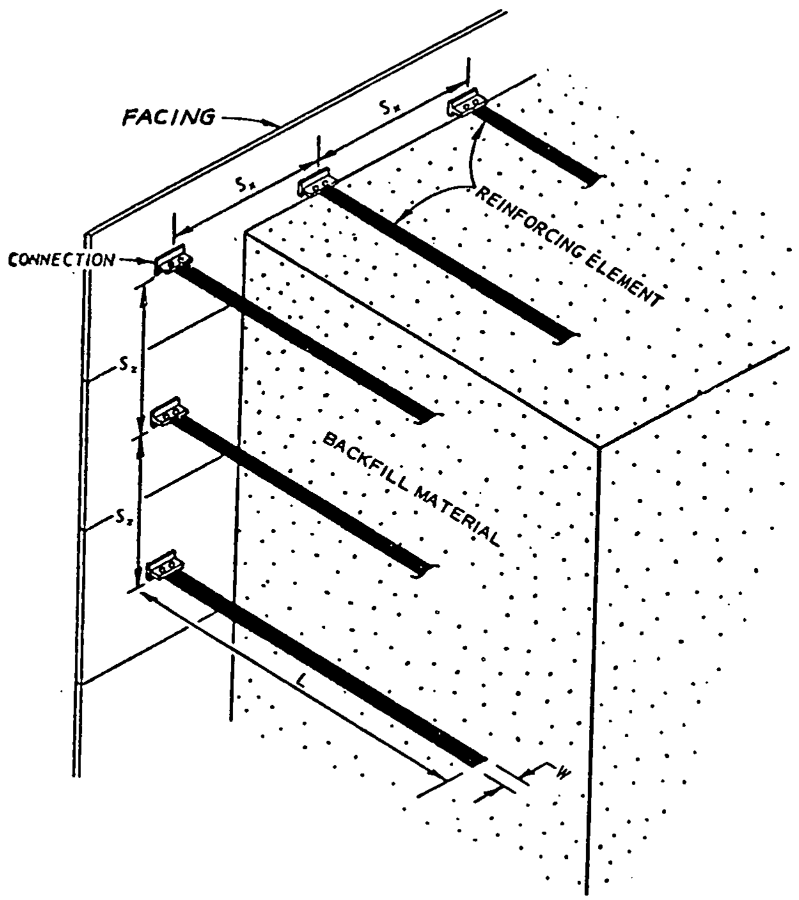


FIGURE 16 Schematic of major elements of a soil reinforcement system.

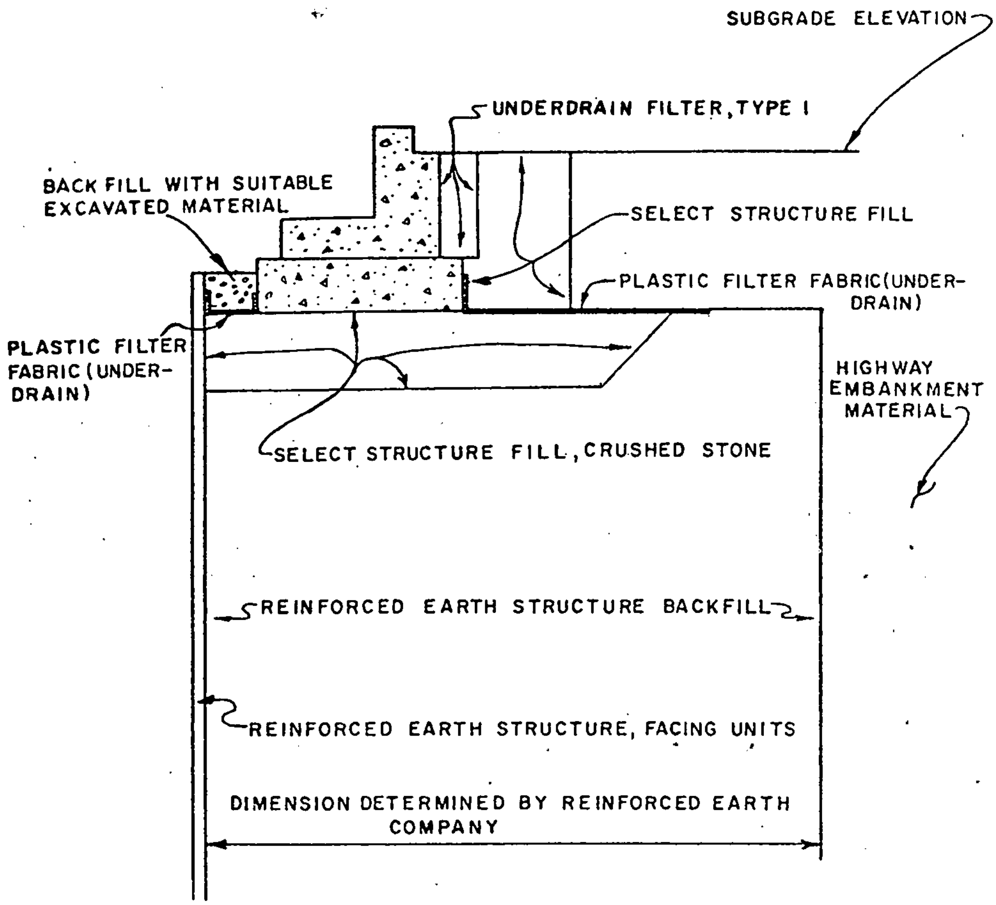


FIGURE 17 Typical Reinforced Earth abutment design (New York State DOT).

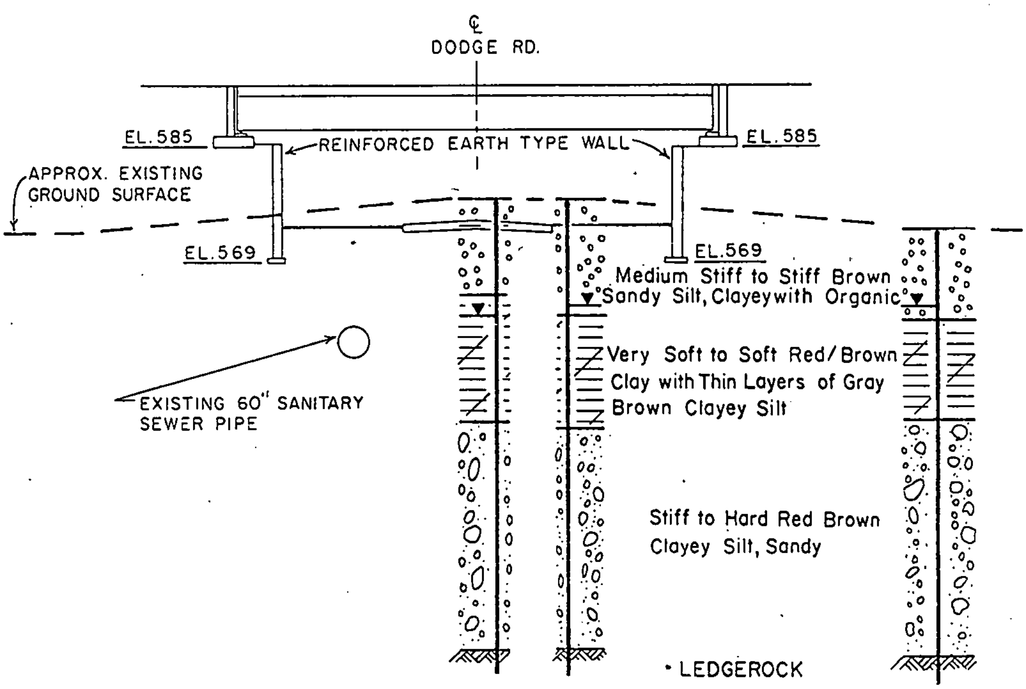


FIGURE 18 Section and profile at Reinforced Earth abutment.

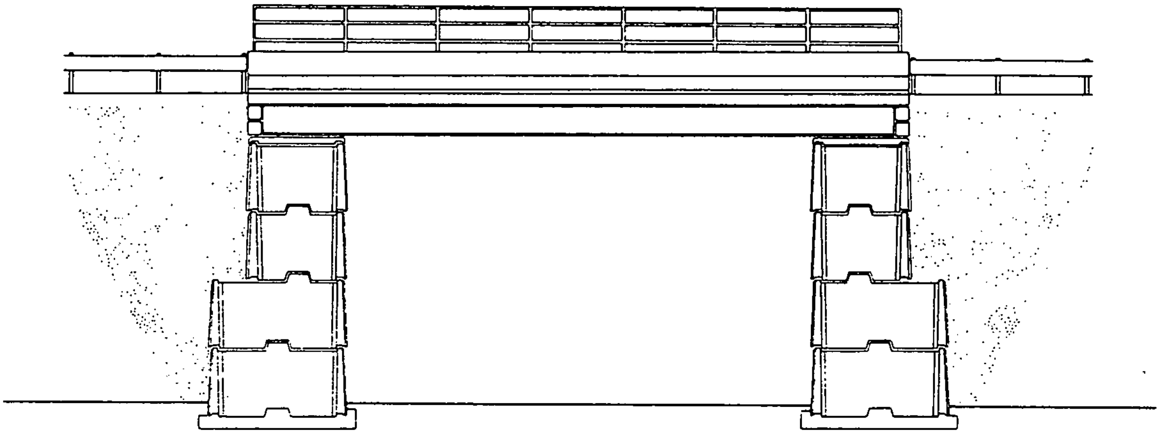


FIGURE 19 Doublewal bridge abutments.

of the reinforcing material. Doublewal is another example of a patented retaining wall system. It consists of precast, interlocking, cellular reinforced concrete elements. The elements may be installed rapidly, and the interior is backfilled with compacted free-draining material. Doublewal now is being incorporated in some retaining wall designs in Connecticut and several other states. The manufacturers suggest that the system is also appropriate for small bridge abutments, such as illustrated in Figure 19.

Stone columns are used for reinforcing soft fine-grained soils. The construction procedure is similar to the vibrocompaction process, which has been described previously. A cylindrical hole is constructed with a vibratory probe that is installed under its own weight, often with the aid of water jets. Gravel backfill is added, compacted and displaced radially into the soft clay to form a column 2 to 3 ft (0.6 to 0.9 m) in diameter. Larger diameters are developed in softer clays. Column lengths of 60 ft (18 m) have been achieved. The stone column functions much like a pile foundation, except that it is more compressible. However, pile caps and structural connections are not required. Construction techniques for stone columns were developed in Germany and Japan, and they have not been used extensively in the United States. Stone columns apparently have not been used to support bridge abutments. However, they were used to support an embankment and a Reinforced Earth wall at Lake Pend Oreille, Idaho where right-of-way restrictions made conventional embankment procedures prohibitive (see Fig. 20). Additional research on stone columns currently is underway and U.S. applications are increasing (*37*).

The preceding ground modification techniques appear to have some potential for increasing the use of shallow foundations for bridge abutments. Many other techniques, including various uses of geotextiles, root piles, soil nailing, and grouting, appear to have less applicability to foundations for highway structures.

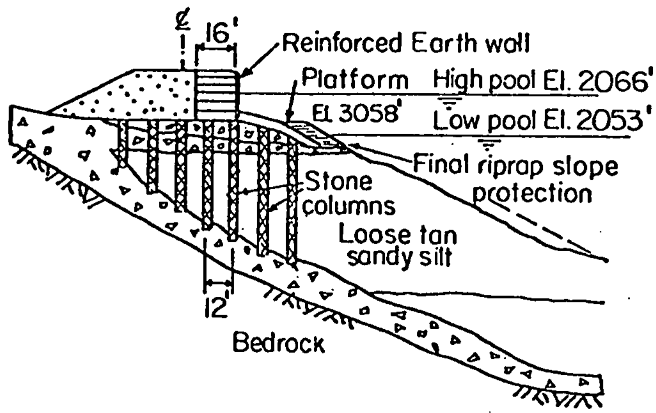


FIGURE 20 Reinforced Earth and stone columns, Lake Pend Oreille, Idaho.

CHAPTER EIGHT

CASE HISTORIES

COST-EFFECTIVENESS

New York

In the mid 1960s, the New York State Department of Transportation developed foundation proposals for two major railroad viaduct projects on Long Island. The subsurface conditions at the Seaford viaduct of the Wantagh-Seaford project consist of a shallow zone of cinders, sand, and silt underlain by approximately 50 ft (15 m) of medium-dense sand with some gravel. This sand is underlain by gray silt and sand. The Long Island Railroad wanted to support the viaducts on piles, but the New York DOT recommended shallow foundations on the basis of the settlement analyses, which are illustrated in Figure 21. Settlements were estimated at less than 1 in. (25 mm) for dead loads and 1¼ to 1 ½ in. (32 to 38 mm) for dead load plus live load and impact effects. Spread footings were constructed, and the resulting dead load settlements generally have been less than ½ in. (13 mm). It has been estimated that $3,000,000 was saved on the Wantagh-Seaford project by using spread footings instead of piles. Similar subsurface conditions and predicted and observed settlements were reported for the Amityville-Lindenhurst project. Although cost estimates have not been presented for this project, the savings are assumed to be even greater than for the Wantagh-Seaford project because of the greater length of the Amityville-Lindenhurst project.

Connecticut

The Connecticut DOT has constructed highway structures on spread footings for many years. In 1959 and 1966 surveys were undertaken to evaluate the performance of bridges supported on spread footings. Observed settlements were compared with settlements predicted during the design of each project. Damages, if any, were assessed and used to evaluate the effects of the observed settlements on the structures (*29*).

A total of 27 bridges and 6 large box culverts were studied. Almost all of the bridges were composed of 1 to 3 simply supported spans. The 15 older bridges, which were constructed between 1941 and 1955, typically were designed with full-height abutments and embankment heights of 20 to 25 ft (6 to 8 m). The embankments were placed after the substructures were completed and frequently were placed after the superstructures were in place. The 12 bridges constructed between 1958 and 1963 typically used perched abutments supported on spread footings in the approach embankments. Usually, compacted select gravel fill was placed beneath the footings. The abutments and superstructure were constructed after the embankment, and often the abutment construction was delayed for 3 to 6 months to allow time for settlements that were due to the fill to be completed. For three cases for which the fill height was 40 ft (12 m), surcharge loads also were used.

All structures were located in the general vicinity of Hartford and underlain by varved clays of glacial lake beds. A typical subsurface profile is shown in Figure 22. The clay strata ranged in thickness from a few feet to more than 200 ft (1 to 60 m). The thickness exceeded 25 ft (8 m) at all but 2 or 3 sites. The clay strata were overconsolidated, with the preconsolidation pressure typically 3 to 4 ksf (140 to 190 kPa) greater than the current effective overburden pressure. At many locations, the varved clays were capped with sand deposits.

The Connecticut surveys produced the following observations:

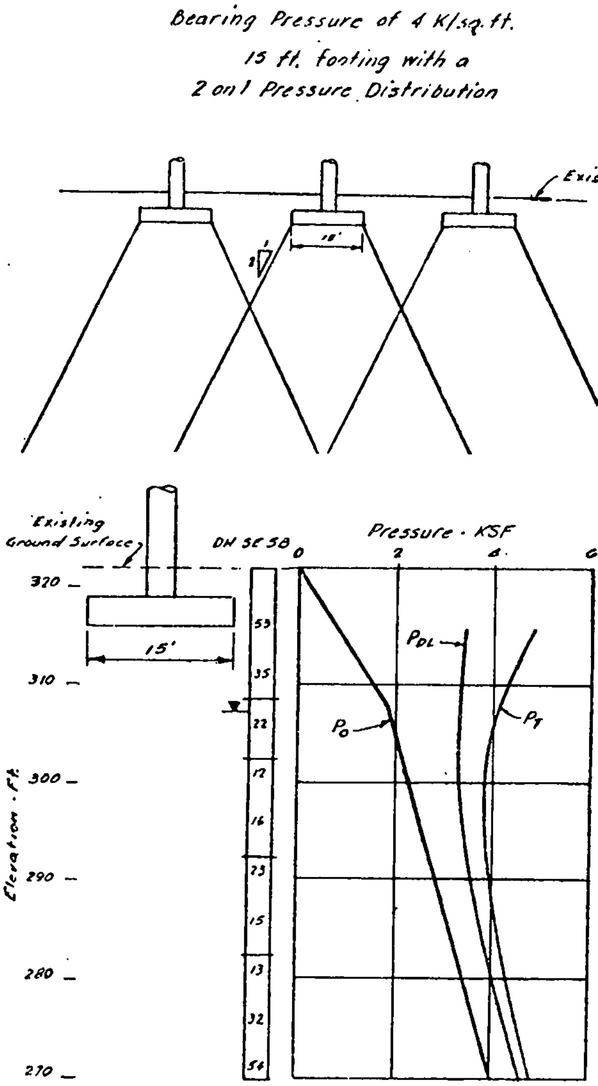
- Observed settlements ranged from ¼ to 8 in. (6 to 200 mm).
- Observed settlements and the settlements estimated during design agreed closely, usually with ± 20 percent.
- When preloads and waiting periods were used, as much as 10 to 14 in. (250 to 350 m) of settlement was completed before construction of abutments.
- Damage was limited to movements at expansion joints and minor hairline cracks.
- No maintenance or repairs have been required, aside from adding mastic filler to a few expansion joints.
- The savings in cost resulting from using spread footings instead of piles for the 33 structures is estimated as $4,500,000, based on construction costs at the time of construction.

Washington

The costs of spread footings and piling for support of bridge abutments have been compared at three sites in Washington (*1, 38*). At each site, perched abutments were supported in the fill on spread footings and the interior piers were supported on piles. As a consequence, reliable bid prices were available for both foundation types at each site. Subsurface conditions and analyses of the performance of spread footings are not described for the projects. However, because spread footings were used, it must be assumed that the predicted performance was satisfactory at each site.

Pilchuck River Bridge

An eight-span continuous prestressed concrete girder bridge was constructed across the Pilchuck River in 1979. The abut-



		Method 1 **						Method 2 ***					
			Dead Load		Live Load Impact				Dead Load		Live Load Impact		
Blows *	Layer Thickness - H	$C = \frac{1+e}{C_c}$	$\log \frac{P_{DL}}{P_o}$	Settlement S_{DL}	$\log \frac{P_T}{P_{DL}}$	Settlement S_{LL+I}	Total Settlement - S_T	E_s	ΔP_{DL} (TSF)	Settlement S_{DL}	ΔP_{LL+I}	Settlement S_{LL+I}	Total Settlement - S_T
35	7	144	.436	.021	.128	.006	.027	460	1.04	.016	.58	.009	.025
22	6	96	.232	.014	.082	.005	.019	300	.68	.014	.34	.007	.021
14	10	74	.147	.020	.059	.008	.028	210	.45	.021	.22	.015	.036
19	10	88	.092	.010	.044	.005	.015	270	.34	.013	.19	.007	.020
32	13	132	.066	.006	.026	.003	.009	410	.29	.009	.13	.004	.013
Total - Ft.				.071		.027	.098			.073		.042	.115
Total - in.				.85		.32	1.17			.88		.50	1.38

* Number of Blows per foot of penetration required to drive a 2" O.D. sampler, using a 140 lb. hammer falling 30 inches.

** From "Compressibility as the Basis for Soil Bearing Value" by D. K. Hough, A.S.C.E. Proceedings, August, 1959.

*** From "Standard Penetration Test and Compressibility of Soils" by E. Schultze and E. Menzenbach, Proceedings, 5th. Int. Conf. Soil Mechanics and Foundation Engineering.

FIGURE 21 Settlement analysis for the Seaford railroad viaduct (New York).

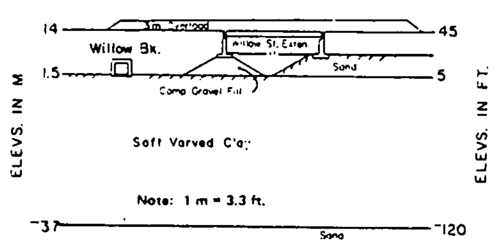


FIGURE 22 Typical Connecticut River Valley profile (29).

ment foundations were designed both as spread footings in the fill and as 12HP53 steel H piles. A schematic diagram of the embankment and abutments is shown in Figure 23. The spread footing design required construction of the embankment to full height, a delay period of 20 to 30 days, and then excavation for the footing construction. At one abutment, a 7-ft (2.1-m) thick zone of unsuitable material was removed and replaced with compacted granular material. The pile design required 22 piles with an average length of 47.5 ft (14.5 m) at one abutment location and 18 piles with an average length of 41.6 ft (12.7 m) at the other abutment location. The comparative costs based on

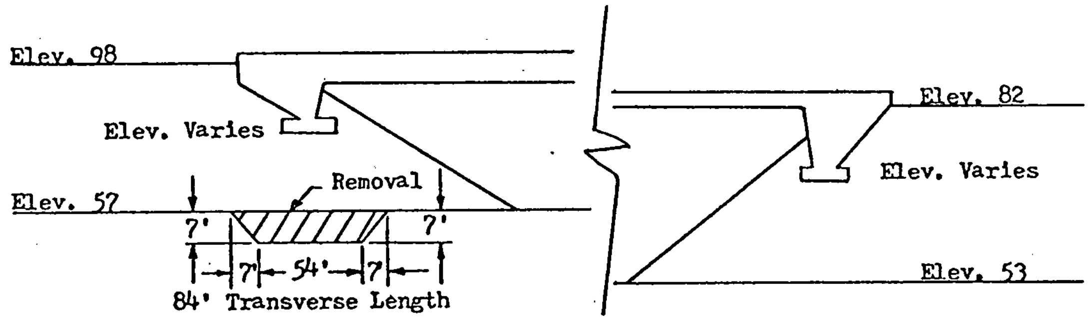


FIGURE 23 Pilchuck River Bridge, Washington (*1*).

actual bids for the project are shown in Table 11. The estimated cost of the spread footing alternate was only 54 percent of the estimated pile foundation cost. The selection of spread footings resulted in a savings of more than $26,000 on this project.

Ellingston Road Crossing

This three-span continuous prestressed concrete girder bridge was designed and constructed in 1976. The project included two abutments (called piers 1 and 4) and two interior piers. The general subsurface profile at the abutment locations is shown in Figure 24. Because the construction schedule included a long delay period, most of the settlement caused by the embankment loads was completed before the abutment construction was scheduled to start. The abutment foundations were designed as spread footings supported in the fill with an allowable footing pressure of 6 ksf (290 kPa). The abutment contained jacking pads as a precaution against additional settlement. The interior pier foundations were designed as 55-ton (490-kN) cast-in-place concrete piles. On the basis of the pile design for the interior piers, it was estimated that a pile foundation for each abutment would require 13 of these piles with an average length of 65 ft (20 m). The actual bid price for the spread footings was approximately one-third of the estimated cost of the pile system, as shown in Table 11. A savings of more than $14,000 was realized through the selection of the spread footing design.

TABLE 11

ABUTMENT FOUNDATION COSTS (WASHINGTON) (*1*).

Bridge Project	Year	Costs ($) Spread Footings	Piles	Savings
Pilchuck River Bridge	1979			
Abutment 1		18,919	32,367	13,448
Abutment 2		12,429	25,358	12,929
Total		31,348	57,725	26,377
Ellingston Road Crossing	1976	7,200	21,900	14,700
North Ft. Lewis Interchange	1969			
Northbound structure		4,961	14,052	9,091
NCD structure		4,042	11,330	7,288
Total		9,003	25,382	16,379

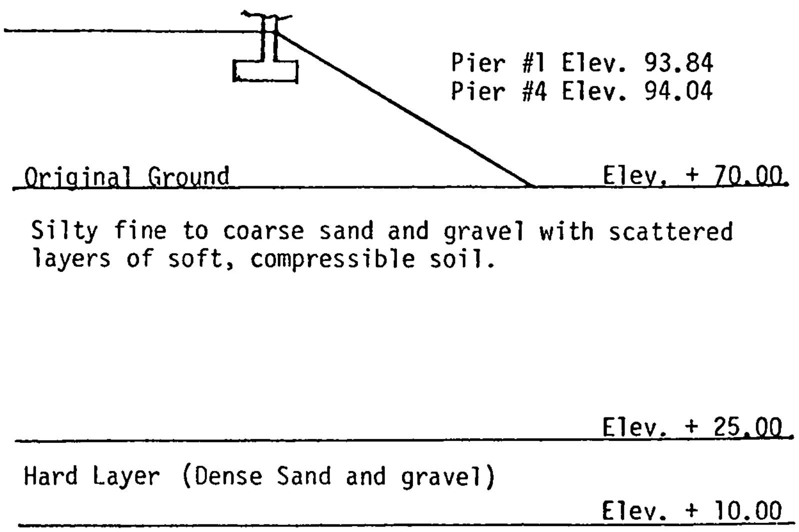


FIGURE 24 Profile of Ellingston Road Bridge approach embankment, Washington (*1*).

FIGURE 25 North Fort Lewis Interchange structure, Washington (*1*).

North Fort Lewis Interchange Overcrossing

Cost comparisons were prepared for two adjacent structures, which were constructed at this site in 1969. The "Northbound" structure is a three-span continuous prestressed girder bridge, which carries the northbound lanes of Interstate 5. The "NCD" structure is a northbound collector-distributor ramp (Fig. 25). The abutments of both structures were designed to be supported with 12HP53 steel H piles, whereas the interior piers were to be supported on spread footings on natural ground using an allowable bearing pressure of 6 ksf (290 kPa). A total of 44 piles, including 4 test piles, were to be driven, 12 per abutment for the Northbound structure and 10 per abutment for the NCD structure. The estimated pile length was 25 ft (8 m) at all four abutments. The bid prices for the pile foundations are shown in Table 11. After the piles were driven for one abutment of the Northbound structure, it was decided to delete the remainder of the piles. The pile cap was simply used, without modification, as a spread footing at each of the other three abutments. Thus, the actual savings resulted only from the deletion of pile material and driving costs. This amounted to a total savings of $6,976, $2,616 for the Northbound structure and $4,360 for the NCD structure.

Seguirant (*38*) notes that the pile caps are larger than the required size of spread footings for the allowable soil pressure at the site. Thus, additional savings would have been realized if the original foundation had been designed for spread footings. The cost of spread footings designed for a very conservative soil pressure of 4 ksf (190 kPa) is compared with the pile foundation cost in Table 11, using 1969 unit bid prices for this project. The spread footing design would have provided savings of more than $16,000 in comparison with the cost of the original pile design and savings of $9,000 from the actual cost of the as-built foundations.

Ontario, Canada

Shields et al. (*39*) have presented some comparative cost data for construction of stub abutments on piles and spread footings in Ontario. The designs are shown in Figure 26. The analysis assumes that the spread footing and the pile cap are of equal size so that the cost differential is limited solely to the difference in the cost of the piling and that of the select granular fill, which is required beneath the spread footing. The pile system consists of 38 steel pipe piles, 12.8 in. (325 mm) outside diameter and 30-ft (18-m) long with a design load of 25 tons (220 kN) per pile. The granular fill is assumed to be 10-ft (3-m) thick. Cost comparisons are shown in Table 12 for the costs of granular fill in various parts of Ontario province. In all cases, there is a savings in selecting the spread footing. An actual abutment design used in 1978 is shown in Figure 27.

TABLE 12

COST COMPARISON FOR ABUTMENT FOOTINGS: TYPICAL HIGHWAY UNDERPASS STRUCTURE IN ONTARIO (*39*).

Ontario Location	On Piles ($/bridge)	On Spread Footing ($) Per Cubic Meter of Fill	Per Bridge	Saving ($/bridge)
Southwest	17 900	10.80	16 650	1 250
East	16 400	5.25	8 800	7 600
Central	16 950	7.85	12 300	4 650
North	20 000	3.90	7 000	13 000

Notes: 1 m³ = 1.31 yd³.
Costs in 1976 Canadian dollars (1.00 1976 Canadian dollar = 1.01 1976 U.S. dollars).

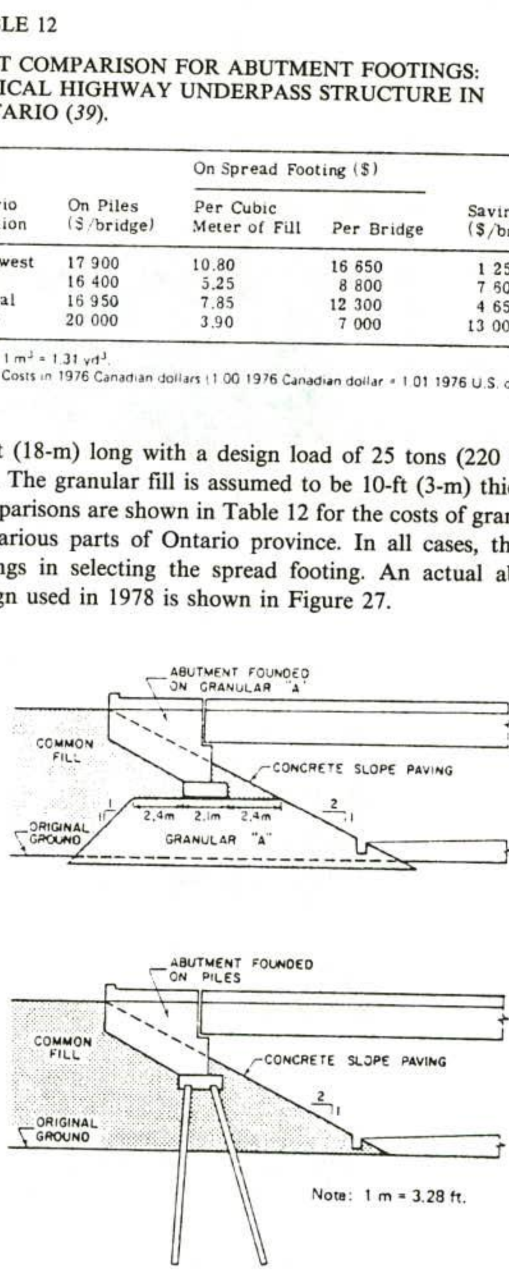
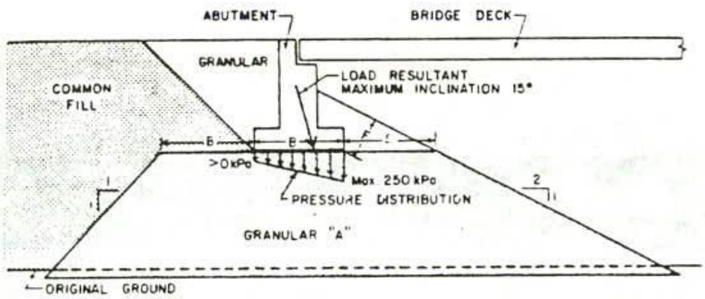


FIGURE 26 Basis for cost comparison, Ontario (*39*).



s depends on minimum frost depth F.

Note: 1 kPa = 6.89 lbf/in^2.

FIGURE 27 New (1978) abutment footing design, Ontario (*39*).

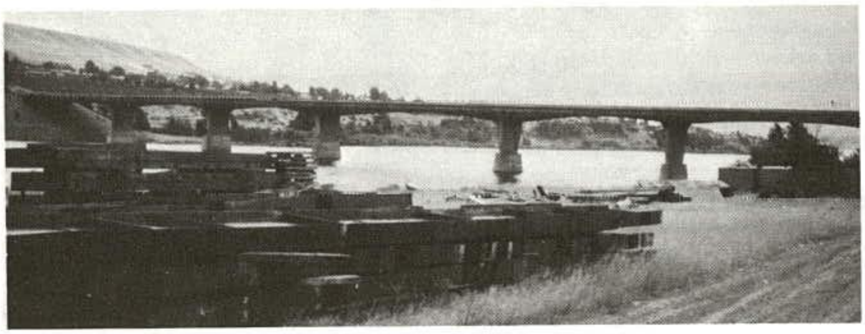


FIGURE 28 Columbia River Bridge at Olds, Washington, a seven-span concrete box girder bridge with both abutments on spread footings (*1*).

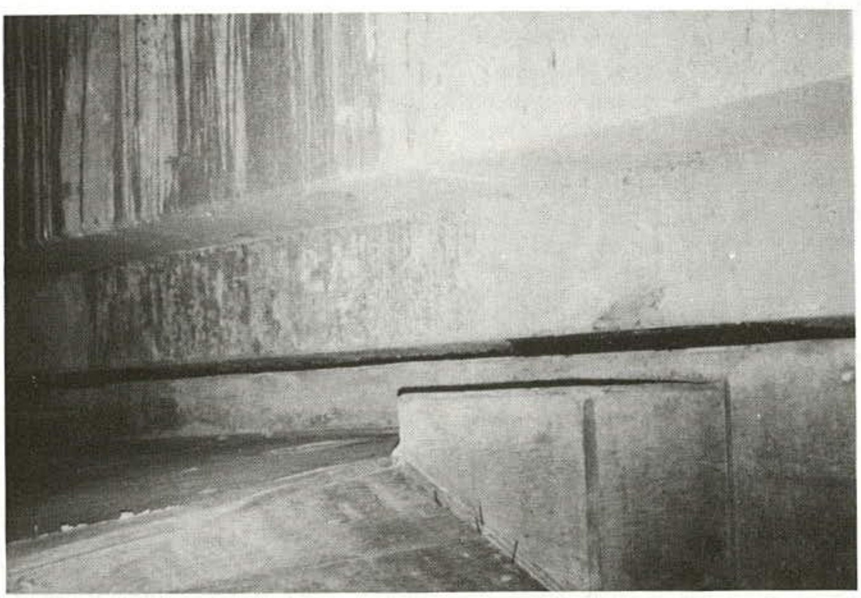


FIGURE 29 Jacking pad at abutment of Columbia River Bridge (*1*).

DESIGN MODIFICATIONS TO FACILITATE USE OF SPREAD FOOTINGS

In most current U.S. practice, the superstructure, substructure, and foundation are designed as separate units rather than as a single system. The superstructure is designed first and tolerable movements are defined. Then the substructure is designed and, finally, the foundations are designed to provide the previously established requirements of the superstructure and substructure. This practice provides a safe design but not necessarily the most economical one. If the superstructure, substructure, and foundations were designed as a single system, it is possible that inexpensive modifications of the structure might allow significant savings in the foundations and thus provide a more economical design. For example, design modifications that make the superstructure more tolerant of movements may allow the use of spread footings where more expensive deep foundations otherwise would be required.

The use of simple spans instead of multi-span continuous members, adjustable bearing and rocker systems, and jacking pads are among the features that will increase the tolerance of the superstructure to foundation movements.

An abutment with jacking pads may provide a particularly economical alternative to deep foundations in cases for which excessive long-term settlement of an abutment on spread footings is predicted. If the abutment settles beyond the tolerance limits of the superstructure, hydraulic jacks are inserted on the jacking pads and used to raise and shim the superstructure. It is reported that typically an abutment can be jacked in 2 or 3 days at a cost of a few thousand dollars (*1*).

The Columbia River Bridge at Olds, Washington, which was constructed in 1975, is a good example of the application of jacking pads on abutments (Fig. 28). The bridge is a seven-span box girder with simply supported end spans and continuous intermediate spans. The intermediate piers are supported on drilled shafts; however, the abutments were placed on spread footings in the approach fills after it was estimated that pile lengths of nearly 200 ft (60 m) would have been required. Simple end spans and jacking pads on the abutments were incorporated into the design at relatively little cost. A jacking pad is illustrated in Figure 29.

The east abutment has been jacked twice for a total of 0.34 ft (0.10 m) and additional jacking may be required if settlement continues at the present rate. The west abutment has not required jacking as of 1983. The abutment movements have not affected the performance of the structure.

CHAPTER NINE

FINDINGS AND RESEARCH NEEDS

FINDINGS

This synthesis presents the foundation engineering practices that are required for adequate consideration of shallow foundations for highway structures. The investigation has identified the current transportation agency practices that encourage, as well as those that hinder, the use of shallow foundations. The major findings of the study are summarized as follows:

1. Significant cost savings may be realized through increased use of shallow foundations. Specific examples of the relative costs of spread footings and deep foundations are cited.
2. Current criteria for tolerable movements of bridge foundations are very conservative. These criteria, which are more stringent than those commonly used for building foundations, unduly restrict the use of spread footings for bridge foundations. Recent investigations (*33*) indicate that the current tolerable movement criteria can be relaxed without affecting the safety and performance of highway bridges.
3. A thorough geotechnical investigation by a competent geotechnical engineering staff is required for adequate consideration of shallow foundations. The investigations should include adequate field exploration, laboratory testing, and appropriate analyses of the response of the foundation to design loads and environmental factors.
4. The state of the art of the geotechnical engineering profession for evaluation and prediction of the performance of shallow foundations is more advanced than the current practices of many transportation agencies. Significant recent advances in the state of the art include (a) improved methods for characterizing and evaluating the engineering properties of geotechnical materials, particularly using new in-situ techniques for evaluating materials that are difficult to sample, and (b) new analytical techniques, many of which are based on numerical analyses and utilization of computers.
5. New and innovative ground improvement techniques are being developed very rapidly. Soil reinforcement systems, deep dynamic compaction, stone columns, and drainage by prefabricated wick drains are among the techniques that are now available to make in-situ conditions more suitable for shallow foundations. Many of these methods already have been used on highway projects and they are likely to be used more frequently in the future.
6. Relatively inexpensive modifications of the superstructure, which can make the structure more tolerant of movements, sometimes may provide substantial savings in foundation costs. Close interaction between structural and geotechnical designers, which would allow the foundation and superstructure to be designed as a single system rather than as separate units, is likely to produce the most economical overall design.

RESEARCH NEEDS

Although this review has indicated some significant advances in the state of the art of geotechnical engineering for shallow foundations, it also has identified areas in which additional research is needed. In general, the development of sophisticated mathematical models of soil behavior has progressed more rapidly than the ability to determine appropriate numerical values of the soil parameters required in these models. Thus, the greatest research needs are related to site characterization and the evaluation of the engineering properties of soil and rock. There also are research needs in analytical methods, ground improvement techniques, and design procedures. The primary research needs may be summarized as follows:

1. Site characterization.
   - Improved exploratory methods for identification of subsurface conditions.
   - Application of seismic, other geophysical, and remote sensing techniques to evaluation of the spatial variability of subsurface conditions.
   - Improved procedures for obtaining undisturbed samples of a wide variety of soil types.
2. Evaluation of engineering properties of soil and rock.
   - Improvement of existing and development of new in-situ devices for direct evaluation of the engineering properties of soil and rock.
   - Experimental verification of the reliability of in-situ test methods.
   - Experimental studies of the effects of stress history, initial state of stress, anisotropy and degree of saturation on the stress-deformation behavior of soils.
   - Experimental studies of the fundamental engineering behavior of various "problem" soils, such as residual soils, cemented sands, stiff fissured clays, calcareous soils, and collapsible soils.
3. Analytical methods and studies.
   - Mathematical models for prediction of lateral movements of bridge abutments.
   - Mathematical models for evaluating soil-structure interaction effects between approach fills and bridge abutments.
   - Rational analyses for many ground improvement systems.
   - Probabilistic studies of the reliability of settlement predictions.
4. Field studies of selected bridges supported on shallow foundations.
   - Verification of anaiytical procedures for prediction of performance.
   - Evaluation of the effectiveness of various ground improvement systems.
5. Improved design concepts and procedures.
   - Revision of design criteria for the tolerable movement of bridge foundations.
   - Applicability of new soil improvement techniques to the support of bridge abutments on shallow foundations.
   - Develoment of structural design modifications that will increase the tolerable movement of the superstructure.
   - Use of shallow foundations in bridge rehabilitation projects.

Many of these research topics are already under investigation or consideration by various organizations, including FHWA.

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