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1 January 2001

US Army Corps
of Engineers

ENGINEERING AND DESIGN

Geotechnical Investigations

ENGINEER MANUAL

AVAILABILITY

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CECW-ET	**DEPARTMENT OF THE ARMY** **U.S. Army Corps of Engineers** **Washington, DC 20314-1000**	EM 1110-1-1804

Manual
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Engineering and Design
GEOTECHNICAL INVESTIGATIONS

1. Purpose. This manual establishes criteria and presents guidance for geotechnical investigations during the various stages of development for both civil and military projects.

2. Applicability. This manual applies to all USACE Commands having either military or civil works responsibilities.

3. Distribution Statement. This manual is approved for public release; distribution is unlimited.

4. Discussion. Geotechnical investigations are made to determine those geologic, seismologic, and soils conditions that affect the safety, cost effectiveness, design, and execution of a proposed engineering project. Because insufficient geotechnical investigations, faulty interpretation of results, or failure to portray results in a clearly understandable manner may contribute to costly construction changes, postconstruction remedial work, and even failure of a structure, geotechnical investigations and subsequent reports are an essential part of all civil engineering and design projects.

FOR THE COMMANDER:

ROBERT L. DAVIS
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EM 1110-1-1804, dated 29 February 1984, and EM 1110-1-1906, dated 30 September 1996.

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Chapter 1
Introduction

1-1. Purpose

This manual establishes criteria and presents guidance for geotechnical investigations during the various stages of development for civil and military projects. The manual is intended to be a guide for planning and conducting geotechnical investigations and not a textbook on engineering geology and soils exploration. Actual investigations, in all instances, must be tailored to the individual projects.

1-2. Applicability

This manual applies to all USACE Commands having either military or civil works responsibilities. The objective of Corps of Engineers Engineer Manuals (EM)[1] is to contain engineer and design technical guidance that will provide essential technical direction and application within the COE. However, an EM cannot provide the designer with two of the most vital tools essential to successful completion of a project: experience and judgement. Engineers and geologists who are just beginning their careers are strongly encouraged to seek the advice of more experienced members of their organization.

1-3. References

Standard references pertaining to this manual are listed in Appendix A. Military Standards (MIL-STD), Army Regulations (AR), Technical Manuals (TM), Engineer Regulations (ER), Engineer Manuals (EM), Engineer Pamphlets (EP), and Engineer Technical Letters (ETL) are identified in the text by the designated Government publication number or performing agency. Additional reading materials are listed in the Bibliography and are indicated throughout the manual by the principal author's last name and date of publication. Publications may be downloaded from the internet at the Corps' web page (www.usace.army.mil/inet/usace-docs/).

1-4. Background

Geotechnical investigations are performed to evaluate those geologic, seismologic, and soils conditions that affect the safety, cost effectiveness, design, and execution of a proposed engineering project. Insufficient geotechnical investigations, faulty interpretation of results, or failure to portray results in a clearly understandable manner may contribute to inappropriate designs, delays in construction schedules, costly construction modifications, use of substandard borrow material, environmental damage to the site, postconstruction remedial work, and even failure of a structure and subsequent litigation. Investigations performed to determine the geologic setting of the project include: the geologic, seismologic, and soil conditions that influence selection of the project site; the characteristics of the foundation soils and rocks; geotechnical conditions which influence project safety, design, and construction; critical geomorphic processes; and sources of construction materials. A close relationship exists between the geologic sciences and other physical sciences used in the determination of project environmental impact and mitigation of that impact. Those individuals performing geotechnical investigations are among the first to assess the physical setting of a project. Hence, senior-level, experienced personnel are required to plan and supervise the execution of a geotechnical investigation. Geotechnical investigations are to be

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[1] A list of acronyms and abbreviations is included as Appendix E to this manual.

carried out by engineering geologists, geological engineers, geotechnical engineers, and geologists and civil engineers with education and experience in geotechnical investigations. Geologic conditions at a site are a major influence on the environmental impact and impact mitigation design, and therefore a primary portion of geotechnical investigations is to observe and report potential conditions relating to environmental impact. Factors influencing the selection of methods of investigation include:

a. Nature of subsurface materials and groundwater conditions.

b. Size of structure to be built or investigated.

c. Scope of the investigation, e.g., feasibility study, formulation of plans and specifications.

d. Purpose of the investigation, e.g., evaluate stability of existing structure, design a new structure.

e. Complexity of site and structure.

f. Topographic constraints.

g. Difficulty of application.

h. Degree to which method disturbs the samples or surrounding grounds.

i. Budget constraints.

j. Time constraints.

k. Environment requirements/consequences.

l. Political constraints.

1-5. Scope of Manual

Increasingly, geotechnical investigations are conducted to evaluate the condition of existing projects as part of Operations and Maintenance. This type of investigation places special constraints on the methods which may be used. These constraints should be kept in mind by the designer.

a. General. Geotechnical investigations for roads and airfields are not discussed. Geotechnical investigations at construction sites may involve exposure to hazardous and toxic waste materials. In cases where such materials are recognized, geotechnical investigators should contact the Mandatory Center for Expertise for assistance. It is of note that many of the techniques and procedures described in this manual are applicable to hazardous, toxic, and radioactive waste (HTRW) work. Geotechnical aspects of HTRW site assessment are discussed in Construction Site Environmental Survey and Clearance Procedures Manual (Draft), EM 1110-1-4000, Walker (1988), and Borrelli (1988).

b. Types of detailed discussions. Chapter 2 provides guidance on geotechnical investigations appropriate to various stages of project development. Chapter 3 provides for implementation of initial, regionally oriented geotechnical investigations. Chapter 4 provides guidance for field procedures for surface investigations. Chapter 5 provides guidance on subsurface investigation procedures. Chapter 6 describes procedures for large-scale, prototype investigations, and Chapter 7 describes laboratory procedures for characterizing geotechnical properties of materials. Appendices and subject matter covered are: Appendix B, details for geologic mapping of construction areas; Appendix C, geologic mapping of tunnels and shafts; Appendix D, examples of drilling logs; Appendix F, soil sampling; and Appendices G and H, penetration resistance testing. Appendix F includes the modified version of the engineering manual on soil sampling. Information on soil sampling is also contained in Appendix C of EM 200-1-3. The text references specific sections of the soil sampling EM where appropriate. Guidance is in general terms where methodologies are prescribed by industry standards and described in accessible references. Where descriptions are otherwise unavailable, they are provided herein. The manual intends to provide general guidance to geotechnical investigation; because of the variability that exists among Corps of Engineers (COE) Districts or Divisions, it is advisable that each district and division prepare separate field investigations manuals. The manual should highlight procedures and formats of presentation that are preferred for geotechnical investigations within that organization. These manuals should be consistent with applicable EM.

Chapter 2
Scope of Investigations

2-1. Background

From project conception through construction and throughout the operation and maintenance phase, geotechnical investigations are designed to provide the level of information appropriate to the particular project development stage. In most instances, initial geotechnical investigations will be general and will cover broad geographic areas. As project development continues, geotechnical investigations become more detailed and cover smaller, more specific areas. For large, complex projects, the geotechnical investigation can involve highly detailed geologic mapping such as a rock surface for a structure foundation. The scope of the various increments of investigation are described in the following paragraphs. Although some material is presented in detail, rigid adherence to an inflexible program is not intended. It is the responsibility of the geotechnical personnel in the field operating activities to design individual geotechnical investigations to the particular project requirements and local conditions. However, there are minimum requirements for geotechnical investigations to be performed as part of the project development stages, and this manual serves to outline these basic standards. All geotechnical investigations should be planned and conducted by the district element having geotechnical design responsibility. No geotechnical investigation should be contracted out unless the district geotechnical design element reviews and approves the scope of work.

Section I
Civil Works Projects

2-2. Reconnaissance and Feasibility Studies

a. Purpose. Reconnaissance studies are made to determine whether a problem has a solution acceptable to local interests and is in accordance with administrative policy. If so, reconnaissance studies provide information to determine whether planning should proceed to the feasibility phase (ER 1110-2-1150). Feasibility studies identify and evaluate the merits and shortcomings of environmental, economic, and engineering aspects of the proposed project. Planning guidelines for conducting these studies are contained in ER 1105-2-100. Guidelines on engineering activities during feasibility and preconstruction planning and engineering studies are provided in the 1110 series of publications.

b. Scope of geotechnical investigations. Geotechnical investigations during planning studies should be designed to provide information at a level such that critical geotechnical features of candidate sites may be compared in the feasibility study report. These investigations should be sufficiently complete to permit selection of the most favorable site areas within the regional physical setting, determine the general type of structures best suited to the site conditions, evaluate the influence of hydrogeology on site design and construction, assess the geotechnical aspects of environmental impact, and to ascertain the costs of developing the various project plans in sufficient detail to allow comparative cost estimates to be developed.

c. Investigation steps. Planning-level geotechnical investigations are generally performed in two parts: development of regional geology and initial site investigations. The regional geology investigations are carried out during early stages of the study. Initial field investigations begin after the regional studies are sufficiently detailed to identify areas requiring geotechnical clarification.

(1) Development of regional geology. Figure 2-1 is a schematic diagram showing the steps involved and data needed to evaluate the regional geology of a site. Knowledge of the regional geology is essential to preliminary planning and selection of sites and to interpretation of subsurface exploration data. With the exception of fault evaluation studies, determination of seismicity and preliminary selection of the design earthquake are performed in conjunction with evaluation of the regional geology. Much of the data needed for describing the regional geology and for determining seismicity are identical, and therefore, the efforts can be combined. Engineering seismology requirements for more in-depth studies of tectonic history, historical earthquake activity, and location of possible active faults are a logical extension of the regional geologic studies. Requirements for conducting earthquake design and analyses, including geological and seismological studies, are contained in ER 1110-2-1806.

(a) Utilization of remote sensing information in assessing the regional geology of a site can greatly increase effectiveness and reduce time and costs. Commonly, a series of remote sensing images, taken at various times, are available for a site. Remote sensing images for evaluating regional geology generally include both aerial photographs and satellite images. Remote sensing analysis can be used to evaluate geomorphic characteristics and geologic structure; map soils, sediment sources, and transport directions; and monitor and evaluate environmental impacts. Use of remote sensing for geological investigations is discussed by Gupta (1991). Remote sensing applications in desert areas are discussed by Rinker et al. (1991). Principal sources of remote sensing imagery include the U.S. Geological Survey (USGS) at 605-594-6151 (or edcwww.cr.usgs/content_products.html on the computer Internet, or World Wide Web) and U.S. Department of Agricultural (USDA) Farm Security Agency at 801-975-3503.

(b) Compiled and properly interpreted regional geologic and field reconnaissance information should be used to formulate a geologic model for each site. The use of a Geographic Information System (GIS) is highly recommended for developing this model. A GIS provides a platform in which to digitally store, retrieve, and integrate diverse forms of geo-referenced data for analysis and display. A GIS can be thought of as a high-order map that has the capability of distilling information from two or more map layers (Star and Estes 1990; Environmental Systems Research Institute (ESRI) 1992). Application of GIS to geotechnical studies enhances data management with respect to project planning/design, field work strategy, map/statistical generation, and identification/correlation of important variables. The application of a GIS to a project depends on the size and complexity of the project and the availability of usable data. Judgement is needed to evaluate the benefits of a GIS versus the high cost of initial construction of the model. A project GIS can be used by all parties (e.g., designers, engineers, geologists, archaeologists) in all phases of a project from site selection to postconstruction operations and maintenance.

(c) Whether done using a GIS or more traditional methods, the geologic model will be revised during successive investigation stages and thereby provide the information necessary to determine the scope of initial field investigations. Procedural information on the steps required to develop the regional geology and perform field reconnaissances is contained in Chapter 3.

(2) Initial field investigations. Figure 2-2 details the general procedures for initial field investigations. Areal extent of the investigations is determined by the size and nature of the project. However, each site investigation should provide information on all critical geotechnical features that influence a site. Procedural information for conducting a surface field investigation is presented in Chapter 4, and for subsurface field investigations in Chapter 5. Major projects, such as dams and reservoirs, electric generating plants, and locks and dams, require comprehensive field investigations. Procedures for carrying out such detailed investigations are discussed in Chapter 6. Areal and site geotechnical mapping allow early modification of initial geologic models and tentative layouts for

CIVIL WORKS FEASIBILITY STUDIES
GEOTECHNICAL INVESTIGATIONS
DEVELOPMENT OF REGIONAL GEOLOGIC MODEL

DATA COLLECTION

Interagency Coordination and Cooperation

Sources of geologic, hydrologic, and soils data; insight into geologic hazards and HTRW problems; seismicity; construction materials; prior regional experience.

Survey of Available Information

Information similar to that obtained in interagency coordination; published data on material properties; geologic conditions and history; known hazards; ground water studies.

Map Studies

Formation descriptions and contacts; soil types and distributions; gross structure; fault locations; landforms, watersheds, slope landslides; locations of springs, quarries, etc.

Remote Sensing Studies

Landforms; watersheds; lineaments; soil- and rock-type boundaries; outcrops; seeps; sinkholes; erosion features; vegetation, etc.

Field Reconnaissances

Ground truth for remote sensing; outcrop descriptions; site landforms and landscape position; soil depths and descriptions; springs; observable structure, bedding, and joints.

DATA ANALYSIS

Distribution of Rock Types

Transition from time-stratigraphic to grouping of rock materials by physical properties; definition of extent and magnitude of discontinuities.

Distribution of Soil Types

Equate geologic/soil survey nomenclature to engineering nomenclature.

Geologic Structure

Establish spatial distribution of rock materials: locate major structural features; determine probable distribution of more detailed structural and textural; features.

Geologic History

Genesis of rock types; relationship to significant physical properties; rock and soil depositional processes; geomorphic history.

Seismicity

Historical seismicity; locations and characteristics of probable capable faults; possible earthquake magnitudes in region; possible intensities at candidate sites; preliminary selection of ground motions at candidate sites.

Hydrogeology

Regional ground water flow patterns and watertable depths; general hydraulic characteristics of subsurface materials; probable ground water and seepage conditions at candidate sites; preliminary assessment of project impact on groundwater.

Construction Materials

Existing sources in region located; probable areas for development rock and soil borrow delineated.

Regional geologic and soils conditions established; preliminary assessments of seismicity and construction materials; tentative models of geologic and geomorphic conditions at potential sites developed; preliminary inputs into Environmental Impact Statements (EIS) and HTRW reports developed

Figure 2-1. Schematic diagram of the development of regional geology

CIVIL WORKS FEASIBILITY STUDIES
GEOTECHNICAL INVESTIGATIONS

INITIAL FIELD INVESTIGATIONS

DATA COLLECTION

Areal and Site Geotechnical Mapping

Distribution of surface materials; structure; relation between materials and geomorphic expression; tentative locations for geophysical and/or subsurface explorations.

Surface Geophysical Surveys

Distribution of distinguishable subsurface materials at potential structure sites and rock and/or soil borrow areas; depth to water table; preliminary data on elastic and electrical properties; initial assessment of homogeneity of subsurface.

Subsurface Explorations

Control borings at potential structure sites and rock and/or soil borrow areas borehole logs; index tests on representative rock and soil samples; in situ hydraulic tests; camera or TV surveys; downhole geophysical logs.

DATA ANALYSIS

Aerial Conditions

Develop map of project area; describe possible material sources, material types, properties, amounts; determine unusual or safety related geologic conditions, faults, landslides, sinkholes, solution susceptible materials, etc.; update and expand hydrogeologic analyses including potential impacts on ground water.

Site Conditions

Correlate surface and subsurface data at potential sites; develop geologic map and sections for each site; assess structural and textural data for materials; assess results of index tests; develop preliminary assessment of rock and soil foundation conditions; delineate conditions affecting choice of structure for each site.

Geologic, soils, and engineering conditions evaluated to a level necessary to ensure each plan is a viable, safe, and complete technical system; geologic models at proposed sites upgraded; level of geotechnical investigations necessary to accomplish design established; need for dynamic analyses tentatively determined; EIS input updated.

Figure 2-2. Schematic diagram of initial field investigations surface geophysical surveys and subsurface explorations. Properly conducted surface geophysical surveys can provide information over relatively large areas on overburden depths, depth to the water table, and critical geologic contacts. Such surveys, prior to exploration drilling, can reduce the number of borings in proposed structure foundation areas and, in some cases, the number of borrow area borings. The surveys should be run along axes of potential dam sites and along canal alinement; at lock, off-channel spillways and tunnel and conduit sites; at potential borrow and quarry sites; and at locations where buried channels, caverns, or other elusive, but important, geologic conditions are suspected.

(a) Exploratory drilling is required at all sites to be included in the feasibility study. The numbers and depths of borings required to provide adequate coverage cannot be arbitrarily predetermined but should be sufficient to reasonably define the subsurface in the various site areas. Investigations necessary for levees, flood walls, pumping plants, recreation areas, and other miscellaneous structures are not as extensive as those required for major structures and projects. Generally, the scope of the regional geologic study is much reduced for projects authorized for site-specific reasons.

(b) The field investigation program should be tailored to site-specific needs. Field investigations in connection with planning of channel improvements or diversion channels should be sufficient to determine the types of materials to be excavated, hydraulic conductivity of the substrate, stability of bank slopes, and susceptibility of the substrate to scour. Assessment of channel stability for flood control projects is discussed in EM 1110-2-1418. In the case of irrigation or perched canals, seepage losses may be a significant problem. The field investigations should examine the need for an impervious lining and the availability of material for this purpose.

d. Reporting for feasibility studies. The reporting of results for feasibility in accordance with the overall study reporting requirements is contained in ER 1105-2-100. The results of all geotechnical investigations performed as part of the feasibility study efforts will be presented in detail. Sufficient relevant information on the regional and specific site geotechnical conditions must be presented to support the rationale for plan selection, project safety, environmental assessments including HTRW potential, and preliminary project design and cost estimates. This information should be presented in summary form in the feasibility report and in sufficient detail in appendices to allow evaluation and review.

(1) The feasibility report should contain summaries of the regional geology, soils, hydrogeology, and seismological conditions plus brief summaries of the areal and site geotechnical conditions for each detailed plan. These summaries should include local topography, geomorphic setting and history, thickness and engineering character of overburden soils, description of rock types, geologic structure, degree of rock weathering, local ground water conditions, possible reservoir rim problems, description of potential borrow areas and quarries, accessibility to sources of construction materials, and potential for HTRW sites. In addition, special foundation conditions such as excavation or dewatering problems, low-strength foundations, and cavernous foundation rock should be described. The summaries should conclude with a discussion of the relative geotechnical merits and drawbacks of each plan.

(2) Discussions of the regional geology and initial field investigations should be presented in an appendix on engineering investigations. The content of the discussion on regional geology should include the items outlined in Figure 2-1. In addition, a discussion of topography should be included. Drawings should be used to explain and augment the detailed discussion of regional geology. As a minimum, the drawings should include a regional geology map, regional geologic sections showing the spatial relationship of rock units and major geologic structures, a regional lineament map, and a map of recorded and observed seismic events (epicenter map). Dearman (1991) describes the principles of

engineering geologic mapping. Because summaries of areal and site geotechnical conditions for each detailed plan will be included in the feasibility report, the detailed discussion of areal and site geology, foundation conditions, and problems presented in the appendix may be limited to the recommended detailed plan. Figure 2-2 contains much of the information which should be included in the detailed discussion of areal and site geotechnical conditions. The discussion should indicate the sources from which information was obtained and will include the following items:

(a) Types of investigations performed.

(b) Areal and site geology (including topography of site or sites).

(c) Engineering characteristics of soil, rock, foundation, and reservoir conditions.

(d) Mineral deposits.

(e) Potential borrow and quarry sites.

(f) Available construction materials.

(g) Conclusions and recommendations.

(h) Graphics.

2-3. Preconstruction Engineering and Design Studies

a. Purpose. Preconstruction engineering and design (PED) studies are typically initiated after a feasibility study has been completed. PED studies are developed to reaffirm the basic planning decisions made in the feasibility study, establish or reformulate the scope of the project based on current criteria and costs, and formulate the design memoranda which will provide the basis for the preparation of plans and specifications. Figure 2-3 schematically outlines the engineering tasks for the PED studies with the requirements for geotechnical information.

b. Scope of geotechnical investigations. Geotechnical investigations performed during the PED studies should be in sufficient detail to assure that authorized measures can be implemented. The emphasis is toward site-specific studies which will provide the detail and depth of information necessary to select the most suitable site and structures to achieve project goals. The studies are performed in a series of incremental steps of increasing complexity beginning with the site selection study on major projects and continuing through feature design studies. Geotechnical procedures for performing field and laboratory investigations for these studies are found in Chapters 4 through 7.

c. Site selection study. This study serves to provide criteria for selecting the most appropriate site for the authorized project.

(1) Preliminary. The initial phase of the PED should begin with a comprehensive review of all geotechnical studies made during the feasibility study period. If there is a significant gap in time between the feasibility and PED studies, considerable geotechnical information may have been generated, compiled, analyzed, and published by Federal and State geotechnical agencies. These data should be obtained and correlated with the completed studies for evidence of significant changes in the geological knowledge of the study region. This is particularly important in the disciplines of seismology and hydrogeology.

CIVIL WORKS DESIGN MEMORANDUM STUDIES

GEOTECHNICAL INVESTIGATIONS

Purpose and Scope

Perform all engineering, design, and reporting for the authorized plan. Serve as a basis for preparation of constructions, plans, and specifications. Primarily concerned with the functional and technical design of structures necessary to achieve project objectives with the development of plans and specifications.

TASKS

- **Develop engineering details of approved project plan.**
  - **Sufficient geologic and soils information to verify project plan, axis selection, types of structures, estimates of costs, and to support the project purpose.**
  - **Field investigations, lab analyses, design computations, to determine adequacy and use of materials, strength, slopes, and protection of critical sections of embankments and foundations.**
  - **Foundation investigations, hydraulic design studies, and cofferdam design studies to determine most economical water control plan.**
- **Update cost and benefit estimates.**
- **Provide basis for cost-sharing agreements, prepare plans and specs, acquire lands, and negotiate relocation agreements.**
  - **Field investigations, analyses, and views of owners of relocations.**
- **Establish design and operating requirements to ensure a safe functional project.**
  - **Describe sources and indicate stability of concrete aggregate, cement, earth and rock borrow materials.**
  - **Investigations and laboratory analyses and typical design computations to determine source, adequacy, and use of construction materials**
- **Facilitate orderly scheduling and programming of funds for detailed design and construction.**
- **Continue environmental assessment based on required additional studies identified in the draft EIS.**
  - **Develop plans for reading and evaluating instrument data and making visual inspections of dams and downstream areas, both related to pool level.**
  - **Determine necessary instrumentation.**
- **Provide basis for recommendations of the District Engineer.**

Figure 2-3. Outline of preconstruction engineering and design studies

(2) Data collection. In the case of major projects such as dams, powerhouses, and navigation structures, some latitude normally exists in the proposed locations of the structures. At this stage, possible structure sites that would serve the intended project purposes should be evaluated before selecting a field investigations program. Geologic and hydraulic information collected during the feasibility study is generally sufficient for this purpose. After the obviously poor sites have been eliminated, a field investigation program should be developed. The type of data and collection methodology are outlined schematically in Figure 2-4. The investigation program should emphasize the completion of surface geologic mapping, expansion of surface geophysical surveys, detailed remote sensing analysis, and broadening of the scope of ground water investigations. Sufficient borings should be made at each potential site so that correlation of surface mapping and geophysical surveys is reasonably accurate. Use of the cone penetrometer and standard penetrometer test methods as part of a subsurface investigation program should be evaluated where geologic conditions are appropriate for these and subsequently more complex site studies. Subsurface sampling should be comprehensive to the point where laboratory indexing of engineering properties of soils and rock types, where appropriate, can be accomplished. Earthquake engineering analyses should be made if the seismicity studies made during the feasibility study indicate their need. At this time, a preliminary seismic evaluation should be made of the proposed structures and trenching performed if local active faulting is identified. The end result should be that areal and site geotechnical conditions are defined to the extent necessary to support design studies needed for reliable cost estimates. Data on proposed sites should be sufficiently complete to fully consider the effects of geotechnical conditions on site selection.

(3) Reporting site selection studies. The reporting of results of site selection studies will be in accordance with ER 1110-2-1150. The site selection design memorandum may be a separate document prepared prior to the PED for complex projects, or may be submitted as a major appendix to the PED. The content of the Site Selection Memorandum will include the items shown in Figure 2-4. Discussions will be augmented by geologic maps and profiles, boring logs, and laboratory and geophysical data all presented at a readable scale. The recommended site must be validated by sufficient geotechnical information in light of current conditions and criteria to avert reformulation of the project during the PED studies because of geotechnical problems.

d. Design investigations. Upon commencement of final design investigations, all previous engineering and design reports for the selected plan are carefully reviewed before initiating additional field investigations. These efforts provide information to support cost estimate decisions regarding the functional and technical design of structures necessary to achieve project objectives and development of construction plans and specifications. Design investigation tasks are outlined schematically in Figure 2-5 and are discussed in the following text.

(1) Preliminary. Upon commencement of the design investigations (postsite selection), all regional and site-specific geotechnical data should be reviewed before commencement of field investigations. New data, particularly that generated by other agencies, both Federal and state, should be obtained and incorporated into the original data base.

(2) Data collection. The foundation and design data collection activities are iterative, developing greater detail as the project design progresses toward the preparation of plans and specifications.

(a) PED data collection. In general, a closer order of subsurface investigations is then used in site selection studies. Where soils strongly influence the foundation design, undisturbed soil sampling should be initiated or expanded to classify and index their engineering properties in more comprehensive detail (Cernica 1993). Rock types and conditions, geologic structure, and engineering properties should

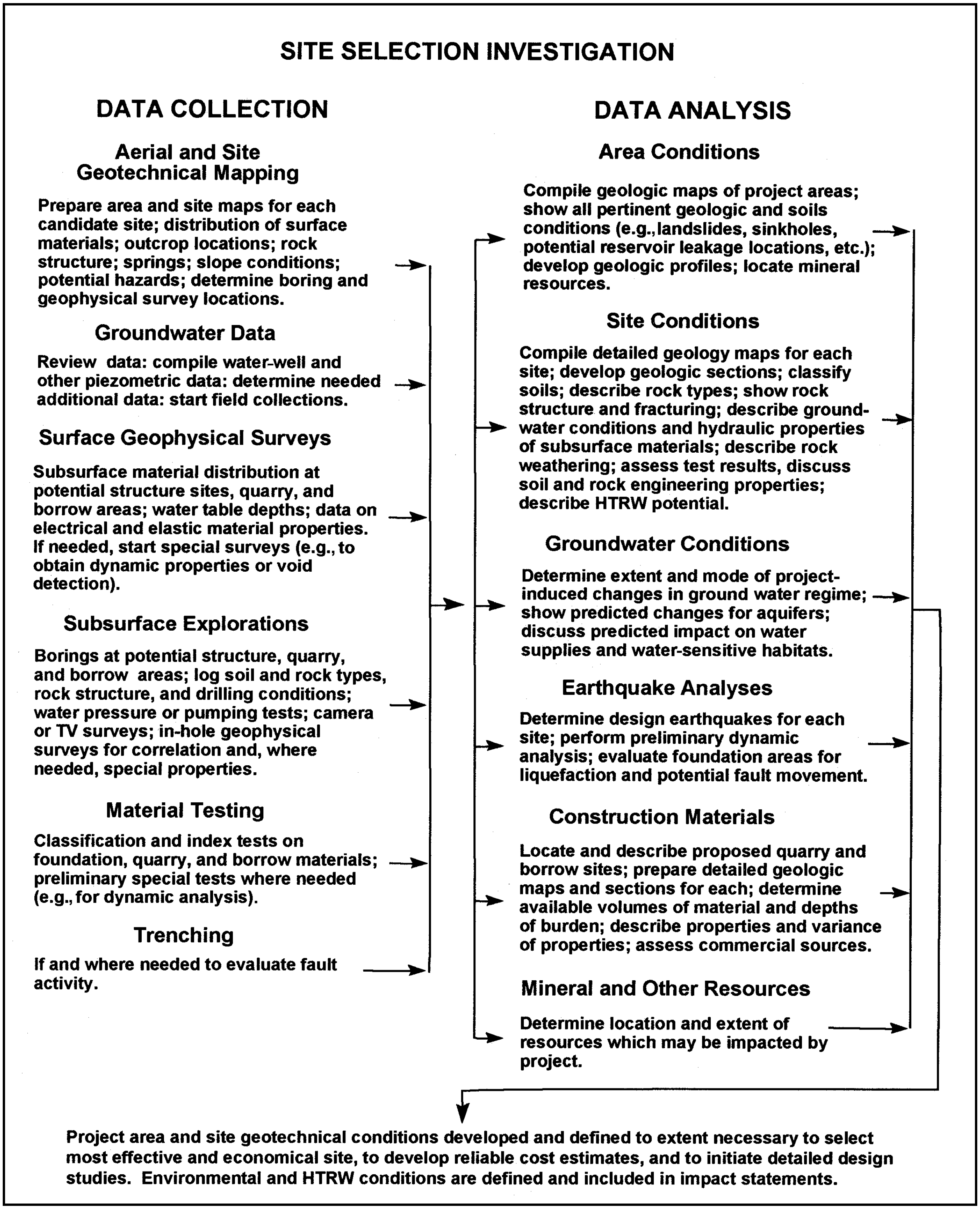

Figure 2-4. Schematic diagram on development of site selection investigations, general design memorandum (GDM)

DESIGN INVESTIGATIONS

DATA COLLECTION

Environmental/Ground water

Continue needed ground water data collection; observation well readings, pump tests, etc.; collect geotechnical data needed to update environmental assessments.

Subsurface Explorations

Expand coverage at selected structure sites, excavations, material sources, and relocations; log soil and rock types, structure, and drilling conditions.

Borehole Photography/TV

Obtain fracture frequency, orientation, and aperture; macrotextural and structural features; boring wall conditions.

Borehole Geophysics

Expand coverage with noncore borings; obtain in situ properties and stratigraphic correlation.

Water Pressure and/or Pumping Tests

Obtain permeabilities; monitor water levels.

Material Testing

Complete classification and index testing; perform engineering properties tests; continue and complete special testing started in earlier stages.

Exploratory Excavations and Constructions

Trenches, pits, adits, calyx holes, test quarries and borrows, test fills, test grout panels, etc.; in situ examination; in situ materials properties tests.

Instrumentation

Install and initiate readings on foundation instrumentation (e.g., piezometers, slope indicators) to develop baseline conditions.

DATA ANALYSIS

Ground water Assessment

Continue analyses started in earlier program; finalize statement of project impact on ground water.

Project Site Conditions

Update site geologic maps, geologic sections, soil and rock classifications, rock structure, material hydraulic characteristics, ground water conditions; complete design earthquake, reservoir leakage, and other special studies.

Structure Excavation Site Conditions

Develop detailed distribution of subsurface materials, select pertinent engineering properties for each material; complete dynamic analyses; analyze data and describe encountered conditions from any test excavations, quarries, grout programs, etc.; discuss all conditions affecting design conditions.

Construction Materials

Finalize volume estimates; show distribution of subsurface materials and their properties; analyze and describe results from test fills; finalize assessment of commercial materials sources.

Instrumentation

Reduce data from various sources; correlate data with events occurring; produce baseline plots for construction and postconstruction conditions.

Relocations

Develop pertinent data for each relocation increment in the same manner as structure/excavation sites.

Constructibility

Assure accurate depiction of site conditions and adoption of structures to geotechnical conditions and constraints.

Geotechnical conditions developed in sufficient detail to establish final design and operating requirements for a safe, functional project, develop design details, prepare final cost estimates, prepare plans and specifications, negotiate relocation agreements, acquire necessary lands and complete environmental HTRW assessments.

Figure 2-5. Schematic diagram for design investigations for postsite studies be defined to the extent necessary to design foundation treatment. In the case of water-retention structures, pump and/or pressure tests should be performed. Installation of observation wells and piezometers should be initiated early in the investigations so that seasonal variation in ground water levels can be observed. Geophysical studies should be expanded to include crosshole surveys. Regional ground water and earthquake engineering analyses should be completed. Upon completion of the PED studies, geotechnical conditions should be developed in sufficient detail to establish design and operating requirements for a safe, functional project. If the overall project scope is such that feature design memoranda are not prepared, the geologic and soils information should be sufficient to support the preparation of plans and specifications.

(b) Feature design data collection. Following the PED study, project complexity and size will frequently require separate feature design memoranda on such structures as concrete dams, navigation locks, outlet works, road relocation, and other similar project features. Generally, each special memorandum requires a geotechnical investigation. The investigation is an extension of previous studies but focuses on the area surrounding the structure under study. These studies are expanded by close-order subsurface investigations which may include large-diameter borings, cone penetrometer or standard penetrometer tests, test excavations, fills, and grouting programs, detailed laboratory testing, pile driving and load tests, and any other method of investigations which will resolve issues or problems that came to light during the PED study. Such issues and problems may include detailed evaluation of underseepage, dynamic response, and stability. For major projects requiring large amounts of concrete and/or protection stone, separate feature design memoranda specific to these materials should be prepared. This investigation is started during the initial study period and completed early in the feature design study so that the major structure requiring the materials can be properly designed. At the completion of the FDM studies, all geotechnical features and problems should have been identified and resolved. The final design, incorporating the geological conditions encountered at the sites and identifying the selected geotechnical design parameters, will be complete so that preparation of plans and specifications for construction of the project can proceed.

(3) Design investigations reports. Reporting results of design investigations will be in accordance with the overall reporting requirements contained in ER 1110-2-1150. In many cases, project complexity and size will require that the design investigations be reported in the general design memorandum (GDM) and an orderly series of feature design memoranda (FDM). This information provides the basis for constructibility assessment and formulation of the final design and preparation of the construction plans and specifications.

(a) GDM. The results of all foundation and design investigations performed as part of the project engineering studies will be summarized in the GDM and presented in detail in appendices to that report. The updated regional geology, engineering seismology, hydrogeology, and earthquake engineering studies should be thoroughly documented. As previously stated, if a separate report has not been prepared, the site selection studies should be presented as an appendix to the GDM. If FDM are not prepared, the geologic and soils information in the PED should be sufficient to support the preparation of plans and specifications. Discussions should be augmented by geologic maps and profiles, boring logs, laboratory and geophysical data, and special studies relating to seismology, ground water, and construction materials.

(b) FDM. The geotechnical discussion that will be included in the various FDM which discuss geological aspects of the project should be similar in scope to that presented in the GDM. However, only geotechnical data that clarify the particular intent of the FDM will be used. The discussion will be augmented by the appropriate geologic maps and profiles, boring logs, and laboratory and geophysical data. The design memoranda that are distinctly geotechnical or have geotechnical input of significant importance include:

(i) Site geology.

(ii) Concrete materials or protection stone.

(iii) Embankment and foundation.

(iv) Outlet works.

(v) Spillway.

(vi) Navigation lock.

(vii) Instrumentation and inspection program.

(viii) Initial reservoir filling and surveillance plan.

(ix) Intake structure.

(x) Relocations (roads and bridges).

e. Formulation and evaluation of construction plans and specifications.

(1) Biddability, constructibility, and operability review. Constructibility review is performed in accordance to ER 415-1-11. Constructibility studies evaluate compatibility of design, site, materials, techniques, schedules, and field conditions; sufficiency of details and specifications; and freedom from design errors, omissions, and ambiguities. District offices will coordinate project review by geotechnical, construction, and engineering personnel to improve the constructibility of design. The review process should occur during the development of the draft plans and specifications and, therefore, not be responsible for major changes in foundation and embankment design, instrumentation, or other geotechnically related features which could impact on the project schedule.

(2) Preparation of plans and specifications. Plans and specifications will be prepared in accordance with ER 1110-2-1200. The plans and specifications will contain an accurate depiction of site conditions and will be carefully prepared to eliminate or depict conditions which might delay the work or be grounds for claims. Plans and specifications will contain a thorough graphic presentation of all borings made in the area under contract. All boring locations will be shown. Factual data representing field surveys such as geophysical and hydrological investigations shall be presented in a usable form, preferably a GIS format. Because of the voluminous nature of laboratory data, these data not presented with the borings or tabulated elsewhere must be indicated as available to all prospective contractors. Other data in this category could include mapping data, photographs, and previously published geologic reports and design documents.

(3) Geotechnical design summary report. For some projects where geotechnical considerations are of paramount importance, a geotechnical design summary report may be prepared and included with the bid documents. Disclosure of the design assumptions and interpretations of data in this type of document often serve to clarify intent during the construction of a project.

2-4. Construction Activities

a. Purpose. In some cases, construction activities such as test fills or test excavations are performed to prepare plans and specifications that are compatible with the project design. These plans and specifications are to assure construction quality and document actual construction conditions.

b. Scope of geotechnical activities. Geotechnical activities in support of the construction phase of a project can be divided into three phases: construction management, quality assurance, and compilation of summary reports.

c. Execution of geotechnical construction activities. Guidelines for conducting construction activities are contained in the following Engineering Regulations: ER 415-2-100, ER 1110-2-1200, ER 1110-2-1925, and ER 1180-1-6. Construction activities are summarized in Figure 2-6 and discussed as follows:

(1) Construction management. Construction management and policies are performed in accordance with ER 415-2-100. It is the goal of the Corps of Engineers to construct and deliver a quality product. The key to obtaining this objective is an effective construction management system staffed by an adequate number of trained and competent personnel. Areas of expertise shall be appropriate to the type of construction project under contract and can include, but not be limited to, foundation preparation, rock and soil excavation, embankment and concrete control and emplacement, and grouting.

(a) Claims and modifications. Regardless of the intensity of geological investigations during the preconstruction phase, differing site conditions, claims, and modifications are to be expected on complex projects. Therefore, engineering should provide necessary support to investigate claims and provide design and cost-estimating assistance for any claims and modifications.

(b) Site visits for verification of quality. On all projects, but especially those too small to support a resident geologist or geotechnical engineer, site visits should be made regularly by qualified personnel to verify that conditions match assumptions used in designing the project features and to assist construction personnel on any issues affecting construction. All visits should be well documented (including an extensive photographic and video record) and be included in appropriate summary reports.

(2) Quality assurance and management. Quality assurance, which is the responsibility of the Government, will be performed in accordance with ER 1180-1-6. The geotechnical staff members assigned to the project have the responsibility to monitor, observe, and record all aspects of the construction effort relating to foundations, embankments, cuts, tunnels, and natural construction materials. Figure 2-7 shows in tabulated form some of the particular items requiring quality assurance particular to geotechnically oriented features.

(a) Quality assurance testing will be performed to assure acceptability of the completed work and verify quality control test procedures and results. An onsite laboratory should be required on major projects to perform all soil and concrete testing. During construction, considerable data are assembled by the project geotechnical quality assurance staff. These data consist generally of foundation mapping and treatment features, embankment-backfill performance data, grouting records, material testing data, pile driving records, and instrumentation results. Special treatment and problem areas, often requiring contract modification, should be well documented.

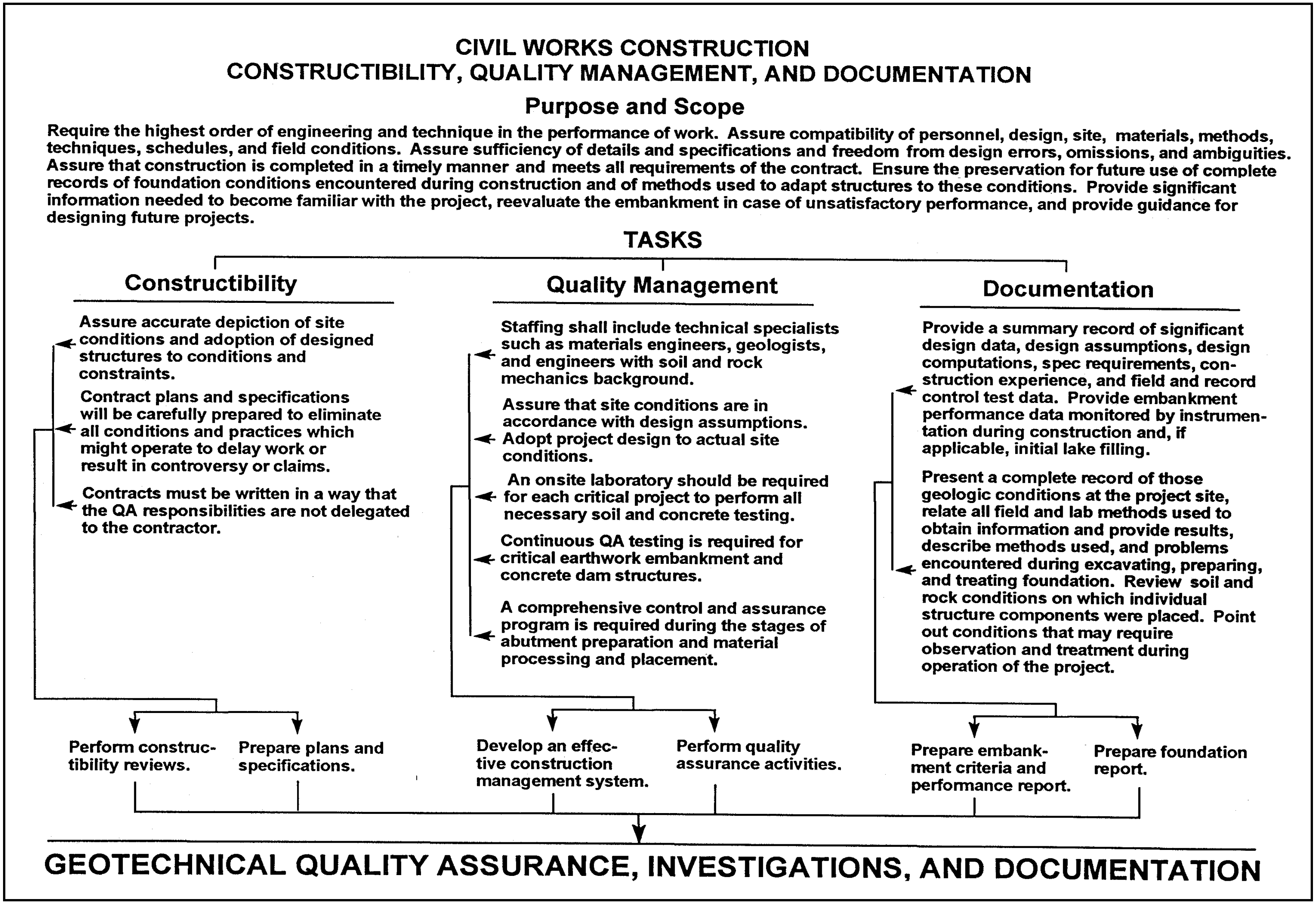

Figure 2-6. Outline of tasks for construction geotechnical activities

CIVIL WORKS CONSTRUCTION
CONSTRUCTIBILITY, QUALITY MANAGEMENT, AND DOCUMENTATION

QUALITY ASSURANCE OF GEOTECHNICAL ACTIVITIES

Excavation Procedures

- Grades
- Unwatering
- Overburden
- Rock
  - Blast patterns/procedures
  - Fragmentation
  - Control of wall rock damage
- Slope stability
- Support
- Preliminary cleanup
- Surface protection

Foundation/Abutment Treatment

- Subsurface
  - Curtain grouting
  - Area grouting
  - Consolidation grouting
  - Caissons, trenches, slurry walls, etc.
- Surface
  - Final cleanup
  - Dental concrete
  - Shotcrete
  - Slurry grouting
- Drainage
  - Adits
  - Drain holes
  - Relief wells

Embankment/Backfill

- Material source
- Material placement
- Control tests
- Slope stability
- Seepage control
- Diversion and closure

Figure 2-7. Critical geotechnical activities that require carefully outlined quality assurance procedures

(b) Early in the construction of the project, the geotechnical staff should develop a data analysis and storage system, preferably one which can be used to monitor construction activities. The Grouting Database Package (Vanadit-Ellis et al. 1995) is a personal computer (PC)-based, menu-driven program that stores and displays hole information, drilling activities, water pressure tests, and field grouting data. Instrumentation Database Package (Woodward-Clyde Consultants 1996) is a menu-driven, PC Windows-based program that can store, retrieve, and graphically present instrumentation data related to construction monitoring. A GIS is an effective and comprehensive means to monitor and analyze all aspects of a construction project, from its development as an idea to postconstruction operations and maintenance. Geotechnical information (data layers) can be a critical part of the data base management and analysis program. A GIS is an organized collection of computer hardware and software, geographic data, and personnel designed to efficiently collect, store, update, manipulate, analyze, and display geographically referenced information.

(c) Figure 2-8 is a flow diagram of the general procedure for carrying out construction investigations and documenting the results. A GIS is specifically designed to compile and analyze spatial data as depicted in Figure 2-8.

(3) Construction foundation and embankment criteria reports.

(a) The purpose of the foundation report is to ensure the preservation for future use of complete records of foundation conditions encountered during construction and methods used to adapt structures to these conditions. The foundation report is an important document for use in evaluating construction claims, planning additional foundation treatment should the need arise, evaluating the cause of foundation or structural feature distress and planning remedial action to prevent failure or partial failure of a structure, planning and design of major rehabilitation or modifications to the structure, providing

CIVIL WORKS CONSTRUCTION

CONSTRUCTIBILITY, QUALITY MANAGEMENT, AND DOCUMENTATION

CONSTRUCTION INVESTIGATIONS AND DOCUMENTATION OF GEOTECHNICAL ACTIVITIES

DATA COLLECTION

Regional and Site Geologic Data

Review all prior maps and narratives. Add data acquired after engineering and design investigations, including preconstruction instrumentation data.

Excavation and Foundation Mapping

Prepare detailed geologic maps of all permanent excavations and structure foundations. Provide complete descriptions of all geologic and foundation treatment features.

Quality Assurance

Compile excavation, foundation/abutment, and embankment/landfill QA activities data.

Subsurface Explorations

Drill to confirm foundation grades, effectiveness of foundation treatment, and to investigate unanticipated conditions.

Material Testing

Field control test to confirm design values.

Instrumentation

Install and monitor piezometers, soil pressure devices, surface monuments, slope indicators, strong motion instruments, and special instruments.

DATA ANALYSIS

Project Geologic Conditions

Update regional and aerial geology. Revise site geologic maps and sections showing excavations, structures, and general conditions.

Foundation/Construction Material Explorations

Assess adequacy of preconstruction explorations. Describe construction explorations.

Excavation Procedures

Reduce and compile data from QA activities. Discuss effectiveness of methods used and compare with design concepts.

Foundation Conditions

Integrate data from excavation and foundation mapping and subsurface explorations. Describe foundation surfaces, type and condition of foundation materials, encountered water, etc.

Foundation and Abutment Treatment

Reduce and compile data from QA activities. Discuss effectiveness and compare with design concepts. Provide record of foundation approvals.

Embankment Construction

Summarize design. Compile data on construction procedures, control tests, placed conditions, seepage control, and stability.

Instrumentation

Reduce preconstruction and construction data. Compare with design predictions.

Geotechnical quality assurance activities and construction investigations data are compiled and analyzed in sufficient detail to present in embankment criteria and performance and foundation reports. Integration of completed project into regional and local environment are reported. Conditions which may require observation and treatment during project operation are identified. Future observations of critical geotechnical features are recommended.

Figure 2-8. Schematic diagram of construction geotechnical investigations and documentation guidance in planning foundation explorations, and in anticipating foundation problems for future comparable construction projects in similar geologic settings. Site geotechnical personnel responsible for the foundation report must begin to formulate the report as soon as possible after construction begins so that completion of the report can be accomplished by those who participated in the construction effort. This report should include collaboration with design and construction personnel. Detailed video recordings of foundation conditions should be an integral component of the foundation report.

(b) For major embankments, an embankment and performance criteria report will be prepared to provide a summary record of significant design data, design assumptions, design computations, specification requirements, construction equipment and procedures, construction experience, field and record control test data, and embankment performance as monitored by instrumentation during construction and during initial reservoir fillings. The report will provide essential information needed by engineers to familiarize themselves with the project, reevaluate the embankment in the event of unsatisfactory performance, and provide guidance for designing comparable future projects. The report should be authored by persons with firsthand knowledge of the project design and construction. The report should be in preparation during construction and completed as soon as possible following project completion.

Section II
Military Construction Projects

2-5. Background

Program development for Military Construction Army (MCA), Air Force, and other Army projects from initial conception by the installation to final public law appropriation by Congress and construction accomplishment will require a general sequence of geotechnical investigations as shown on Table 2-1. For design phase investigation of facilities required to resist nuclear weapons effects, the guidance in this manual should be supplemented with appropriate material from TM 5-858-3. Information systems to support military construction activities are discussed in ER 1110-3-110.

Table 2-1
Sequence of Geotechnical Investigations with Project Development Stages

Project Development Stages		
Military Construction	**Civil Works**	**Geotechnical Investigations**
Preconstruction and site selection studies	Reconnaissance and feasibility study	Thorough literature search Development of regional geology Field reconnaissances and initial field investigations
Final design studies	Preconstruction planning and engineering General Design Memorandum and Feature Design Memoranda	Review of regional geology Site selection investigations Foundation and design investigations
Construction	Construction	Constructibility review, quality assurance, and postconstruction documentation activities

2-6. Preconcept and Site Selection Studies

a. Purpose and scope. Preconcept information compilation occurs during the guidance year of the MCA program development flow. During the year, the Major Area Commands (MACOM) will prepare annual programs, set priorities, and submit their programs. Initial action consists of preparation of Project Development Brochures (PDB). PDBs contain data necessary to plan, budget, and initiate design of construction projects. The initial PDB is general in nature and provides information regarding project and site conditions. The initial DD Form 1391 contains the preliminary information about the project. A preliminary site survey and subsurface evaluation is included. The preliminary site survey should include a background check to assess the potential for encountering HTRW on potential sites. After approval by the Department of the Army, the second PDB is formulated, generally by the District. This PDB contains total user requirements and complete site and utility support information. Information on general subsurface conditions and any special foundation requirements such as deep foundations or special treatment is included. The preconcept and site selection studies culminate in preconcept control data based on the approved PDB, including a cost estimate and necessary reference data.

b. Scope of geotechnical investigations. Geotechnical investigations during preconcept studies should be performed to a level which assures adequate information. They should be performed by the District's geotechnical element and be sufficiently complete to permit selection of the most favorable site within the regional physical setting, determine the general type of structure best suited to the site conditions, assess the geotechnical aspects of environmental impact, and ascertain the costs of the project. The scope of the investigations should not be greater than that necessary to accomplish these aims. For projects on existing military installations, much of the information needed for preconcept studies is already available and the additional investigation requirements will be minimal. For projects in new areas where information is not readily available, the investigation requirements will be similar to those for Civil Works Feasibility studies. Emphasis, however, will be placed on site-specific parameters relating to size and special requirements for each project.

c. Reporting. Geotechnical investigation results will be reflected in the information and decisions presented in DD Form 1391 and in developed PDBs. Results of drilling and testing programs and special investigations should be compiled in summary reports.

2-7. Concept Studies

a. Purpose and scope. Concept studies provide drawings and data developed prior to initiation of final design, and constitute approximately 35 percent of total design. They serve to define the functional aspects of the facility and provide a firm basis on which the district engineer can substantiate project costs and initiate final design. Included are project site plans, materials and methods of construction, and representative cross sections of structure and foundation conditions. The concept design is accomplished during the design year stage of the program development plan and leads to development of budget data. Budget data reflect firm construction requirements and contain a current working estimate. Data are used in congressional budget hearings.

b. Scope of geotechnical investigations. Geotechnical investigations for concept studies should be performed by the district's geotechnical element and should provide information similar to that included for the preconcept studies but should advance the information to the level required for design and budget development.

c. Reporting. Reporting of the results of the geotechnical investigations is included in the design analysis developed to the 35-percent design study level. Emphasis should be placed on selection of foundation types and the influences of subsurface conditions.

2-8. Final Design Studies

a. Purpose and scope. Final design studies provide a complete set of working drawings for a project, accompanied by appropriate technical specifications, design analyses, and detailed cost estimates covering all architectural, site, and engineering details. These documents are used to obtain construction bids and to serve as construction contract documents.

b. Scope of geotechnical investigations. Geotechnical investigations for final design should be performed by the district's geotechnical element and should provide additional information to the preconcept and concept stage investigations for a final and complete design. All detailed considerations for economic designs, environmental influences, and construction processes should be finalized. The level of information should be similar to that for Civil Works General Design Memorandum studies.

c. Reporting. The results of the geotechnical investigations will be included in the final design analysis. Information should be similar to that collected for Civil Works General Design Memorandum studies, except additional emphasis will be placed on analysis for selection of foundation types and details of the foundation design.

2-9. Construction Activities

a. Scope of geotechnical investigations. In addition to quality control testing, geotechnical activities will be required during construction for special considerations or problems. Such activities will be necessary to confirm design assumptions, analyze changed conditions, determine special treatment requirements, analyze failures, and provide new materials sources.

b. Reporting. Investigation results will be provided in special summary reports and in construction foundation reports as required for major or unique projects.

Chapter 3
Regional Geologic and Site Reconnaissance Investigations

3-1. Background

Regional geologic and site reconnaissance investigations are made to develop the project regional geology and to scope early site investigations. The steps involved and the data needed to evaluate the regional geology of a site are provided in Figure 2-1. The initial phase of a geologic and site reconnaissance investigation is to collect existing geologic background data through coordination and cooperation from private, Federal, State, and local agencies. Geologic information collected should then be thoroughly reviewed and analyzed to determine its validity and identify deficiencies. Geologic data should also be analyzed to determine additional data requirements critical to long-term studies at specific sites, such as ground water and seismicity, that will require advance planning and early action. Upon completion of the initial phase, a geologic field reconnaissance should be conducted to examine important geologic features and potential problem areas identified during collection of background data. Field observations are used to supplement background data and identify the need to collect additional data.

a. Geologic model. Geologic background and field data that are determined to be valid should be used to construct a geologic model for each site. The model, which will require revisions as additional information is obtained, should indicate possible locations and types of geologic features that would control the selection of project features. Preliminary geologic, seismic, hydrologic, and economic studies should be used to indicate the most favorable sites before preliminary subsurface investigations are started. Proper coordination and timing of these studies, and incorporation into a GIS, can minimize costs and maximize confidence in the results.

b. Small projects. Many civil works projects are too small to afford a complete field reconnaissance study as outlined below. For smaller projects, emphasis should be placed on compilation and analysis of existing data, remote sensing imagery, and subsurface information derived from on-site drilling and construction excavations. A geologist or geotechnical engineer should be available to record critical geotechnical information that comes to light during investigations. An extensive photographic and video record taken by personnel with some background in geology or geotechnical engineering can serve as a reasonable proxy for onsite investigations.

Section I
Coordination and Information Collection

3-2. Interagency Coordination and Cooperation

Sources of background information available from other organizations can have a substantial influence on project economy, safety, and feasibility. During initial investigations, project geologists may be unfamiliar with both the regional and local geology. Limited funds must be allocated to many diverse areas of study (e.g., economics, real estate, environment, hydrology, and geology). For these reasons, contacts should be made with Federal, State, and local agencies to identify available sources of existing geologic information applicable to the project. A policy of formal coordination with the USGS has been established as outlined in the following text. In addition, informal coordination should be maintained with state geological surveys because critical geologic data and technical information are often available from these agencies. Other organizations listed below may also provide valuable information.

a. Coordination with USGS. The 27 October 1978 Memorandum of Understanding (MOU) with the USGS provided for exchange of information to assure that all geologic features are considered in project planning and design. The MOU outlines three main activities:

(1) The USGS provides the Corps of Engineers with existing information and results of research and investigations of regional and local geology, seismology, and hydrology relevant to site selection and design.

(2) The USGS advises the Corps of Engineers on geologic, seismologic, and hydrologic processes where knowledge is well developed and on specific features of site and regional problems.

(3) The Corps provides the USGS with geologic, seismologic, and hydrologic data developed from Corps studies.

The MOU requires that the USGS be notified in writing if planning studies are to be initiated at a new site or reinitiated at a dormant project. The notification should specify the location of interest and identify specific geologic, seismologic, and hydrologic considerations for which information is needed.

b. Other organizations. Contacts and visits to offices of the following organizations can produce valuable geotechnical information in the form of published maps and reports and unpublished data from current projects.

(1) Federal agencies.

(a) Department of Agriculture. - Forest Service - Natural Resource Conservation Service

(b) Department of Energy.

(c) Department of Interior. - Bureau of Indian Affairs - Bureau of Land Management

- Bureau of Reclamation - Fish and Wildlife Service - Geological Survey

- National Park Service - National Biological Service.

(d) Department of Transportation. - Federal Highway Administration regional and state division offices.

(e) Environmental Protection Agency regional offices.

(f) Nuclear Regulatory Commission.

(g) Tennessee Valley Authority.

(2) State agencies.

(a) Geological Surveys, Departments of Natural Resources, and Departments of Environmental Management.

(b) Highway departments.

(3) Municipal engineering and water service offices.

(4) State and private universities (geology and civil engineering departments).

(5) Private mining, oil, gas, sand, and gravel companies.

(6) Geotechnical engineering firms.

(7) Environmental assessment firms.

(8) Professional society publications.

3-3. Survey of Available Information

Information and data pertinent to the project can be obtained from a careful search through published and unpublished papers, reports, maps, records, and consultations with the USGS, state geologic and geotechnical agencies, and other Federal, state, and local agencies. This information must be evaluated to determine its validity for use throughout development of the project. Deficiencies and problems must be identified early so that studies for obtaining needed information can be planned to assure economy of time and money**.** Especially in the case of larger projects, data are most effectively compiled and analyzed in a GIS format. Table 3-1 summarizes the sources of topographic, geologic, and special maps and geologic reports. Most states regulate well installation and operation and maintain water well data bases that extend back many years. Wells may be municipal, industrial, domestic, or may have been drilled for exploration or production of natural gas. The information that is commonly available includes date of installation, screened interval, installer's name, depth, location, owner, and abandonment data. Lithologic logs may also be available and, in rare cases, production and water quality information.

Section II
Map Studies and Remote Sensing Methods

3-4. Map Studies

Various types of published maps, such as topographic, geologic, mineral resource, soils, and special miscellaneous maps, can be used to obtain geologic information and develop regional geology prior to field reconnaissance and exploration work. The types of available maps and their uses are described in Dodd, Fuller, and Clark (1989).

a. Project base map. Spatial components typically used to define a GIS referenced base map include: topographic maps, aerial photographs (digital orthophotos), monumentation/survey control maps, surface/subsurface geology maps, land use maps, bathymetry maps, and various forms of remotely sensed data. Project-specific planimetric maps, digital terrain models (DTMs), and digital elevation models (DEMs) are produced through photogrammetric methods and can be generated using a GIS. A DTM may be used to interpolate and plot a topographic contour map, generate two-dimensional (2-D) (contour) or three-dimensional (3-D) (perspective) views of the modeled surface, determine earthwork quantities, and produce cross sections along arbitrary alignments.

Table 3-1
Sources of Geologic Information

Agency	Type of Information	Description	Remarks
USGS	Topographic maps	U.S. 7.5-minute series 1:24,000 (supersedes 1:31,680). Puerto Rico 7.5-minute series1:20,000 (supersedes 1:30,000) Virgin Island 1:24,000 series. U.S. 15-minute series 1:62,500 (1:63,360 for Alaska) U.S. 1:100,000-scale series (quadrangle, county, or regional format) U.S. 1:50,000-scale county map series U.S. 1:250,000-scale series Digital elevation models are available for entire U.S. at 1:250,000, and for certain areas at 1:100,000 and 1:24,000 scales. Digital line graphs are available for some areas at 1:24,000, 1:65,000, 1:100,000 for: - Hydrography - Transportation - Boundaries - Hypsography	Orthophotoquad monocolor and color infrared maps also produced in 7.5- and 15-min series. New index of maps for each state began in 1976. Status of current mapping from USGS regional offices and in monthly USGS bulletin, "New publications of the U.S. Geological Survey." Topographic and geological information from the USGS can be accessed through the ESIC (1 800 USAMAPS)
USGS	Geology maps and reports	1:24,000 (1:20,000 Puerto Rico), 1:62,500, 1:100,000, and 1:250,000 quadrangle series includes surficial bedrock and standard (surface and bedrock) maps with major landslide areas shown on later editions 1:500,000 and 1:2,500,000	New index of geologic maps for each state began in 1976. List of geologic maps and reports for each state published periodically
USGS	Miscellaneous maps and reports	Landslide susceptibility rating, swelling soils, engineering geology, water resources, and ground water	Miscellaneous Investigation Series and Miscellaneous Field Studies Series, maps and reports, not well cataloged; many included as open file reports
USGS	Special maps	1:7,500,000 and 1:1,000,000: Limestone Resources, Solution Mining Subsidence, Quaternary Dating Applications, Lithologic Map of U.S., Quaternary Geologic Map of Chicago, Illinois, and Minneapolis, Minnesota, areas	
USGS	Hydrologic maps	Hydrologic Investigations Atlases with a principal map scale of 1:24,000; includes water availability, flood areas, surface drainage precipitation and climate, geology, availability of ground and surface water, water quality and use, and streamflow characteristics	Some maps show ground water contours and location of wells
USGS	Earthquake hazard	Seismic maps of each state (began in 1978 with Maine); field studies of fault zones; relocation of epicenters in eastern U.S.; hazards in the Mississippi Valley area; analyses of strong motion data; state-of-the-art workshops	Operates National Strong-Motion Network and National Earthquake Information Service publishes monthly listing of epicenters (worldwide). Information is available through ESIC (1 800 USAMAPS)

(Sheet 1 of 3)

Table 3-1 (Continued)

Agency	Type of Information	Description	Remarks
USGS	Mineral resources	Bedrock and surface geologic mapping; engineering geologic investigations; map of power generating plants of U.S. (location of build, under construction, planned, and type); 7.5-min quadrangle geologic maps and reports on surface effects of subsidence into underground mine openings of eastern Powder River Basin, Wyoming	
USGS	Bibliography	"Bibliography of North American Geology," (USGS 1973)	USGS Professional Paper
American Geological Institute	Bibliography	(American Geological Institute) print counterpart.	
		"Bibliography and Index of Geology" to "Geo Ref" digital index (USGS 1973)	1977 to present, 12 monthly issues plus yearly cumulative index
National Oceanic and Atmospheric Administration (NOAA)	Earthquake hazards	National Geophysical Center in Colorado contains extensive earthquake hazard information (303-497-6419)	
National Aeronautics and Space Administration (NASA)	Remote sensing data	Landsat, skylab imagery	
NOAA	Remote sensing data		
Earth Observation Sattelite (EOSAT)	Remote sensing data		
US Fish and Wildlife Service (FWS)	Wetlands	The National Wetlands Inventory maps at 1:24,000 for most of the contiguous U.S.	Available as maps or mylar overlays
USGS	Flood-prone area maps	1:24,000 series maps outlining floodplain areas not included in Corps of Engineers reports or protected by levees	Stage 2 of 1966 89th Congress House Document 465
USAEWES	Earthquake hazard	"State-of-the-Art for Assessing Earthquake Hazards in the United States," Miscellaneous Paper S-73-1 (Krinitzsky 1995)	Series of 29 reports, 1973 to present
Natural Resources Conservation Service (NRCS)	Soil survey reports	1:15,840 or 1:20,000 maps of soil information on photomosaic background for each county. Recent reports include engineering test data for soils mapped, depth to water and bedrock, soil profiles grain-size distribution, engineering interpretation, and special features. Recent aerial photo coverage of many areas. Soils maps at 1:7,500,000 and 1:250,000, 1:31,680, and 1:12,000 scale are available in digital format for some areas.	Reports since 1957 contain engineering uses of soils mapped, parent materials, geologic origin, climate, physiographic setting, and profiles

(Sheet 2 of 3)

Table 3-1 (Concluded)

Agency	Type of Information	Description	Remarks
State Geologic Agencies	Geologic maps and reports	State and county geologic maps; mineral resource maps; special maps such as for swelling soils; bulletins and monographs; well logs; water resources, ground water studies	List of maps and reports published annually, unpublished information by direct coordination with state geologist
Defense Mapping Agency (DMA)	Topographic maps	Standard scales of 1:12,500, 1:50,000, 1:250,000 and 1:1,000,000 foreign and worldwide coverage, including photomaps	Index of available maps from DMA
American Association of Petroleum Geologists	Geological highway map series	Scale approximately 1 in. equal 30 miles shows surface geology and includes generalized time and rock unit columns, physiographic map, tectonic map, geologic history summary, and sections	Published as 12 regional maps including Alaska and Hawaii
TVA	Topographic maps, geologic maps and reports	Standard 7.5-min TVA-USGS topographic maps, project pool maps, large-scale topographic maps of reservoirs, geologic maps, and reports in connection with construction projects	Coordinate with TVA for available specific information
U.S. Department of Interior, Bureau of Reclamation (USBR)	Geologic maps and reports	Maps and reports prepared during project planning and design studies	List of major current projects and project engineers can be obtained. Reports on completed projects by interlibrary loan or from USAEWES for many dams
Agricultural Stabilization and Conservation Services Aerial Photography Field Office (APFO)	Aerial photographs	The APFO offers aerial photographs across the U.S. Typically a series of photographs taken at different times, as available for a given site	Information is available at 801-975-3503
USGS Earth Resources Observation Systems (EROS) Center (EDC)	Aerial photographic coverage	The EDC houses the nation's largest collection of space and aircraft acquired imagery	Information is available at 605-594-6151 or 1 800 USAMAPS
Satellite Pour l' Observation de la Terre' (SPOT)	Remote sensing imagery	High-resolution multispectral imagery produced by France's SPOT satellite imager is available for purchase	Contact for SPOT images is at 800-275-7768

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(1) Geotechnical parameters resulting from surface and subsurface explorations can be georeferenced to a DTM resulting in a spatial data base capable of producing geologic cross sections and 2- and 3-D strata surface generation. Georeferencing spatial data requires that the information be precisely located. Global Positioning System (GPS) techniques offer a rapid and reliable way to accomplish this (EM 1110-1-1003). Even with a GPS however, surveyed monuments and benchmarks must be identified and used as control points in the survey. Benchmark and brass cap information is available through the National Geodetic Survey of NOAA for the entire United States guidance and criteria for monumentation installation and documentation on Corps projects is outlined in EM 1110-1-1002.

(2) A GIS can be used to streamline and enhance regional or site-specific geotechnical investigations by: (a) Verifying which information is currently available and what new data must be obtained or generated to fulfill requirements for the desired level of study; (b) Sorting and combining layers of information to evaluate the commonality of critical parameters and compatibility of proposed alternatives/sites; and (c) Assigning quantitative values and relational aspects of data combinations and classifications, e.g., computing the probability of correctly assigning a given liquifaction potential for a proposed foundation construction method at a given site location. In this respect, a geotechnically augmented GIS database can be used to quantify reliability and uncertainty for specific design applications and assumptions. Burrough (1986), ESRI (1992), Intergraph (1993), and Kilgore, Krolak, and Mistichelli (1993) provide further discussions of GIS uses and capabilities.

b. Topographic maps. Topographic maps provide information on landforms, drainage patterns, slopes, locations of prominent springs and wet areas, quarries, man-made cuts (for field observation of geology), mines, roads, urban areas, and cultivated areas. Requirements for topographic mapping and related spatial data are outlined in EM 1110-1-1005. If older topographic maps are available, especially in mining regions, abandoned shafts, filled surface pits, and other features can be located by comparison with later maps. Many topographic maps are available in digital format for computer analysis and manipulation. Image files of an entire 7-1/2 min (1:24,000) topographic map, for example, can be purchased. Digital elevation maps (DEM) provide a regular grid of elevation points that allow the user to reproduce the topography in a variety of display formats.

(1) Optimum use of topographic maps involves the examination of large- and small-scale maps. Certain features, such as large geologic structures, may be apparent on small-scale maps only. Conversely, the interpretation of active geomorphic processes will require accurate, large-scale maps with a small-contour interval. As a general rule, the interpretation of topographic maps should proceed from small-scale (large-area) maps through intermediate-scale maps to large-scale (small-area) maps as the geologic investigation proceeds from the general to the specific.

(2) Certain engineering geology information can be inferred from topographic maps by proper interpretation of landforms and drainage patterns. Topography tends to reflect the geologic structure, composition of the underlying rocks, and the geomorphic processes acting on them. The specific type of geomorphic processes and the length of time they have been acting on the particular geologic structure and rock type will control the degree to which these geologic features are evident on the topographic maps. Geologic features are not equally apparent on all topographic maps, and considerable skill and effort are required to arrive at accurate geologic interpretations. Analysis of aerial photographs in combination with large-scale topographic maps is an effective means to interpret the geology and geomorphology of a site. Information of geotechnical significance that may be obtained or inferred from aerial photographs and topographic maps includes physiography, general soil and rock types, rock structure, and geomorphic history.

c. Geologic maps. Surficial and "bedrock" geologic maps can be used to develop formation descriptions, formation contacts, gross structure, fault locations, and approximate depths to rock (U.S. Department of Interior 1977; Dodd, Fuller, and Clarke 1989). Maps of 1:250,000 scale or smaller are suitable for the development of regional geology because they can be used with remote sensing imagery of similar scale to refine regional geology and soils studies. Large-scale geologic maps (1:24,000) are available for some areas (Dodd, Fuller, and Clarke 1989). State geologic surveys, local universities, and geotechnical and environmental firms may be able to provide detailed geologic maps of an area. Large-scale geologic maps provide information such as local faults, orientations of joints, detailed lithologic descriptions, and details on depth to rock.

d. Mineral resource maps. Mineral resource maps produced by the USGS and state geological services are important sources of geologic information. For example, the USGS coal resources evaluation program includes preparation of geologic maps (7.5-min quadrangle areas) to delineate the quantity, quality, and extent of coal on Federal lands. The USGS and state geologic service maps provide information on oil and gas lease areas and metallic mineral resource areas. Mineral resource maps also include information on natural construction materials such as quarries and sand and gravel deposits. These maps can be used in estimating the effects of proposed projects on mineral resources (such as access for future recovery, or reduction in project costs by recovery during construction).

e. Hydrologic and hydrogeologic maps. Maps showing hydrologic and hydrogeologic information provide a valuable source of data on surface drainage, well locations, ground water quality, ground water level contours, seepage patterns, and aquifer locations and characteristics. The USGS (Dodd, Fuller, and Clarke 1989), state geologic surveys, local universities, and geotechnical and environmental firms may provide this information.

f. Seismic maps. Krinitzsky, Gould, and Edinger (1993) show the distribution of seismic source areas for the United States and potential magnitude of earthquakes associated with each zone. Maps showing the timing and location of >4.5 magnitude earthquakes in the United States for the period 1857-1989 have been published by Stover and Coffman (1993).

(1) The Applied Technology Council (1978) published a seismic coefficient map for the United States for both velocity-based and acceleration-based coefficients. Seismic coefficients are dimensionless units that are the ratio between the acceleration associated with a particular frequency of ground motion and the response in a structure with the acceleration of the ground (Krinitzsky 1995). For the same ground motion frequency, seismic coefficients systematically vary for different types of structures (e.g., dams, embankments, buildings). These coefficients include a judgmental factor, representing experience on the part of structural engineers.

(2) Spectral Acceleration (%g) Maps for various periods of ground motion are being generated to assess seismic hazard potential. The Building Seismic Safety Council will publish updated seismic hazard potential maps in 1997 that will be in the form of spectral values for periods of 0.3 and 1.0 sec (E. L. Krinitzsky, personal communication 1996).

g. Engineering geology maps. An engineering geology map for the conterminous United States has been published by Radbruch-Hall, Edwards, and Batson (1987). Regional engineering geology maps are also available. More detailed maps may be available from state geologic surveys. Dearman (1991) describes the principles of engineering geologic mapping.

3-5. Remote Sensing Methods

Conventional aerial photographs and various types of imagery can be used effectively for large-scale regional interpretation of geologic structure, analyses of regional lineaments, drainage patterns, rock types, soil characteristics, erosion features, and availability of construction materials (Rasher and Weaver 1990, Gupta 1991). Geologic hazards, such as faults, fracture patterns, subsidence, and sink holes or slump topography, can also be recognized from air photo and imagery interpretation, especially from stereoscopic examinations of photo pairs. Technology for viewing stereoscopic projections on the PC is available. Detailed topographic maps can be generated from aerial photography that have sufficient surveyed ground control points. Remote sensing images that are in digital format can be processed to enhance geologic features (Gupta 1991). Although it is normally of limited value to site-specific studies, satellite imagery generated by Landsat, Sky Lab, the Space Shuttle, and the French Satellite Pour l'Observation de la Terre (SPOT) satellites are useful for regional studies. Remote sensing methods listed below can be used to identify and evaluate topographic, bathymetric, and subsurface features:

a. Topographic/surface methods.

(1) Airborne photography (mounted on helicopter or conventional aircraft).

(2) Airborne spectral scanner (mounted on helicopter or conventional aircraft).

(3) Photogrammetry (for imagery processing or mapping of airborne/satellite spectral scanned data).

(4) Satellite spectral scanner (e.g., Landsat).

(5) Satellite synthetic aperture radar (SAR).

(6) Side-looking airborne radar (SLAR).

b. Topographic/subsurface methods.

(1) Ground Penetrating Radar (GPR).

(2) Seismic.

(3) Gravitometer.

(4) Magnetometer.

c. Bathymetric methods.

(1) Fathometer (vessel mounted).

(2) Side-scan sonar (vessel mounted or towed).

(3) Seismometer/subbottom profiler (bathymetry subsurface, vessel mounted, or towed).

(4) Magnetometer (vessel towed).

(5) Gravitometer (vessel towed).

(6) Remotely Opertated Vehicle (ROV) mounted video or acoustic sensor.

(7) SeaBat (multibeam echo sounder) technology.

Gupta (1991) provides more detailed discussions of remote sensing techniques and their application to geotechnical investigations. Additional information concerning remote sensing surveying of bathymetry can be obtained in EM 1110-2-1003. Additional information concerning photographic imaging and photogrammetric mapping can be obtained in EM 1110-1-1000.

Section III
Field Reconnaissance and Observations

3-6. Field Reconnaissance

After a complete review of available geotechnical data, a geologic field reconnaissance should be made to gather information that can be obtained without subsurface explorations or detailed study (Dearman 1991). It is desirable that the geological field reconnaissance be conducted as part of a multidisciplinary effort. The composition of a team would depend upon the type and size of the project, the project effect on the area in question, and on any special problems identified as a result of early office studies. The team should include engineering geologists, soils engineers, planning engineers, archeologists, and representatives of other disciplines as appropriate. Duties include: field checking existing maps, cursory surface mapping (aided by aerial photographs), examining nearby natural and man-made outcrops, and traversing local waterways that expose rock and soil.

3-7. Observations

Observations made during field reconnaissances can be divided into five categories:

a. Examination of geologic/hydrologic features and geologic hazards to confirm, correct, or extend those identified during early office studies, and the preparation of regional geologic maps.

b. Assessment of site accessibility, ground conditions, and right-of-entry problems that could affect field exploration work.

c. Identification of cultural features that could affect exploration work and site location, especially utilities.

d. Evaluation of the condition of existing structures and construction practices that would indicate problem soil and rock conditions.

e. Identification of areas that could be contaminated by HTRW.

(1) Observations of geologic features should include rock outcrops and soil exposures to verify or refine available geologic maps. The strike and dip of major joint sets and evidence of joint sheeting or steeply dipping beds that would affect the stability of natural or excavated slopes should be noted. Indications of slope instability such as scarps, toe bulges, leaning trees, etc. should also be recorded. Table 3-2 outlines special geologic features and conditions which should be considered. The location of

sources of construction materials, such as large stone, sand and gravel deposits, clay soils, and active or abandoned quarries, are also important. Observable hydrologic features include surface drainage flow, springs and seeps in relation to formation members, and marshy or thick vegetation areas indicating high ground water tables.

(2) The location of cultural features, such as power lines, pipelines, access routes, and ground conditions that could restrict the location of or access to borings, should be noted. Historical and archaeological sites that may impact site location or construction practices should be identified and noted for further cultural resource potential studies. Local construction practices and the condition of existing structures and roads should be observed and potential problems noted. The location of abandoned mine workings such as adits, benches, shafts, and tailings piles should be noted.

(3) Field observations have special value in planning subsequent investigations and design studies because adverse subsurface conditions often can be anticipated from surface evidence and the regional geology. Suitable alternatives for foundation or structure types may be suggested by comprehensive field observations.

(4) Field reconnaissance can identify the need for new mapping and new aerial photographic coverage. Such coverage should be coordinated with planners early in the study process to ensure sufficient and timely coverage.

(5) Any potential environmental hazard such as former landfills, surface impoundments, mining activity, industrial sites, signs of underground storage tanks, or distressed vegetation should be recorded and assessed for HTRW potential.

Section IV
Information Development

3-8. Summary

Compiled and properly interpreted regional geologic data, coupled with information obtained during field reconnaissances, will provide the information necessary to identify suitable sites and to determine the scope of site investigations. Specifically, regional geology and site reconnaissance studies should result in the following:

a. Regional geologic conditions identified and incorporated into a regional geologic map.

b. Preliminary assessment made of regional seismicity.

c. Tentative location of sources of construction materials.

d. Tentative models of geologic conditions at suitable sites.

e. Input for Environmental Impact Statement.

f. Identification and assessment of potential for HTRW at prospective sites.

Table 3-2
Special Geologic Features and Conditions Considered in Office Studies and Field Observations

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
Landslides	Stability of natural and excavated slopes	Determine presence or age in project area or at construction sites Compute shear strength at failure. Do failure strengths decrease with age of slopes-- especially for clays and clay shales?	Estimate areal extent (length and width) and height of slope Estimate ground slope before and after slide (may correspond to residual angle of friction) Check highway and railway cuts and deep excavations, quarries, and steep slopes	Are landslides found offsite in geologic formations of same type that will be affected by project construction? What are probable previous and present ground water levels? Do trees slope in an unnatural direction?
Faults and faulting; past seismic activity	Of decisive importance in seismic evaluations; age of most recent fault movement may determine seismic design earthquake magnitude, may be indicative of high state of stress which could result in foundation heave or overstress in underground works	Determine existence of known faults and fault history from available information Examine existing boring logs for evidence of faulting from offset of strata	Verify presence at site, if possible, from surface evidence; check potential fault traces located from aerial imagery Make field check of structures, cellars, chimneys, roads, fences, pipelines, known faults, caves, inclination of trees, offset in fence lines	Are lineaments or possible fault traces apparent from regional aerial imagery?
Stress relief cracking and valley rebounding	Valley walls may have cracking parallel to valley. Valley floors may have horizontal cracking. In some clay shales, stress relief from valley erosion or glacial action may not be complete	Review pertinent geologic literature and reports for the valley area. Check existing piezometer data for abnormally low levels in valley sides and foundation; compare with normal ground water levels outside valley	Examine wells and piezometers in valleys to determine if levels are lower than normal ground water regime (indicates valley rebound not complete)	
Sinkholes; karst topography	Major effect on location of structures and feasibility of potential site (item 13)	Examine air photos for evidence of undrained depressions	Locate depressions in the field and measure size, depth, and slopes. Differences in elevation between center and edges may be almost negligible or many feet. From local residents, attempt to date appearance of sinkhole	Are potentially soluble rock formations present such as limestone, dolomite, or gypsum? Are undrained depressions present that cannot be explained by glaciation? Is surface topography rough and irregular without apparent cause?

(Sheet 1 of 4)

Table 3-2 (Continued)

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
Anhydrites or gypsum layers	Anhydrites in foundations beneath major structures may hydrate and cause expansion, upward thrust and buckling Gypsum may cause settlement, subsidence, collapse or piping. Solution during life of structure may be damaging	Determine possible existence from available geologic information and delineate possible outcrop locations	Look for surface evidence of uplift; seek local information on existing structures Check area carefully for caves or other evidence of solution features	Are uplifts caused by possible hydrite expansion or "explosion"?
Caves	Extent may affect project feasibility or cost. Can provide evidence regarding faulting that may relate to seismic design. Can result from unrecorded mining activity in the area		Observe cave walls carefully for evidence of faults and of geologically recent faulting. Estimate age of any broken stalactites or stalagmites from column rings	Are any stalactites or stalagmites broken from apparent ground displacement or shaking?
Erosion resistance	Need for total or partial channel slope protection is determined	Locate contacts of potentially erosive strata along drainage channels	Note stability of channels and degree of erosion and stability of banks	Are channels stable or have they shifted frequently? Are banks stable or easily eroded? Is there extensive bank sliding?
Internal erosion	Stability of foundations and dam abutments affected. Gravelly sands or sands with deficiency of intermediate particle sizes may be unstable and develop piping when subject to seepage flow	Locate possible outcrop areas of sorted alluvial materials or terrace deposits	Examine seepage outcrop areas of slopes and riverbanks for piping	
Area subsidence	Area subsidence endangers long- term stability and performance of project	Locate areas of high ground water withdrawal, oil fields and subsurface solution mining of underground mining areas	Check project area for new wells or new mining activity	Are there any plans for new or increased recovery of subsurface water or mineral resources?
Collapsing soils	Need for removal of shallow foundation materials that would collapse upon wetting determined	Determine how deposits were formed during geologic time and any collapse problems in area	Examine surface deposits for voids along eroded channels, especially in steep valleys eroded in fine-grained sedimentary formations	Were materials deposited by mud flows?

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Table 3-2 (Continued)

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
Locally lowered ground water	May cause minor to large local and area settlements and result in flooding near rivers or open water and differential settlement of structures	Determine if heavy pumping from wells has occurred in project area; contact city and state agencies and USGS	Obtain ground water levels in wells from owners and information on withdrawal rates and any planned increases. Observe condition of structures. Contact local water plant operators	
Abnormally low pore water pressures (lower than anticipated from ground water levels)	May indicate effective stresses are still increasing and may cause future slope instability in valley sites	Compare normal ground water levels with piezometric levels if data are available		Is the past reduction in vertical stresses a possible cause of low pore water pressure. Examples are deep glacial valleys and deep excavations like that for the Panama Canal, where pore pressures in clay shale were reduced by stress relief.
In situ shear strength from natural slopes	Provides early indication of stability of excavated slopes or abutment and natural slopes around reservoir area	Locate potential slide areas. Existing slope failures should be analyzed to determine minimum in situ shear strengths	Estimate slope angles and heights, especially at river bends where undercutting erosion occurs. Determine if flat slopes are associated with mature slide or slump topography or with erosion features	Are existing slopes consistently flat, indicating residual strengths have been developed?
Swelling soils and shales	Highly preconsolidated clays and clay shales may swell greatly in excavations or upon increase in moisture content	Determine potential problem and location of possible preconsolidated strata from available information	Examine roadways founded on geologic formations similar to those at site. Check condition of buildings and effects of rainfall and watering	Do seasonal ground water and rainfall or watering of shrubs or trees cause heave or settlement?
Varved clays	Pervious layers may cause more rapid settlement than anticipated. May appear to be unstable because of uncontrolled seepage through pervious layers between overconsolidated clay layers or may have weak clay layers. May be unstable in excavations unless well points are used to control ground water	Determine areas of possible varved clay deposits associated with prehistoric lakes. Determine settlement behavior of structures in the area	Check natural slopes and cuts for varved clays; check settlement behavior of structures	

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Table 3-2 (Concluded)

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
Dispersive clays	A major factor in selecting soils for embankment dams and levees	Check with Soil Conservation Service and other agencies regarding behavior of existing small dams	Look for peculiar erosional features such as vertical or horizontal cavities in slopes or unusual erosion in cut slopes. Perform “crumb” test	
Riverbank and other liquefaction areas	Major effect on riverbank stability and on foundation stability in seismic areas	Locate potential areas of loose fine-grained alluvial or terrace sand; most likely along riverbanks where loose sands are present and erosion is occurring	Check riverbanks for scallop-shaped failure with narrow neck (may be visible during low water). If present, determine shape, depth, average slope, and slope of adjacent sections. Liquefaction in wooded areas may leave trees inclined at erratic angles. Look for evidence of sand boils in seismic areas	
Filled areas	Relatively recent filled areas would cause large settlements. Such fill areas may be overgrown and not detected from surface or even subsurface evidence	Check old topo maps if available for depressions or gullies not shown on more recent topo maps	Obtain local history of site from area residents	
Local overconsolidation from previous site usage	Local areas of a site may have been overconsolidated from past heavy loadings of lumber or material storage piles			Obtain local history from residents of area

(Sheet 4 of 4)

If an information system or electronic data base management capability already exists, serious consideration should be given at the beginning of a project to incorporate the geotechnical information into the system. Newly obtained/generated data should be incorporated into the information system data base for future project use. Data describing the site conditions encountered during construction should also be incorporated into the geotechnical data base. By electronically recording geotechnical information through the life cycle of a project, reliable diagnostic and forensic analysis can be conducted. Moreover, a GIS provides a powerful management tool for the postconstruction (operation and maintenance) phase of the project.

Chapter 4
Surface Investigations

4-1. Description of Operation

This chapter describes field operations that do not involve significant disturbance of the ground at the time the investigation is conducted. This type of investigation typically occurs at a preliminary stage of projects and supplies generalized information. However, these investigations can involve mapping specific locations in great detail during construction. The end product is commonly a pictorial rendering of conditions at the site. The degree of accuracy and precision required in such a rendering varies with the application and purpose for which the information is to be used. Some leeway in the degree of accuracy is required because of the inherent difficulty in presenting a 3-D subject in two dimensions. With computer-aided design and drafting (CADD) systems and specialized engineering application software packages, it is possible to portray 3-D information of greater complexity more effectively.

This chapter and the next describe in detail elements necessary for completion of a successful field investigation program for large civil and military projects. Several elements are applicable for refinement of regional geology investigations discussed in Chapter 3. Many civil works projects are, however, too small to afford complete onsite field investigations as outlined in the next two chapters. For smaller projects, emphasis should be placed on compilation and analysis of existing data, remote sensing imagery, and surface and subsurface information derived from onsite drilling and construction excavations. The following discussion can serve as a guide to the types of critical geotechnical information needed to support design and construction decisions.

Section I
Geologic Field Mapping

4-2. Areal Mapping

The purpose of areal mapping is to develop an accurate picture of the geologic framework of the project area. The area and the degree of detail to be mapped can vary widely depending on the type and size of the project and on the complexity of the regional geology. In general, the area to be mapped should include the project site(s) as well as the surrounding area that could influence or could be influenced by the project. The information available from other sources should have been identified and collected during preliminary investigations (Chapter 3). If this was not done, or if for any reason it appears that additional useful information may be available, this information should be obtained and evaluated before expensive field investigations are begun. Only if existing geologic studies of an area have been combined with current geologic mapping and appropriate remote sensing techniques can an areal mapping program be considered complete. Such analysis is best carried out in a GIS. Utilization of GPS is a low cost and efficient alternative for providing precise horizontal and vertical measures to establish ground control points for georeferencing remote sensing images and for locating monitoring wells and other geologic sampling stations. GPS procedures are described in EM 1110-2-1003. For initial surface mapping, hand-held electronic distance measuring instruments are commonly sufficiently accurate and are efficient.

a. Reservoir projects. Geologic and environmental features within the reservoir and adjacent areas that should be studied and mapped include the following:

(1) Faults, joints, stratigraphy, and other significant geologic features.

(2) Karst topography or other features that indicate high reservoir leakage potential.

(3) Water well levels, springs, surface water, water-sensitive vegetation, or other evidence of the ground water regime.

(4) Soluble or swelling rocks such as gypsum or anhydrite.

(5) Potential landslide areas around the reservoir rim.

(6) Valuable mineral resources.

(7) Mine shafts, tunnels, and gas and oil wells.

(8) Potential borrow and quarry areas and sources of construction materials.

(9) Shoreline erosion potential.

(10) Landfills, dumps, underground storage tanks, surface impoundments, and other potential environmental hazards.

b. *Other projects.* The geologic features listed above are applicable in part to navigation locks and dams, main-line levees, coastal and harbor protection projects, and large or complex military projects. However, the scope and detail of the area mapped depend on the type and size of the project. Environmental engineering aspects of site investigations are covered in EM 1110-2-1202, -1204, -1205, and -1206 and Keller (1992). Procedures to investigate sedimentation in river and reservoir sites are discussed in EM 1110-2-4000.

4-3. Site Mapping

Large-scale and detailed geologic maps should be prepared for specific sites of interest within the project area and should include proposed structure areas and borrow and quarry sites. Investigation of the geologic features of overburden and rock materials is essential in site mapping and subsequent explorations. Determination of the subsurface features should be derived from a coordinated, cooperative study by geotechnical engineers and geologists. The geologist should contribute information on origin, distribution, and manner of deposition of the overburden and rock. The geotechnical engineer or engineering geologist should determine the engineering properties of the site foundation and potential construction materials, potential problem materials or conditions, application of geologic conditions to design, and the adaptation of proposed structures to foundation conditions.

a. *Structure sites.* A good preliminary geologic map should be prepared prior to making any subsurface borings to provide an approximate picture of the geologic conditions and hazards at a site. Such a map permits borings to be strategically located. For each proposed boring, an estimate should be made of the subsurface conditions that will be encountered, such as depths to critical contacts and to the water table. This estimate is possible, at least in an approximate manner, if geologic mapping has been performed to determine the geologic structure, lithology, and stratigraphy. The process of progressively refining the model of the geologic structure and stratigraphy by comparison with boring information is the most efficient and cost-effective means to develop a complete understanding of the geologic site conditions. A digital format, such as CADD, provides a cost- and time-effective way to refine the model as new information becomes available.

b. Borrow and quarry sites. Sources of materials for embankment construction, riprap protection, and aggregates for concrete or road construction can often be located and evaluated during the course of regional mapping. It is sometimes necessary to expand the field area to locate suitable types and quantities of construction materials. In these instances, remote sensing techniques including analysis of aerial photography may be useful. Alternate plans that would make use of materials nearer to the project but lower in quality should be tentatively formulated and evaluated. A complete borrow and quarry source map should include all soil types encountered and all rock types with adequate descriptions of surficial weathering, hardness, and joint spacings.

(1) Processed rock products are usually most economically acquired from commercial sources. Test results are often available on these sources through state or Federal offices. The procedures for approval of construction materials sources are outlined in EM 1110-2-2301.

(2) Evaluation of soil and/or rock sources should be based upon sampling and laboratory analysis. By making field estimates of the thicknesses of various deposits, a geologic map may be used to estimate quantities available. Geologic maps can also be used to make a preliminary layout of haul and access roads and to estimate haul distances. A GIS is ideally suited to evaluate the quality and quantity of available quarry material, cost of excavation, and optimal transport routes.

4-4. Construction Mapping

Construction maps record in detail geologic conditions encountered during construction. Traditionally, a foundation map is a geologic map with details on structural, lithologic, and hydrologic features. It can represent structure foundations, cut slopes, and geologic features in tunnels or large chambers. The map should be prepared for soil and rock areas and show any feature installed to improve, modify, or control geologic conditions. Some examples are rock reinforcing systems, permanent dewatering systems, and special treatment areas. The mapping of foundations is usually performed after the foundation has been cleaned just prior to the placement of concrete or backfill. The surface cleanup at this time is generally sufficient to permit the observation and recording of all geologic details in the foundation. An extensive photographic and videographic record should be made during foundation mapping.

a. The person in charge of foundation mapping should be familiar with design intent via careful examination of design memoranda and discussion with design personnel. The actual geology should be compared with the geologic model developed during the design phase to evaluate whether or not there are any significant differences and how these differences may affect structural integrity. The person in charge of foundation mapping should be involved in all decisions regarding foundation modifications or additional foundation treatment considered advisable based on conditions observed after preliminary cleanup. Design personnel should be consulted during excavation work whenever differences between the actual geology and the design phase geological model require clarification or change in foundation design. Mapping records should include details of all foundation modifications and treatment performed.

b. Geologic maps and sections of the project which relate to construction and postconstruction procedures, hazards, or problems should be prepared for the Construction Foundation Report. Also, an edited video recording of excavation procedures, final foundation surfaces, treatment, etc. should be an integral part of the final report. The various geological data layers and video information are best compiled, analyzed, and prepared for presentation in a GIS.

c. Appendix B provides detailed guidance on technical procedures for mapping foundations. Mapping of tunnels and other underground openings must be planned differently from foundation mapping. Design requirements for support of the openings may require installation of support before an adequate cleanup can be made for mapping purposes. Consequently, mapping should be performed as the heading or opening is advanced and during the installation of support features. This requires a well trained geologist, engineering geologist, or geological engineer at the excavation at all times. Specifications should be included in construction plans for periodic cleaning of exposure surfaces and to allow a reasonable length of time for mapping to be carried out. Technical procedures for mapping tunnels are outlined in Appendix C and can be modified for large chambers.

Section II
Surface Geophysical Explorations

4-5. Background

Geophysical exploration consists of making indirect measurements from the earth's surface or in boreholes to obtain subsurface information. Geologic information is obtained through analysis or interpretation of these measurements. Boreholes or other subsurface explorations are needed to calibrate geophysical measurements. Geophysical explorations are of greatest value when performed early in the field exploration program in combination with limited subsurface exploration. They are appropriate for a rapid location and correlation of geologic features such as stratigraphy, lithology, discontinuities, ground water, and the in situ measurement of elastic moduli and densities. The cost of geophysical explorations is generally low compared with the cost of core borings or test pits, and considerable savings may be realized by judicious use of these methods.

4-6. Methods

The six major geophysical exploration methods are seismic, electrical resistivity, sonic, magnetic, radar, and gravity. Of these, the seismic and electrical resistivity methods have found the most practical application to the engineering problems of the Corps of Engineers (Steeples and Miller 1990, Society of Exploration Geophysicists 1990). Potential applications of selected geophysical methods are summarized in Tables 4-1 and 4-2. EM 1110-1-1802, Society of Exploration Geophysicists (1990), and Annan (1992) provide detailed guidance on the use and interpretation of geophysical methods. Special applications of microgravimetric techniques for sites with faults, fracture zones, cavities, and other rock irregularities have been made (Butler 1980).

Table 4-1
Applications of Selected Geophysical and Other Methods for Determination of Engineering Parameters[1]

Method	Basic Measurement	Application	Advantages	Limitations
		Surface		
Refraction seismic	Travel time of compressional waves through subsurface layers	Velocity determination of compression wave through subsurface. Depths to contrasting interfaces and geologic correlation of horizontal layers	Rapid, accurate, and relatively economical technique. Interpretation theory generally straightforward and equipment readily available	Incapable of detecting material of lower velocity underlying higher velocity. Thin stratum sometimes not detectable. Interpretation is not unique
Reflection seismic	Travel time of compressional waves reflected from subsurface layers	Mapping of selected reflector horizons. Depth determinations, fault detection, discontinuities, and other anomalous features	Rapid, thorough coverage of given site area. Data displays highly effective	Even with recent advances in high-resolution, seismic technology applicable to civil works projects is limited in area of resolution
Rayleigh wave dispersion	Travel time and period of surface Rayleigh waves	Inference of shear wave velocity in near-surface materials	Rapid technique which uses conventional refraction seismographs	Requires long line (large site). Requires high-intensity seismic source rich in low-frequency energy. Interpretation complex
Vibratory (seismic)	Travel time or wavelength of surface Rayleigh waves	Inference of shear wave velocity in near-surface materials	Controlled vibratory source allows selection of frequency, hence wavelength and depth of penetration (up to 200 ft). Detects low-velocity zones underlying strata of higher velocity. Accepted method	Requires large vibratory source, specialized instrumentation, and interpretation
Reflection profiling (seismic-acoustic)	Travel times of compressional waves through water and subsurface materials and amplitude of reflected signal	Mapping of various lithologic horizons; detection of faults, buried stream channels, and salt domes, location of buried man-made objects; and depth determination of bedrock or other reflecting horizons	Surveys of large areas at minimal time and cost; continuity of recorded data allows direct correlation of lithologic and geologic changes; correlative drilling and coring can be kept to a minimum	Data resolution and penetration capability are frequency-dependent; sediment layer thickness and/or depth to reflection horizons must be considered approximate unless true velocities are known; some bottom conditions (e.g., organic sediments) prevent penetration; water depth should be at least 15 to 20 ft for proper system operation

[1] From EM 1110-1-1802.

(Sheet 1 of 5)

Table 4-1 (Continued)

Method	Basic Measurement	Application	Advantages	Limitations
		Surface (Continued)		
Electrical resistivity	Electrical resistance of a volume of material between probes	Complementary to refraction seismic. Quarry rock, ground water, sand and gravel prospecting. River bottom studies and cavity detection	Economical nondestructive technique. Can detect large bodies of "soft" materials	Lateral changes in calculated resistance often interpreted incorrectly as depth related; hence, for this and other reasons, depth determinations can be grossly in error. Should be used in conjunction with other methods, e.g., seismic
Acoustic (resonance)	Amplitude of acoustically coupled sound waves originating in an air-filled cavity	Traces (on ground surface) lateral extent of cavities	Rapid and reliable method. Interpretation relatively straightforward. Equipment readily available	Still in experimental stage - limits not fully established. Must have access to some cavity opening
Ground Penetrating Radar	Travel time and amplitude of a reflect signal microwave	Rapidly profiles layering conditions. Stratification, dip, water table, and presence of many types of anomalies can be determined	Very rapid method for shallow site investigations. Online digital data processing can yield "onsite" look. Variable density display highly effective	Transmitted signal rapidly attenuated by water. Severely limits depth of penetration. Multiple reflections can complicate data interpretation
Gravity	Variations in gravitational field	Detects anticlinal structures, buried ridges, salt domes, faults, and cavities	Reasonably accurate results can be obtained, provided extreme care is exercised in establishing gravitational references	Equipment very costly. Requires specialized personnel. Anything having mass can influence data (buildings, automobiles, etc). Data reduction and interpretation are complex. Topography and strata density influence data
Magnetic	Variations of earth's magnetic field	Determines presence and location of magnetic materials in the subsurface. Locates ore bodies	Minute quantities of magnetic materials are detectable	Only useful for locating magnetic materials. Interpretation highly specialized. Calibration on site extremely critical. Presence of any metallic objects influences data
		Borehole		
Uphole/downhole (seismic)	Vertical travel time of compressional and/or shear waves	Velocity determination of vertical P- and/or S-waves. Identification of low-velocity zones	Rapid technique useful to define low-velocity strata. Interpretation straightforward	Care must be exercised to prevent undesirable influence of grouting or casing

(Sheet 2 of 5)

Table 4-1
(Continued)

Method	Basic Measurement	Application	Advantages	Limitations
		Borehole (Continued)		
Crosshole (seismic)	Horizontal travel time of compressional and/or shear waves	Velocity determination of horizontal P- and/or S-waves. Elastic characteristics of subsurface strata can be calculated	Generally accepted as producing reliable results. Detects low-velocity zones provided borehole spacing not excessive	Careful planning with regard to borehole spacing based upon geologic and other seismic data an absolute necessity. Snell's law of refraction must be applied to establish zoning. A borehole deviation survey must be run. Highly experienced personnel required. Repeatable source required
Borehole spontaneous potential	Natural earth potential	Correlates deposits, locates water resources, studies rock deformation, assesses permeability, and determines ground water salinity	Widely used, economical tool. Particularly useful in the identification of highly porous strata (sand, etc.)	Log must be run in a fluid filled, uncased boring. Not all influences on potentials are known
Single-point resistivity	Strata electrical resistance adjacent to a single electrode	In conjunction with spontaneous potential, correlates strata and locates porous materials	Widely used, economical tool. Log obtained simultaneous with spontaneous potential	Strata resistivity difficult to obtain. Log must be run in a fluid filled, uncased boring. Influenced by drill fluid
Long and short normal resistivity	Near-hole electrical resistance	Measures resistivity within a radius of 16 and 64 in.	Widely used, economical tool	Influenced by drill fluid invasion. Log must be run in a fluid filled, uncased boring
Lateral resistivity	Far-hole electrical resistance	Measures resistivity within a radius of 18.7 ft	Less drill fluid invasion influence	Log must be run in a fluid filled, uncased boring. Investigation radius limited in low-moisture strata
Induction resistivity	Far-hole electrical resistance	Measures resistivity in air- or oil-filled holes	Log can be run in a nonconductive casing	Large, heavy tool
Borehole imagery (acoustic)	Sonic image of borehole wall	Detects cavities, joints, fractures in borehole wall. Determine attitude (strike and dip) of structures	Useful in examining casing interior. Graphic display of images. Fluid clarity immaterial	Highly experienced operator required. Slow log to obtain. Probe awkward and delicate. Borehole must be less than a 6-in. diam

(Sheet 3 of 5)

Table 4-1 (Continued)

Method	Basic Measurement	Application	Advantages	Limitations
		Borehole (Continued)		
Continuous sonic (3-D) velocity	Time of arrival of P- and S-waves in high-velocity materials	Determines velocity of P- and S-waves in near vicinity of borehole. Potentially useful for cavity and fracture detection. Modulus determinations. Sometimes S-wave velocities are inferred from P-wave velocity and concurrently run nuclear logs through empirical correlations	Widely used method. Rapid and relatively economical. Variable density display generally impressive. Discontinuities in strata detectable	Shear wave velocity definition questionable in unconsolidated materials and soft sedimentary rocks. Only P-wave velocities greater than 5,000 fps can be determined
Natural gamma radiation	Natural radioactivity	Lithology, correlation of strata, may be used to infer permeability. Locates clay strata and radioactive minerals	Widely used, technically simple to operate and interpret	Borehole effects, slow logging speed, cannot directly identify fluid, rock type, or porosity. Assumes clay minerals contain potassium 40 isotope
Gamma-gamma density	Electron density	Determines rock density of subsurface strata	Widely used. Can be applied to quantitative analyses of engineering properties. Can provide porosity	Borehole effects, calibration, source intensity, and chemical variation in strata affect measurement precision. Radioactive source hazard
Neutron porosity	Hydrogen content	Moisture content (above water table) Total porosity (below water table)	Continuous measurement of porosity. Useful in hydrology and engineering property determinations. Widely used	Borehole effects, calibration, source intensity, and bound water all affect measurement precision. Radioactive source hazard
Neutron activation	Neutron capture	Concentration of selected radioactive materials in strata	Detects elements such as U, Na, Mn. Used to determine oil-water contact (oil industry) and in prospecting for minerals (Al, Cu)	Source intensity, presence of two or more elements having similar radiation energy affect data
Borehole magnetic	Nuclear precession	Deposition, sequence, and age of strata	Distinguishes ages of lithologically identical strata	Earth field reversal intervals under study. Still subject of research
Mechanical caliper	Diameter of borehole	Measures borehole diameter	Useful in a wet or dry hole	Must be recalibrated for each run. Averages three diameters
Acoustic caliper	Sonic ranging	Measures borehole diameter	Large range. Useful with highly irregular shapes	Requires fluid filled hole and accurate positioning

(Sheet 4 of 5)

Table 4-1 (Concluded)

Method	Basic Measurement	Application	Advantages	Limitations
		Borehole (Continued)		
Temperature	Temperature	Measures temperature of fluids and borehole sidewalls. Detects zones of inflow or fluid loss	Rapid, economical, and generally accurate	None of importance
Fluid resistivity	Fluid electrical resistance	Water-quality determinations and auxiliary log for rock resistivity	Economical tool	Borehole fluid must be same as ground water
Tracers	Direction of fluid flow	Determines direction of fluid flow	Economical	Environmental considerations often preclude use of radioactive tracers
Flowmeter	Fluid velocity and quantity	Determines velocity of subsurface fluid flow and, in most cases, quantity of flow	--	--
Sidewall sampling	--	--	--	--
Fluid sampling	--	--	--	--
Borehole dipmeter	--	--	--	--
Borehole surveying	Azimuth and declination of borehole drift	Determines the amount and direction of borehole deviation from the vertical normal	A reasonably reliable technique. Method must be used during the conduct of crosshole surveys to determine distance between seismic source and receivers	Errors are cumulative, so care must be taken at each measurement point to achieve precise data
Downhole flow meter	Flow across the borehole	Determines the rate and direction of ground water flow	A reliable, cost-effective method to determine lateral foundation leakage under concrete structures	Assumes flow not influenced by emplacement of borehole

Note: Blanks indicate no data.

(Sheet 5 of 5)

Table 4-2
Numerical Rating of Geophysical Methods to Provide Specific Engineering Parameters[1] Engineering Application

Geophysical Method	Depth to Rock	P-Wave Velocity	S-Wave Velocity	Shear Modulus	Young's Modulus	Poisson's Ratio	Lithology	Material Boundaries Stratigraphy	Dip of Strata	Density	In Situ State of Stress	Temperature	Permeability	Percent Saturation	Ground water Table	Ground water Quality	Ground water Aquifers	Flow Rate and/or Direction	Borehole Diameter	Obstructions	Rippability	Fault Detection	Cavity Detection	Cavity Delineation	Location of Ore Bodies	Borehole Azimuth and Inclination
<u>Surface</u>																										
Refraction (seismic)	4	4	4	4	4	4	1	3	4	2	1	0	0	2	2	0	2	0	0	2	4	3	2	2	3	0
Reflection (seismic)	4	0	0	0	0	0	1	4	4	0	0	0	0	0	2	0	1	0	0	2	0	4	3	3	3	0
Rayleigh wave dispersion	1	0	2	2	0	0	1	3	0	2	1	0	0	0	0	0	0	0	0	1	0	0	0	1	2	0
Vibratory (seismic)	2	0	4	4	4	0	1	3	0	2	1	0	0	0	0	0	0	0	0	2	2	1	2	2	3	0
Reflection profiling (seismic-acoustic)	4	0	0	0	0	0	1	4	4	0	0	0	0	0	0	0	0	0	0	3	0	4	3	3	4	0
Electrical potential[2]	0	0	0	0	0	0	0	1	0	0	0	0	1	1	2	3	3	3	0	0	0	3	3	3	4	0
Electrical resistivity	3	0	0	0	0	0	1	3	2	0	0	0	2	1	4	0	4	2	0	3	2	0	4	4	4	0
Acoustic (resonance)[2]	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	0	0	4	0	0
Radar[2,3]	3	0	0	0	0	0	1	3	2	0	0	0	2	3	3	0	0	2	0	3	0	3	3	3	3	0
Electromagnetic[2]	4	0	0	0	0	0	3	4	1	0	0	0	1	2	3	1	2	0	0	0	0	3	0	0	4	0
Gravity	3	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	4	0	1	3	3	3	0
Magnetic[2,3]	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	2	2	4	0
<u>Borehole</u>																										
Uphole/downhole (seismic)	4	4	4	4	4	4	1	4	0	2	1	0	0	2	2	0	2	0	0	1	2	3	0	2	2	0
Crosshole (seismic)	4	4	4	4	4	4	1	4	2	2	1	0	0	2	2	0	2	0	0	3	2	3	3	2	3	0

(Continued)

[1] Numerical rating refers to applicability of method in terms of current use and future potential:
- 0 = Not considered applicable
- 1 = Limited
- 2 = Used or could be used, but not best approach
- 3 = Excellent potential but not fully developed
- 4 = Generally considered as excellent approach; state of art well developed
- A = In conjunction with other electrical and nuclear logs

[2] Methods not included in EM 1110-1-1802.

[3] Airborne or inhole survey capability not considered.

Table 4-2 (Concluded)

Geophysical Method	Depth to Rock	P-Wave Velocity	S-Wave Velocity	Shear Modulus	Young's Modulus	Poisson's Ratio	Lithology	Material Boundaries Stratigraphy	Dip of Strata	Density	In Situ State of Stress	Temperature	Permeability	Percent Saturation	Ground water Table	Ground water Quality	Ground water Aquifers	Flow Rate and/or Direction	Borehole Diameter	Obstructions	Rippability	Fault Detection	Cavity Detection	Cavity Delineation	Location of Ore Bodies	Borehole Azimuth and Inclination
<u>Borehole (Continued)</u>																										
Crosshole acoustic[2]	4	4	4	4	4	0	1	3	4	0	0	0	1	0	3	0	0	0	0	1	3	3	3	3	0	0
Crosshole resistivity[2]	3	0	0	0	0	0	1	3	1	0	0	0	1	0	3	0	3	0	0	1	0	2	3	3	0	0
Borehole spontaneous potential	2	0	0	0	0	0	4	4	4	2	0	0	0	0	4	2	4	0	0	1	0	2	2	1	3	0
Single-point resistivity	2	0	0	0	0	0	4	4	1	0	0	0	0	1	4	2	4	0	0	1	0	1	1	1	2	0
Long and short normal resistivity	2	0	0	0	0	0	4	4	1	1	0	0	0	4	3	0	2	0	0	0	0	1	1	2	4	0
Lateral resistivity	2	0	0	0	0	0	3	4	1	1	0	0	0	4	3	0	2	0	0	0	0	1	1	2	4	0
Induction-resistivity[2]	2	0	0	0	0	0	4	4	1	1	0	0	0	4	3	0	2	0	0	0	0	1	1	2	4	0
Borehole imagery acoustic	4	0	0	0	0	0	2	3	1	0	1	0	2	0	2	0	0	0	0	1	0	2	2	3	0	0
Interval (3-D) velocity	2	4	2	2	2	2	2	3	1	2	1	0	0	1	1	0	0	0	0	1	0	3	2	2	2	0
Natural gamma radiation	2	0	0	0	0	0	4	4	1	2	0	0	3A	1A	3A	2	2A	1	0	0	0	3A	1	1	4	0
Gamma-gamma density	3A	0	0	0	0	0	4	4	1	3A	0	0	2A	3A	2A	0	0	0	0	0	3	3A	2	1	4	0
Neutron porosity	2A	0	0	0	0	0	4	4	1	3A	0	0	2	3A	3A	0	0	0	0	0	0	3A	2	1	4	0
Neutron activation[2]	2A	0	0	0	0	0	3	1	1	0	0	0	2A	2	3A	0	0	2	0	1	0	1	0	0	4	0
Borehole gravity[2]	1	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	2	4	0	0
Mechanical caliper	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
Acoustic caliper	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	2	1	0	0	0	0	0	0
Temperature	0	0	0	0	0	0	0	0	0	0	1	4	1	0	2	4	4	2	0	0	0	0	1	2	1	0
Fluid resistivity	0	0	0	0	0	0	0	1	0	1	0	0	1	4	4	4	4	0	0	0	0	0	3	1	1	0
Tracers[2]	0	0	0	0	0	0	1	0	0	0	0	0	1	0	2	0	4	4	0	1	0	0	0	3	0	0
Flowmeter[2]	0	0	0	0	0	0	0	0	0	0	0	0	2	2	2	0	4	4	0	2	0	0	0	2	0	0
Sidewall sampling[2]	4	0	0	0	0	0	4	4	1	4	2	0	4	4	2	0	0	0	0	2	2	2	1	0	4	0
Fluid sampling[2]	0	0	0	0	0	0	0	0	0	0	0	4	1	0	4	4	4	2	0	0	0	0	0	0	0	0
Borehole dipmeter[2]	0	0	0	0	0	0	2	1	4	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	1	2
Borehole surveying	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4

Chapter 5
Subsurface Investigations

5-1. Background

Subsurface investigations require use of equipment to gain information below the ground surface. The equipment is typically invasive and requires disturbance of the ground to varying degrees. Most of these exploration techniques are relatively expensive and therefore should be carefully planned and controlled to yield the maximum amount of information possible. It should be kept in mind that the quality of the information produced can vary significantly. If procedures are not followed carefully and data not interpreted properly, radically different conclusions can be reached. For example, poor drilling techniques could produce samples that might yield lower strength values. Therefore, only competent, senior geotechnical personnel should be charged with planning a subsurface investigation, and only qualified geotechnical professionals and technicians should do the drilling and data collecting, reducing, analyzing, and interpreting.

5-2. Location of Investigations

An important piece of information for all geotechnical investigations that seems obvious but commonly not given sufficient attention is the accurate determination of the location of investigation. It is always preferable to select boring and test pit locations that fully characterize geotechnical conditions. Although correlation of information from offsite may be technically defensible, because of variability of geologic materials, the legal defensibility of a piece of information is commonly lost if it is even slightly removed from the site. Of course, it is not always possible to locate a boring on a structure because of obstacles or right-of-entry difficulties. Heavily urbanized areas present particular difficulties in these aspects. However, it is important to keep in mind that correlations and interpretations may be subject to later scrutiny should a change of conditions claim be filed. All locations should be determined using either conventional surveying methods or by a GPS (EM 1110-1-1003). A GPS has the significant advantage of having the positional information downloaded directly into a GIS.

5-3. Protection of the Environment

a. After the locations for field investigations work have been determined, routes of access to the area and the specific sites for borings and excavations should be selected with care to minimize damage to the environment. Environmental engineering aspects of civil works projects are discussed in EM 1110-2-1202, -1204, -1205, and -1206 and Keller (1992). Operation of equipment will be controlled at all times and the extent of damaged areas will be held to the minimum consistent with the requirements for obtaining adequate data. Local laws pertaining to permissible levels of sediment flow from the site should be investigated. After the exploratory sites have served their purpose, the disturbed areas will be restored to a natural appearance. All borings and test pits should be backfilled in accordance with state environmental regulations.

b. Most states are now the primary regulatory agency for ground water quality assurance. As part of this responsibility many now require the certification of drillers. These regulations primarily apply to water well installation, but they may also apply to investigation programs. Ground water quality assurance has been the subject of considerable discussion from the standpoint of Federal Government responsibility for compliance with these regulations. Generally, Government drillers are not required to have state certification but, in some instances, may be forced to comply for political reasons. This is not a clear-cut issue, and it should be resolved before beginning a drilling program.

c. The Federal Government has responsibility to ensure that environmental consciousness is maintained during the conduct of geotechnical investigations. Unfortunately, drilling rigs are inherently dirty. Proper maintenance of drilling rigs will minimize this problem. For HTRW exploratory drilling, drilling rigs must be steam cleaned and all tools, equipment, and personnel decontaminated in accordance with procedures established in the quality assurance and control (QAAC) plan. Fluids used in drilling operations, be they hydrocarbons that have leaked from the hydraulic system or a constituent of a drilling mud, are potentially toxic and should be controlled or eliminated wherever possible. EM 1110-1-4000 discusses requirements for maintenance and operation of drilling equipment at USACE HTRW sites. Aller et al. (1989) provide further guidance on acceptable design and installation of monitoring wells.

Section I
Borings

5-4. Major Uses

Borings are required to characterize the basic geologic materials at a project. The major uses for which borings are made are as follows:

a. Define geologic stratigraphy and structure.

b. Obtain samples for index testing.

c. Obtain ground water data.

d. Perform in situ tests.

e. Obtain samples to determine engineering properties.

f. Install instrumentation.

g. Establish foundation elevations for structures.

h. Determine the engineering characteristics of existing structures.

Borings are classified broadly as disturbed, undisturbed, and core. Borings are frequently used for more than one purpose, and it is not uncommon to use a boring for purposes not contemplated when it was made. Thus, it is important to have a complete log of every boring, even if there may not be an immediate use for some of the information. If there is doubt regarding the range of borehole use or insufficient information to determine optimum borehole size, then the hole should be drilled larger than currently thought needed. A slightly larger than needed borehole is considerably less expensive than a second borehole.

5-5. Boring and Sampling Methods

a. Common methods discussed. Many methods are used to make borings and retrieve samples. Some of the more common methods are discussed in the following paragraphs. Many of these are also discussed in detail in Chapter 3, Appendix F; Das (1994); Hunt (1984); and Aller et al. (1989). Some factors that affect the choice of methods are:

(1) Purpose and information required.

(2) Equipment availability.

(3) Depth of hole.

(4) Experience and training of available personnel.

(5) Types of materials anticipated.

(6) Terrain and accessibility.

(7) Cost.

(8) Environmental impacts.

(9) Disruption of existing structure.

b. Auger borings. Auger borings provide disturbed samples that are suitable for determining soil type, Atterberg limits, Proctor testing, and other index properties but generally give limited information on subsoil stratification, consistency, or sensitivity. Auger borings are most useful for preliminary investigations of soil type, advancing holes for other sampling methods, determining depth to top of rock, and for monitor well installation in soils. Auger borings can be made using hand, helical, barrel, hollow-stem, or bucket augers. Auger samples are difficult to obtain below the ground water table, except in clays. However, hollow-stem augers with a continuous split barrel sampler can retrieve some unconsolidated material from below the water table. Paragraph 3-4, Appendix F, describes the types of augers used in subsurface exploration. Paragraph 8-2, Appendix F, discusses sampling procedures when augering.

(1) Truck-mounted auger rigs currently come equipped with high yield and high tensile strength steel augers. New hydraulics technology can now apply torque pressures upward of 27,000 Nm (20,000 ft lb). With this amount of torque, augers are capable of boring large size holes and of being used in soft rock foundation investigations. Because augers use no drilling fluids, they are advantageous for avoiding environmental impacts. Appendix F, paragraph 3-3, describes auger drilling rigs. Another advantage of using augers is the ability (using hollow stems) for soil sampling, i.e., taking undisturbed samples below the bit.

(2) Currently, many drilling rigs are actually a combination of auger/core/downhole hammer units. A hollow-stem auger has the “drill through” capability (i.e., the auger can drill to refusal, then a wireline core barrel and drill rods can be inserted to finish the hole). The auger acts as a temporary casing to prevent caving of the softer materials as sampling progresses. However, the augers are not water tight and water loss should be anticipated. Hollow-stem augers should not be used as temporary casing in areas where HTRW is anticipated. Temporary steel casing driven into the surface of competent bedrock or PVC casing permanently grouted into the competent bedrock surface is required when HTRW is anticipated.

c. Drive borings. Drive borings provide disturbed samples that contain all soil constituents, generally retain natural stratification, and can supply data on penetration resistance. Drive boring is a nonrotating method for making a hole by continuous sampling using a heavy wall drive barrel. Push, or drive, samplers are of two types: open samplers and piston samplers. Open samplers have a vented sampler head attached to an open tube that admits soil as soon as the tube is brought in contact with the soil. Some open samplers are equipped with a cutting shoe and a sample retainer. Piston samplers have a movable piston located within the sampler tube. The piston helps to keep drilling fluid and soil cuttings out of the tube as the sampler advances. The piston also helps to retain the sample in the sampler tube. Where larger samples are required, the most suitable drill for this method is the cable tool rig. The cable tool rig has the capability to provide a downward driving force (drill stem on drive clamps) to make a hole and an upward force (drilling jars) to remove the drive barrel from the hole.

(1) Vibratory samplers offer a means of obtaining disturbed samples of saturated, cohesionless soils rapidly and with relatively inexpensive equipment (Appendix F). The simplest devices consist of a small gasoline engine providing hydraulic power to a vibrating head clamped to aluminum tubing secured on a tripod. The rapid vibrations within the head drives the sampling tube into the ground and forces the soil up into the tube. A rubber packer secured into the open end of the sampling tube after driving creates a seal to retain the sample as the tube is withdrawn with a hand winch.

(2) Another device, the Becker hammer drill, was devised specifically for use in sand, gravel, and boulders by Becker Drilling, LTD, Canada. The Becker drill uses a diesel-powered pile hammer to drive a special double-wall, toothed casing into the ground. Drilling fluid is pumped through an annulus to the bottom of the hole where it forces cuttings to the surface through the center of the casing. The cuttings are collected for examination. Becker drill casings are available in 14-cm (5.5-in.), 17-cm (6.6-in.), and 23-cm (9.0-in.) outside diameters (OD), with sampling inside diameters (ID) of 8.4 cm (3.3 in.), 10.9 cm (4.3 in.), and 15.2 cm (6.0 in.), respectively. Paragraph 5-23 and Appendix H describe Becker penetration test procedures. Appendix F, paragraph 3-3, discusses the Becker hammer drilling equipment and operation.

(3) The Standard Penetration Test (SPT) method of drive boring, described in ASTM D 1586-84 (ASTM 1996b), is probably the most commonly used method for advancing a hole by the drive method. Slight variations of this method, primarily concerning the sampling interval, cleanout method, and the refusal criteria exist from office to office but the fundamental procedure follows the ASTM standard. Appendix G presents procedures for SPT sampling and testing. Appendix G is compatible with the ASTM D 1586-84 standard and provides additional guidance in evaluating the test data. In this method, a standard configuration, 5-cm (2-in.) OD split barrel sampler at the end of a solid string of drill rods is advanced for a 0.45-m (1.5-ft) interval using a 623-Newton (N) (140-lb) hammer dropped through a 76-cm (30-in.) free fall. The blows required to advance the hole for each 15-cm (6-in.) interval are recorded on ENG Form 1836. The standard penetration resistance, or “N” value, is the sum of the blows required for the second and third 15-cm (6-in.) drives. The hole is then cleaned or reamed to the top of the next interval to be sampled and the procedure is repeated. Refusal is generally defined as 50 blows per 15 cm (per half foot) of penetration. When used to define the top of rock, great care and close examination of samples are required to minimize uncertainties. A few of the applications of SPT data are listed in paragraph 5-23a. This impact method may also be used with larger sample tubes and heavier hammers. Correlation studies to normalize data from larger holes to the SPT have been performed but are not completely reliable. The Becker hammer drill data can provide correlations of soil density and strength in coarse-grained soils similarly to the SPT test in finer-grained soils (paragraph 5-23a).

(4) Drive borings can be advanced quickly and economically with hollow-stem augers using a "plug" assembly that is either manually or mechanically set in the opening at the end of the auger string and then removed prior to sampling. Removal is commonly facilitated using a wire line system of retrieval. Where overburden prohibits the use of augers to advance the boring due to boulders or resistant rock lenses or ledges, other methods can be used. Traditionally, a roller rock bit using drilling mud will advance the hole at a modest cost in time and dollars. Where extremely difficult drilling conditions exist, an ODEX (eccentric reamer) down-the-hole air hammer system or other coring advancer apparatus can be used to penetrate the toughest boulders or ledges while still permitting the use of standard penetration or even undisturbed sampling to be conducted.

d. Cone penetration borings. The Cone Penetration Test (CPT) or Dutch cone boring is an in situ testing method for evaluating detailed soil stratigraphy as well as estimating geotechnical engineering properties (Schmertmann 1978a). The CPT involves hydraulically pushing a 3.6-cm (1.4-in.) diam special probe into the earth while performing two measurements, cone resistance and sleeve friction resistance. The probe is normally pushed from a special heavy duty truck but can also be performed from a trailer or drilling rig. Because of the weight of the truck or trailer needed to conduct CPT borings, access to soft ground sites is limited. Recent developments in CPT technology make it possible to retrieve physical soil samples and ground water or soil-gas samples with the same drive string used to perform the cone penetration test. CPT vehicles with push capacities up to 267 kiloNewtons (kN) (30 tons) have been developed. The Tri-Service Site Characterization and Analysis Penetrometer System (SCAPS), which is used to detect underground HTRW, is a technical variation of the CPT. The use of SCAPS reduces the time and cost of site characterization and restoration monitoring by providing rapid onsite real-time data acquisition/processing (i.e., in situ analysis) and onsite 3-D visualization of subsurface stratigraphy and regions of potential contamination. The Triservices operate several SCAPS vehicles including those of the U.S. Army Engineer District, (USAED) Kansas City, Savannah, and Tulsa, and the U.S. Army Engineer Waterways Experiment Station (USAEWES). Additional discussion of CPT is given in paragraph 5-23f.

e. Undisturbed borings. Appendix F, Chapters 5 and 6, discuss procedures for undisturbed sampling of soils. True "undisturbed" samples cannot be obtained because of the adverse effects resulting from sampling, shipping, or handling. However, modern samplers, used with great care, can obtain samples that are satisfactory for shear strength, consolidation, permeability, and density tests provided the possible effects of sample disturbance are considered. Undisturbed samples can be sliced to permit detailed study of subsoil stratification, joints, fissures, failure planes, and other details. Undisturbed samples of clays and silts can be obtained as well as nearly undisturbed samples of some sands.

(1) There are no standard or generally accepted methods for undisturbed sampling of noncohesive soils. One method that has been used is to obtain 7.6-cm (3-in.) Shelby (thin-wall) tube samples, drain them, and then freeze them prior to transporting them to the laboratory. Another method used consists of in situ freezing, followed by sampling with a rotary core barrel. Care is necessary in transporting any undisturbed sample, and special precautions must be taken if transporting sands and silts. For both methods, disturbance by cryogenic effects must be taken into account. Fixed-piston (Hvorslev) samplers, wherein a piston within a thin-walled tube is allowed to move up into the tube as the sampler is pushed into the soil, are adapted to sampling cohesionless and wet soils (Appendix F, paragraph 5-1*a*(2)).

(2) Undisturbed borings are normally made using one of two general methods: push samplers or rotary samplers. Push sampling types involve pushing a thin-walled tube using the hydraulic system of the drilling rig, then enlarging the diameter of the sampled interval by some "cleanout" method before beginning to sample again. Commonly used systems for push samples include the drill-rig drive,

whereby pressure is applied to a thin-walled (Shelby) sampling tube through the drill rods, the Hvorslev fixed-piston sampler, and the Osterberg hydraulic piston sampler. Rotary samplers involve a double tube arrangement similar to a rock coring operation except that the inner barrel shoe is adjustable but generally extends beyond the front of the rotating outer bit. This minimizes the disturbance to the sample from the drilling fluid and bit rotation. Commonly used rotational samplers include the Denison barrel and the Pitcher sampler. The Pitcher sampler has an inner barrel affixed to a spring-loaded inner sampler head that extends or retracts relative to the cutting bit with changes in soil stiffness. Drilling fluids are commonly used with rotary drilling equipment to transport cuttings to the surface and to increase the stability of the borehole. Chapter 4 of Appendix F discusses the types, preparation, and use of drilling fluids. The standard for thin-walled tube sampling of soils is ASTM D 1587-94 (ASTM 1996c), "Standard Practice for Thin-Walled Tube Sampling of Soils."

f. Rock core boring. Cored rock samples are retrieved by rotary drilling with hollow core barrels equipped with diamond- or carbide-embedded bits. The core is commonly retrieved in 1.5- to 3-m (5- to 10-ft) lengths. The "N" size hole (approximately 75 mm or 3 in.) is probably the core size most widely used by the Corps of Engineers for geotechnical investigations and produces a satisfactory sample for preliminary exploration work and, in many instances, for more advanced design studies. Other hole sizes, including B (approx 60 mm or 2.3 in.) and H (approximately 99 mm or 4 in.), are also quite satisfactory for geotechnical investigations. The decision on hole size should be based upon anticipated foundation conditions, laboratory testing requirements, and the engineering information desired. A double- or triple-tube core barrel is recommended because of its ability to recover soft or broken and fractured zones. The use of wireline drilling, whereby the core barrel is retrieved through the drill rod string, eliminates the need to remove the drill rods for sampling and saves a great deal of time in deep borings. Table 5-1 summarizes core and hole sizes commonly used in geotechnical studies. The rock boring is advanced without sampling using solid bits, including fishtail, or drag, bits, tri-cone and roller rock bits, or diamond plug bits.

(1) Most rock boring in the Corps of Engineers is accomplished using truck-mounted rotary drilling rigs. Skid-mounted rigs are also sometimes used in areas with poor access. Rotary drilling rigs are driven by the power takeoff from the truck engine or by independent engines. Boreholes are advanced by rotary action coupled with downward pressure applied to the drill bit and the cleaning action of the drilling fluid. Two types of pulldown mechanisms are normally used. Truck-mounted rotary drilling rigs equipped with a chain pulldown drive mechanism are capable of drilling to depths of 60 to 300 m (200 to 1,000 ft). Hydraulic feed drive rotary drilling rigs are capable of drilling to depths of 150 to 750 m (500 to 2,500 ft).

(2) Core recovery in zones of weak or intensely fractured rock is particularly important because these zones are typically the critical areas from the standpoint of foundation loading and stability. The use of larger-diameter core barrels in soft, weak, or fractured strata can improve core recovery and provides a statistically better size sample for laboratory testing. The advantages of larger cores must be weighed against their higher costs.

(3) Although the majority of rock core borings are drilled vertically, inclined, and horizontally oriented, borings may be required to adequately define stratification, jointing, and other discontinuities. A bias exists in the data favoring discontinuities lying nearly perpendicular to the boring. Discontinuities more nearly parallel to the boring are not intersected as often, and therefore, their frequency will appear to be much lower than it actually is. Inclined borings should be used to investigate steeply inclined jointing in abutments and valley sections for dams, along spillway and tunnel

Table 5-1
Typical Diamond Core Drill Bit and Reaming Shell Dimensions

	Bit Size		Reaming Shell
Size	**OD, mm (in.)**	**ID, mm (in.)**	**OD and hole diam, mm (in.)**
	"W" Group - "G" and "M" Design		
EWG (EWX), EWM	37.3 (1.470)	21.5 (0.845)	37.7 (1.485)
AWG (AWX), AWM	47.6 (1.875)	30.1 (1.185)	48.0 (1.890)
BWG (BWX), BWM	59.6 (2.345)	42.0 (1.655)	59.9 (2.360)
NWG (NWX), NWM	75.3 (2.965)	54.7 (2.155)	75.7 (2.980)
HWG	98.8 (3.890)	76.2 (3.000)	99.2 (3.907)
	"W" Group - "T" Design		
RWT	29.5 (1.160)	18.7 (0.735)	29.9 (1.175)
EWT	37.3 (1.470)	23.0 (0.905)	37.7 (1.485)
AWT	47.6 (1.875)	32.5 (1.281)	48.0 (1.890)
BWT	59.6 (2.345)	44.4 (1.750)	59.9 (2.360)
NWT	75.3 (2.965)	58.8 (2.313)	75.7 (2.980)
HWT	98.8 (3.890)	81.0 (3.187)	99.2 (3.907)
	Large-Diameter Design		
2-3/4 X 3-7/8	97.5 (3.840)	68.3 (2.690)	98.4 (3.875)
4 X 5-1/2	138.1 (5.435)	100.8 (3.970)	139.6 (5.495)
6 X 7-3/4	194.4 (7.655)	151.6 (5.970)	196.8 (7.750)
	Wireline Sizes		
AQ		*27.0 ($1^{1}/_{16}$)*	*48.0 ($1^{57}/_{64}$)*
BQ		*36.5 ($1^{7}/_{16}$)*	*60.0 ($2^{23}/_{64}$)*
NQ		*47.6 ($1^{7}/_{8}$)*	*75.8 ($2^{63}/_{64}$)*
HQ		*63.5 ($2^{1}/_{2}$)*	*96.0 ($3^{25}/_{32}$)*
PQ		*85.0 ($3^{11}/_{32}$)*	*122.6 ($4^{53}/_{64}$)*

alinement, and in foundations for other structures. In nearly vertical bedding, inclined borings can be used to reduce the total number of borings needed to obtain core samples of all strata.

(4) If precise geological structure is to be evaluated from core samples, techniques involving oriented cores are required. In these procedures, the core is scribed or engraved with a special drilling tool (Goodman 1976) so that its orientation is preserved. In this manner, both the dip and strike of any joint, bedding plane, or other planar surface can be ascertained. A more common procedure for obtaining dip and strike of structural features is the use of borehole photography or television. If the orientation of bedding is consistent across the site, it can be used to orient cores from borings which are angled to this bedding. Once oriented, the attitudes of discontinuities can be measured directly from the core.

(5) Large-diameter borings or calyx holes, 0.6 m (2 ft) or more in diameter, are occasionally used in large or critical structures. Their use permits direct examination of the sidewalls of the boring or shaft and provides access for obtaining high quality undisturbed samples. Direct inspection of the sidewalls may reveal details, such as thin, weak layers or shear planes that may not be detected by continuous undisturbed sampling. Large-diameter borings are produced with augers in soil and soft rock, and with large-diameter core barrels in hard rock.

5-6. Drilling in Embankments

The Corps of Engineers developed a special regulation concerning drilling operations in dam and levee embankments and their soil foundations (ER 1110-1-1807). In the past, compressed air and other drilling fluids have been used as circulating media to remove drill cuttings, stabilize bore holes, and cool and lubricate drilling bits. There have been several incidents of damage to embankments and foundations when drilling with air, foam, or water as the circulating medium. Damage has included pneumatic fracturing of the embankment while using air or air with foam, and erosion of embankment or foundation materials and hydraulic fracturing while using water. The new ER establishes a policy for drilling in earth embankments and foundations and replaces ER 1110-1-1807. The following points summarize the guidance provided in the new document:

a. Personnel involved in drilling in dam and levee embankments shall be senior and well qualified. Designs shall be prepared and approved by geotechnical engineers or engineering geologists. Drillers and "mud" specialists shall be experts in their fields.

b. Drilling in embankments or their foundations using compressed air or other gas or water as the circulating medium is prohibited.

c. Cable tool, auger, and rotary tool are recommended methods for drilling in embankments. One Corps District reports using a churn drill (a cable tool rig) to sample the clay core of a dam to a depth of 90 m (300 ft) with no damage to the core. If the cable tool method is used, drilling tools must be restricted to hollow sampling (drive) barrels in earth embankment and overburden materials. Appendix F, page 3-6, of this manual discusses the use of churn drills. If rotary drilling is used, an engineered drilling fluid (mud) designed to prevent caving and minimize intrusion of the drilling fluid into the embankment shall be used. An appendix in ER 1110-1-1807 provides detailed procedures for rotary drilling.

Section II
Drillhole Inspection and Logging

5-7. Objectives

A major part of field investigations is the compilation of accurate borehole logs on which subsequent geologic and geotechnical information and decisions are based. A field drilling log for each borehole can provide an accurate and comprehensive record of the lithology and stratigraphy of soils and rocks encountered in the borehole and other relevant information obtained during drilling, sampling, and in situ testing. To accomplish this objective, an experienced geologist, soils engineer, or civil engineer with good geotechnical training and experience should be present during drilling. The duties of the field inspector include the following:

a. Making decisions on boring location, depth, and number and quality of samples required.

b. Observing and describing drilling tools and procedures.

c. Observing, classifying, and describing geologic materials and their discontinuities.

d. Selecting and preserving samples.

e. Performing field tests on soils (hand penetrometer, torvane).

f. Photographing site conditions and rock cores.

g. Observing and recording drilling activities and ground water measurements.

h. Overseeing and recording instrument installation activities.

i. Completing the drilling log, ENG FORM 1836 and/or entering information in BLDM (Nash 1993).

j. Recording information and data from in situ tests.

The logs of borings are normally made available to contractors for use in preparing their bids. The descriptions contained on the logs of borings give the contractor an indication of the type of materials to be encountered and their in situ condition. Special care must be taken to ensure a clear differentiation in logs between field observations and laboratory test results. Guidance on soil identification and description, coring, and core logging is provided in the remainder of this section.

5-8. Soil Identification and Description

A thorough and accurate description of soils is important in establishing general engineering properties for design and anticipated behavior during construction. The description must identify the type of soil (clay, sand, etc.), place it within established groupings, and include a general description of the condition of the material (soft, firm, loose, dense, dry, moist, etc.). Characterization of the soils within a site provides guidance for further subsurface exploration, selection of samples for detailed testing, and development of generalized subsurface profiles (Das 1994). Initial field soil classification with subsequent lab tests and other boring data are recorded on the logs of borings. Soils should be described in accordance with ASTM D 2488-93 (ASTM 1996d). For civil works, the most widely used classification is the Unified Soil Classification System (USCS). The USCS outlines field procedures for determining plasticity, dilatancy, dry strength, particle size, and other engineering parameters. The USCS is described by Schroeder (1984) and in Technical Memorandum 3-357 (USAEWES 1982). A number of references provide detailed procedures to evaluate the physical properties of soils, including Cernica (1993), Lambe and Whitman (1969), Terzaghi, Peck, and Mesri (1996), and Means and Parcher (1963). In some cases, a standardized description of color using Munsell charts is useful. Some of the procedures, such as determining dry strength, may be impractical under certain field conditions and may be omitted where necessary. However, the checklists included in the procedure, if followed conscientiously, provide for a thorough description of soils. Examples for presenting soils data on ENG FORM 1836 are shown in Appendix D. Examples of well logs in the Boring Log Data Manager format are also presented in Appendix D.

5-9. Coring

Core drilling, if carefully executed and properly reported, can produce invaluable subsurface information. Basic procedures that should be followed and the information obtained can form the basis for comparison for widely diverse sites and conditions. The following subparagraphs outline procedures to report observations made during coring operations.

a. Drilling observations. During the coring operation, a great deal of information is available about the subsurface conditions that may or may not be apparent in the core recovered from the hole. Observation of the drilling action must be made and reported to present as complete a picture as possible of the subsurface conditions.

(1) If coring with water as a circulating medium, the inspector should note the amount of water return relative to the amount being injected through the drill rods and its color. Careful observation of drill water return changes can indicate potential intervals where pressure test takes can be anticipated and correlated. Changes in the color of the return water can indicate stratigraphic changes and degrees of weathering such as clay-filled joints and cavity fillings.

(2) If available, hydraulic pressure being exerted by the drill should be recorded on each run as well as the fluid water pressure. While the drill is turning, the inspector should correlate drilling depths to drilling action (e.g., smooth or rough), increases and decreases applied by the drill operator to the feed control valve, and the rate of penetration. Rod drop depths, which indicate open zones, should be recorded. Changes in drilling rates can be related to changes in composition and/or rock structure and, in areas of poor core recovery, may provide the only indication of the subsurface conditions.

b. Procedural information. Regardless of the program undertaken, all logs should at least include the following: size and type of core bit and barrel used; bit changes; size, type, and depth of casing; casing shoe and/or casing bit used; problems or observations made during placement of the casing; change in depth of casing setting during drilling; depth, length, and time for each run; length/depth of pull (the actual interval of core recovered in the core run); amount of core actually recovered; amount of core loss or gain; and amount of core left in the hole (tape check). The inspector should note the presence of a flange on the bottom of a core string because a flange indicates that the core was retrieved from the bottom of the drilled hole. From these data the unaccountable loss, i.e., the core that is missing and unaccounted for, should be computed. Core loss should be shown on the graphic log and by blocks or spacers in the core box at its most likely depth of occurrence based upon the drilling action and close examination of the core. The boring should be cleaned and the total depth taped to determine the amount of cored rock left in the hole on the final run.

5-10. Core Logging

Each feature logged shall be described in such a way that other persons looking at the core log will recognize what the feature is, the depth at which it occurred in the boring, and its thickness or size. They should also be able to obtain some idea of the appearance of the core and an indication of its physical characteristics. The log shall contain all the information obtainable from the core pertaining to the rock as well as discontinuities. Examples for presenting core logging data on ENG FORM 1836 are shown in Appendix D.

a. Rock description. Each lithologic unit in the core shall be logged. The classification and description of each unit shall be as complete as possible. A recommended order of descriptions is as follows:

(1) Unit designation (Miami oolite, Clayton formation, Chattanooga shale).

(2) Rock type and lithology.

(3) Hardness, relative strength, or induration..

(4) Degree of weathering.

(5) Texture.

(6) Structure.

(7) Discontinuities (faults, fractures, joints, seams).

(a) Orientation with respect to core axis.

(b) Asperity (surface roughness).

(c) Nature of infilling or coating, if present.

(d) Staining, if present.

(e) Tightness.

(8) Color.

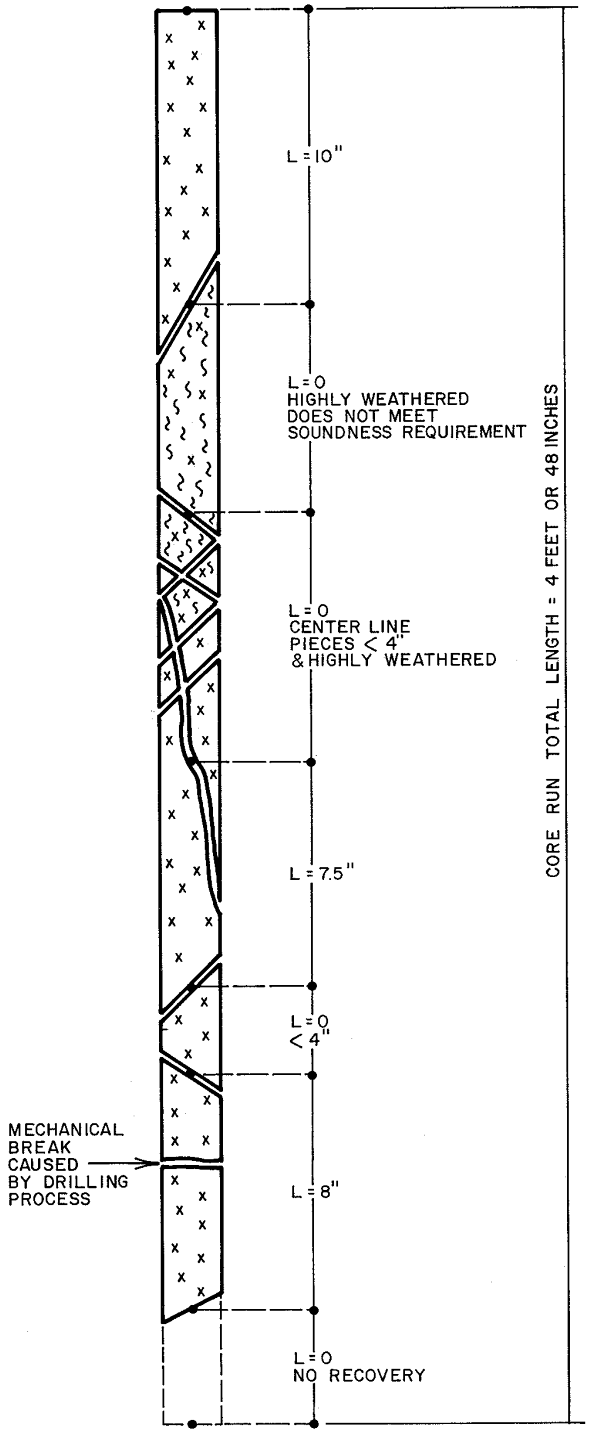
(9) Solution and void conditions.

(10) Swelling and slaking properties, if apparent.

(11) Additional descriptions such as mineralization, inclusions, and fossils.

Criteria for these descriptive elements are contained in Table B-2 (Appendix B). Murphy (1985) provides guidelines for geotechnical descriptions of rock and rock masses. Geological Society Engineering Group Working Party Report (1995) suggests a description and classification scheme of weathered rocks for engineering purposes. Variation from the general description of the unit and features not included in the general description should be indicated at the depth and the interval in the core where the feature exists. These variations and features shall be identified by terms that will adequately describe the feature or variation so as to delineate it from the general description. Features include zones or seams of different color and texture; staining; shale seams, gypsum seams, chert nodules, and calcite masses; mineralized zones; vuggy zones; joints; fractures; open and/or stained bedding planes, roughness, planarity; faults, shear zones, and gouge; cavities, thickness, open or filled, and nature of filling; and core left in the bottom of the hole after the final pull.

b. Rock quality designation. A simple and widely used measure of the quality of the rock mass is provided by the Rock Quality Designation (RQD), which incorporates only sound, intact pieces 10 cm (4 in.) or longer in determining core recovery. In practice, the RQD is measured for each core run and reported on ENG Form 1836. Many of the rock mass classification systems in use today are based, in part, on the RQD. Its wide use and ease of measurement make it an important piece of information to be gathered on all core holes. It is also desirable because it is a quantitative measure of core quality at the time of drilling before handling and slaking have had major effect. Deere and Deere (1989) reevaluated the use of RQD from experience gained in the 20 years since its inception. They recommended modifications to the original procedure after evaluating results of field use. Figure 5-1 illustrates the modified procedure of Deere and Deere.



$$\text{RQD} = \frac{\sum \text{LENGTH OF SOUND CORE PIECES} > 4 \text{ INCHES (100mm.)}}{\text{TOTAL CORE RUN LENGTH}}$$

$$\text{RQD} = \frac{10 + 7.5 + 8}{48} \times 100\%$$

RQD = 53 % (FAIR)

RQD (ROCK QUALITY DESIGNATION)	DESCRIPTION OF ROCK QUALITY
0 - 25 %	VERY POOR
25-50 %	POOR
50-75 %	FAIR
75-90 %	GOOD
90-100%	EXCELLENT

Figure 5-1. Illustration of Deere and Deere (1989) modified procedure for calculating RQD

(1) RQD was originally recommended for NX size (5.474-cm- or 2.155-in.-diam) core, but Deere and Deere expanded its use to the somewhat smaller NQ wireline sizes (4.763 cm or 1-7/8 in.) and to larger wireline sizes up to 8.493 cm (3-11/32 in.) and other core sizes up to 15 cm (6 in.). They discouraged RQD use with the smaller BQ (3.651-cm or 1-7/16-in.) and BX (4.204-cm or 1.655-in.) cores because of core breakage.

(2) Core segment lengths should be measured along the centerline or axis of the core, as illustrated in Figure 5-1.

(3) The inspector should disregard mechanical breaks (breaks caused by drilling action or handling) when calculating RQD.

(4) RQD should be performed at the time the core is retrieved to avoid the effects of postremoval slaking and separation of core along bedding planes, as in some shales.

(5) Emphasis should be placed on core being "sound." Pieces of core that do not meet the subjective "soundness" test should not be counted. Indicators of "unsound" rock are discolored or bleached grains or crystals, heavy staining, pitting, or weak grain boundaries. Unsound rock is analogous to "highly weathered" rock, which is characterized by weathering extending throughout the rock mass.

Several papers have appeared since Deere and Deere (1989) suggesting alternatives or modified applications of RQD to systems of discontinuities that are perhaps less amenable to analysis by the original procedure. Boadu and Long (1994) established a relation between RQD and fractal dimension (the degree to which a system is self-similar at different scales). The relationships may have application in fracture geometries with complex distributions. Eissa and Sen (1991) suggest alternative analytical methods to RQD when dealing with fracture networks, that is, sets of fractures in more than one direction. Similar alternative approaches to systems of fractures in three dimensions (a volumetric approach) were proposed by Sen and Eissa (1991). Special attention should be paid to the nature of all discontinuities. These are most often what control the engineering behavior of the foundation rock mass and slope stability.

c. Solution and void conditions. Solution and void conditions shall be described in detail because these features can affect the strength of the rock and can indicate potential ground water seepage paths. Where cavities are detected by drilling action, the depth to top and bottom of the cavity should be determined by measuring. Filling material, where present and recovered, should be described in detail opposite the cavity location on the log. If no material is recovered from the cavity, the inspector should note the probable conditions of the cavity, as determined by observing the drilling action and the color of the drilling fluid. If drilling action indicates material is present, e.g., a slow rod drop, no loss of drill water, or noticeable change in color of water return, it should be noted on the log that the cavity was probably filled and the materials should be described as well as possible from the cuttings or traces left on the core. If drilling action indicates the cavity was open, i.e., no resistance to the drilling tools and/or loss of drilling fluid, it should be noted on the drilling log. By the same criteria, partially filled cavities should be noted. If possible, filling material should be sampled and preserved. During the field logging of the core at the drilling site, spacers should be placed in the proper position in core boxes to record voids and losses.

d. Photographic and video record. A color photographic record of all core samples should be made. Photographs should be taken as soon as possible after retrieving the core samples. The core photographs can be reproduced on 20- by 25-cm (8- by 10-in.) prints, two or three core boxes to a photograph, and the photographic sheets placed in a loose-leaf binder for convenient reference. Photographs often enhance the logged description of cores particularly where rock defects are abundant.

In the event that cores are lost or destroyed, the photographic record becomes the only direct, visual means for review of subsurface conditions without expensive redrilling. A video recording of the drilling operation provides an excellent record of drilling equipment and procedures. Moreover, video may provide a record of critical events or conditions that were not obvious at the time, or occurred too quickly to be recorded manually.

5-11. Drilling Log Form and the Boring Log Data Management Program

All soil and rock drilling logs will be recorded using ENG FORM 1836 as the standard, official log of record. As a general rule, the depth scale on each sheet should normally be 3 m (10 ft) per page and no smaller than 6 m (20 ft). Examples of completed drilling logs are shown in Appendix D. A PC-based, menu-driven boring log data management program (BLDM) is available for free to COE personnel through CEWES-GS-S. The BLDM allows users to create and maintain boring log data, print reports, and create data files which can be exported to a GIS (Nash 1993). Examples of BLDM output are presented in Appendix D.

Section III
Borehole Examination and Testing

5-12. Borehole Geophysical Testing

A wide array of downhole geophysical probes is available to measure various formation properties (Tables 4-1 and 4-2). Geophysical probes are not a substitute for core sampling and analysis, however, but they are an economical and valuable supplement to the core sample record. Some very sophisticated analyses of rock mass engineering properties are possible through the use of downhole geophysics. These services are available through commercial logging companies and various Government agencies. Recent developments in microcomputer technology have made it possible to apply procedures known as crosshole tomography to borehole seismic and resistivity data (Cottin et al. 1986; Larkin et al. 1990). Through computer analysis of crosshole seismic and resistivity data, tomography produces a 3-D rendition of the subsurface. The level of detail possible depends upon the distance between holes, the power of the source, and the properties of the rock or soil mass. The method can be used for both indurated and nonindurated geomaterials.

5-13. Borehole Viewing and Photography

The interpretation of subsurface conditions solely by observation, study, and testing of rock samples recovered from core borings often imposes an unnecessary limitation in obtaining the best possible picture of the site subsurface geology. The sidewalls of the borehole from which the core has been extracted offer a unique picture of the subsurface where all structural features of the rock formation are still in their original position. This view of the rock can be important, particularly if portions of rock core have been lost during the drilling operation and if the true dip and strike of the structural features are required. Borehole viewing and photography equipment includes borescopes, photographic cameras, TV cameras, sonic imagery loggers, caliper loggers, and alinement survey devices. Sonic imagery and caliper loggers are discussed in detail in EM 1110-1-1802. Alinement survey services are available from commercial logging or drilling firms and from the U.S. Army Engineer Waterways Experiment Station (CEWES-GG-F). Borehole viewing systems and services are often obtained now from private industry or from the few COE offices that have the capabilities.

5-14. Borehole Camera and Borescope

Borehole film cameras that have limited focus capability are satisfactory for examining rock features on the sidewalls of the borehole. However, the small viewing area and limited focus reduce the usefulness in borings that have caved or that have cavities. They are best used for examining soft zones for which core may not have been recovered in drilling, for determination of the dip and strike of important structural features of the rock formation, and to evaluate the intrusion of grout into the rock mass. The camera's film must be processed before the images can be examined. The borescope, basically a tubular periscope, has limited use because of its small viewing area, limited depth, and cumbersome operation. It is relatively inexpensive to use, however.

5-15. Borehole TV Camera and Sonic Imagery

The TV camera has variable focus and is suitable for examining the nature and approximate dimensions of caving sections of open boreholes or boreholes filled with clear water. The TV camera provides both real-time imagery and a permanent record of the viewing session. The sonic imagery (televiewer) system uses acoustic pulses to produce a borehole wall image and can be used in a hole filled with drilling mud. The TV camera is used to examine cavities in the rock such as solution voids in calcareous formations, open cooling joints, and lava tunnels in volcanic rocks, mines, tunnels, and shafts. Most TV systems are capable of both axial (downhole) and radial (sidewall) viewing. The televiewer can be used to distinguish fractures, soft seams, cavities, and other discontinuities. Changes in lithology and porosity may also be distinguished. Specially designed borehole television cameras and sonic imagers or televiewers can be used to determine the strike and dip of discontinuities in the borehole wall. The Corps of Engineers has this capability at the U.S. Army Engineer District, Walla Walla, WES, and the U.S. Army Engineer Division Laboratory, Southwestern.

5-16. Alinement Surveys

Alinement surveys are often necessary if the plumbness and/or orientation of a hole is important. Older methods employed a compass and photograph system which was relatively easy to use. More modern systems are electronic. Alinement surveys may be critical in deep holes where instrumentation packages are to be installed or where precise determinations of structural features in the rock formation are required.

Section IV
Exploratory Excavations

5-17. Test Pits and Trenches

Test pits and trenches can be constructed quickly and economically by bulldozers, backhoes, pans, draglines, or ditching machines. Depths generally are less than 6 to 9 m (20 to 30 ft), and sides may require shoring if personnel must work in the excavations. Test pits, however, hand dug with pneumatic jackhammers and shored with steel cribbing, can be dug to depths exceeding 18 m (60 ft). Test pits and trenches generally are used only above the ground water level. Test pits that extend below the water table can be kept open with air or electric powered dewatering pumps. Exploratory trench excavations are often used in fault evaluation studies. An extension of a rock fault into much younger overburden materials exposed by trenching is usually considered proof of recent fault activity. Shallow test pits are commonly used for evaluating potential borrow areas, determining the geomorphic history, and assessing cultural resource potential.

5-18. Calyx Hole Method

Large-diameter calyx holes have been used successfully on some jobs to provide access for direct observation of critical features in the foundations. These holes are very expensive to drill (possibly $2,300 per meter or $700 per foot), so their use is very limited. However, where in situ observation of a very sensitive feature, such as a shear zone or solution feature in the abutment of an arch dam, cannot be achieved reasonably by any other means, the calyx hole may be the procedure of choice.

Section V
Ground Water and Foundation Seepage Studies

5-19. General Investigation

The scope of ground water studies is determined by the size and nature of the proposed project. Efforts can range from broad regional studies at a reservoir project to site-specific studies, such as pumping tests for relief well design, water supply at a recreational area, or pressure tests performed to evaluate the need for foundation grouting. Ground water studies include observations and measurements of flows from springs and of water levels in existing production wells, boreholes, selected observation wells, and piezometers. This information is used with site and regional geologic information to determine water table or piezometric surface elevations and profiles, fluctuations in water table elevations, the possible existence and location of perched water tables, depths to water-bearing horizons, direction and rate of seepage flow, and potential for leakage from a proposed reservoir or beneath an embankment or levee. Complex investigations are made only after a thorough analysis has been made of existing or easily acquired data. Results from ground water and foundation seepage studies provide data needed to design dewatering and seepage control systems at construction projects, indicate the potential for pollution and contamination of existing ground water resources due to project operation, show potential for interference to aquifers by the construction of a project, and determine the chemical and biological quality of ground water and that relationship to project requirements. Investigation and continued monitoring of ground water fluctuations are key dam safety issues.

a. Wells. Existing wells located during field geologic reconnaissance should be sounded or water levels obtained from the well owners. Pumping quantities, seasonal variations in ground water and pumping levels, depths of wells and screen elevations, corrosion problems, and any other relevant information should be acquired wherever available. Any settlement records attributable to ground water lowering from pumping should be obtained. This information should be compared with water well records obtained during preliminary studies to develop a complete hydrologic picture for the project area.

b. Borings. Water levels recorded on drilling logs are another source of information. However, they may not reflect true water levels, depending on soil types and time of reading after initial drilling. The influence of drilling fluids on water level readings should be kept in mind when evaluating boring data. Loss of drilling fluids can indicate zones of high permeability. Where ground water level information is needed, installation of piezometers or observation wells in borings should be considered.

c. Piezometers and observation wells. The most reliable means for determining ground water levels is to install piezometers or observation wells. Piezometers measure excess hydrostatic pressures beneath dams and embankments. All information developed during preliminary studies on the regional ground water regime should be considered in selecting locations for piezometers and observation wells. For types of piezometers, construction details, and sounding devices, refer to EM 1110-2-1908, Part 1, and TM 5-818-5/AFM 88-5, Chapter 6/NAVFAC P-418. All piezometer borings should be logged carefully and "as built" sketches prepared that show all construction and backfill details (Figure 5-2).

(1) The selection of the screened interval is critical to the information produced, since the water level recorded will be the highest of all intervals within the screen/filter length. Careful evaluation of the conditions encountered in the hole with regard to perched or confined aquifers is essential to a sensible selection of the screened interval and interpretation of the data. One of the greatest benefits of a piezometer or observation well is that it allows for measurement of fluctuations in piezometric levels over time. To take advantage of this benefit, it is necessary to provide for periodic readings. This can be accomplished through manual reading by an automated system, depending on the location and critical importance of the area being monitored.

(2) Other information that can be derived from observation wells and piezometers are temperature and water quality data. Tracer tests can sometimes be conducted to determine the direction and rate of ground water flow.

d. Springs and surface water. The water elevation, flow rate, and temperature of all springs located within the project area should be measured. Water should be sampled for chemical analysis to establish a baseline level. Soil or rock strata at the spring should be evaluated to locate permeable horizons. Flow rates at springs should be measured during dry and wet seasons to determine the influence of rainfall on seepage conditions. The elevation of water levels in lakes and ponds should be measured during the wet and dry seasons to evaluate the extent of surface water fluctuations.

e. Geophysical methods. Geophysical methods, such as seismic refraction, can be used to determine the depth to saturated material. Depending on the accuracy required and the accuracy of the method, a minimal number of piezometers should be installed to verify the geophysical data. Surface resistivity surveys can indicate the presence of and depth to water (Society of Exploration Geophysicist 1990). Ground penetrating radar can also be used to detect the presence and location of ground water (Annan 1992). Fetter (1988) discusses these and other geophysical methods to characterize the hydrology and hydrogeology of a site.

f. Tracer testing. In some areas, especially karst terrains, it is of particular interest to determine flow paths in the ground water system. Although complex, flow paths in karst, where seepage velocities are high, can be evaluated by conducting tracer tests using either environmentally benign dyes or biological tracers such as pollen. The tracer element is introduced into a boring or other access points and monitored at an exit point such as a spring. The travel time from the introduction to detection is recorded. Numerous tests at different locations can be run and a picture of the ground water flow regime developed.

5-20. Permeability Testing

Permeabilities of foundation materials can be determined from slug and pumping tests in piezometers and wells, laboratory tests of undisturbed samples, and pressure tests in rock foundations. The permeability of sands can be roughly estimated from the D_{10} fraction (TM 5-818-5). Fracture and joint analysis is important in evaluating permeability of rock foundations. General reviews of methods to evaluate permeability of soil and rock in the subsurface include Bentall (1963), Davis and DeWiest (1966), Dawson and Istok (1991), Driscoll (1986), Fetter (1988), Heath (1983), Lohman (1972), and Walton (1970).

DRILLING LOG	DIVISION: ENVIRONMENTAL ENGINEERING	CLIENT: U.S. ARMY - APG D.S.H.E-E.M.D	SHEET 1 OF 2 SHEETS
1. PROJECT: FTA WESTERN BOUNDARY INVESTIGATION		8. SCREENED INTERVAL: 45.0-50.0 FT	SCREEN TYPE GALVINIZED STEEL 0.010 IN CONTINUOUS SLOT
2. LOCATION: WB-P1	MSPCS: 656,502 NORTHING 1,541,174 EASTING	9. SAMPLING METHOD (SOIL) 2.0 FT SPLIT SPOON	(AIR) PHOTOVAC MICROTIP HL2000
3. COUNTY: HARFORD	STATE: MARYLAND	10. DRILLING EQUIPMENT: GUS PECHE BRAT 22R	
4. HOLE NO. *(AS SHOWN IN STATE RECORDS)* HA-92-0494		11. ELEVATION GROUND WATER *(MEASURED FROM GROUND LEVEL)* 10.7 FT MSL	
5. NAME OF DRILLER: JAMES MARSH		12. DATE HOLE: STARTED: 14 OCT 92	COMPLETED: 15 OCT 92
6. DRILLING AGENCY: LAYNE ENVIRONMENTAL SERVICES		13. ELEVATION OF TOP OF HOLE: 40.70 FT	TOP OF RISER 43.41 FT
7. DEPTH OF HOLE 77.0 FT		NAME OF INSPECTOR MARK A. LEWIS	

DEPTH FEET	CLASSIFICATION OF MATERIALS *(DESCRIPTION)*	USCS	BLOW COUNTS	%RECOVERY SAMPLE INTERVAL	SAT.	MOIST	DAMP	LITHOLOGY	WELL COMPLETION DIAGRAM	REMARKS *(Drilling time, water loss, depth of weathering, ect., if significant)*	DEPTH FEET
	Light brown sandy silt									Above Ground Completion	
5	Brown silty fine to med. sand with trace fine gravels, subangular	SM									5
	White to tan fine sand, poorly sorted	SP								20:1 Portland Cement Bentonite Grout	
	Tan silty fine sand, subangular	SM									
	Tan fine to coarse sands, poorly sorted	SP							1.5 INCH CARBON STEEL RISER		
10	Brown sandy clay, slightly plastic	SC									10
	Brown medium to coarse poorly sorted sand with some fine gravels, subrounded	SP									
15											15
20	Light brown silty clay, slight to medium plasticity	CL									20
25	Yellowish orange silty fine sand, subangular	SM									25
	Brown sandy clay, medium plasticity	SC									
30										▽	30
										No Organic Vapor Readings Above Background in Breathing Zone	
35	Light brown fine sand, subrounded									Bentonite Slurry Seal	35
40											40
45		SP								Filter Pack #2 Sand	45

Figure 5-2. Example of a report-quality log with lithologic, blow count, moisture, and well completion information. Note that the header contains a variety of details concerning this monitoring well (Continued)

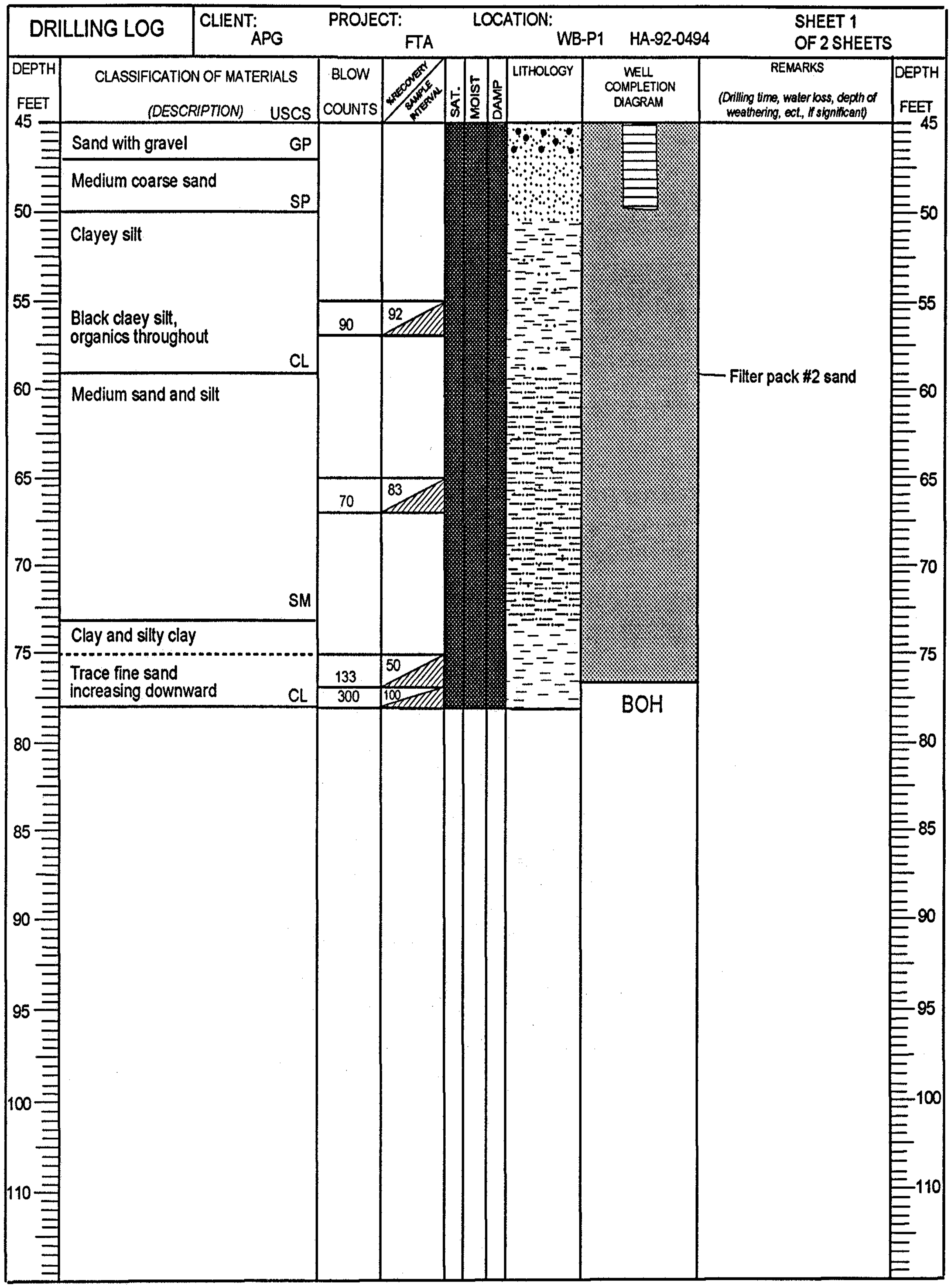


Figure 5-2. (Concluded)

a. Tests in piezometers or wells. Permeability tests can easily be made in piezometers or wells. They should be performed as part of piezometer installation procedures, both to obtain permeability information and to assure that the piezometer is working satisfactorily. Appropriate piezometer permeability tests are constant head, falling or rising head, and slug. The information obtained is representative of a smaller volume of material than that tested in pumping tests. However, procedures are simple, costs are low, and results may be useful if interpreted with discretion. Test details are discussed in EM 1110-2-1908 (Part 1), TM 5-818-5, U.S. Department of Interior (1977), Mitchell, Guzikowski, and Villet (1978), and Bennett and Anderson (1982).

b. Pumping tests. Pumping tests are the traditional method for determining permeability of sand, gravels, or rock aquifers. Observation wells should be installed to measure the initial and lowered ground water levels at various distances from the pumped well. At known or suspected HTRW sites, disposal of pumped water is a major consideration. For details of pumping tests and analyses, refer to TM 5-818-5. Pumping tests are usually desirable for the following:

(1) Large or complex projects requiring dewatering.

(2) Design of underseepage systems for dams or levees.

(3) Special aquifer studies.

(4) Projects where water supply will be obtained from wells.

(5) Projects immediately downstream from existing embankments.

c. Permeability of rock. Most rock masses contain, in addition to intergranular pore spaces, complex interconnecting systems of joints, fractures, bedding planes, and fault zones that, collectively, are capable of transmitting ground water. Fracture or joint permeability is normally several magnitudes higher than the matrix permeability of the discrete blocks or masses of rock contained between the joints. The permeability of some rock masses, such as sandstones and conglomerates, is governed by interstitial voids similar to that of soils. Secondary weathering and solutioning of limestones and dolostones may produce large void spaces and exceptionally high permeabilities. Although the permeability of rock results from interconnecting systems of joints, fractures, and formational voids, the equivalent rock mass permeability can frequently be modeled as a uniform porous system. Although it is necessary to keep the hydrologic model manageable, the shortcoming of this approach is that most rock masses are anisotropic with regard to permeability. The influence of this on a practical level is that it is easy to over- or underestimate the ground water effects in rock. As an example, if a pumping test is conducted with monitoring wells oriented along a line perpendicular to the predominant water-bearing joint set, the results will underestimate the radius of influence along the joint set. Therefore, the layout of pumping tests must be well thought out beforehand. At least a preliminary fracture and joint analysis should be conducted prior to laying out a pump test.

d. Fracture and joint analysis. Because joint or fracture permeability frequently accounts for most of the flow of water through rocks, an accurate description of in situ fracture conditions of a rock mass is critical to predicting performance of drains, wells, and piezometer responses. Joints typically occur in sets which have similar orientations. There may be three or more sets of joints in a rock mass. Joint sets that occur in the rock mass at the site should be identified and the preferred orientation and range in orientation of each joint set recorded. Features such as joint orientation, spacing, joint width, and the degree and type of secondary mineral filling should be carefully noted for each joint set. Once all joint sets of a site have been identified and evaluated, their relative importance to ground water flow should be assessed. Joints and fractures can be evaluated by developing the structure and stratigraphy of the site from accessible outcrops and from borehole logs.

5-21. Pressure Tests

a. Pressure tests are performed to measure the permeability of zones within rock masses. Pressure test results are used in assessing leakage in the foundation and as a guide in estimating grouting requirements. Pressure tests are typically conducted during exploratory core drilling and are a relatively inexpensive method of obtaining important hydrogeologic information about a rock mass. Hydraulic pressure testing should be considered an integral part of the exploratory core drilling process in all cases where rock seepage characteristics could affect project safety, feasibility, or economy. The testing interval is typically 1.5 to 3 m (5 to 10 ft) but may be varied to fit specific geological conditions observed during the core drilling operations. Zones to be tested should be determined by (1) examining freshly extracted cores, (2) noting depths where drilling water was lost or gained, (3) noting drill rod drop, (4) performing borehole or TV camera surveys, and (5) conducting downhole geophysical surveys. In rock with vertical or high angle joints, inclined borings are necessary to obtain meaningful results. Types of tests and test procedures are described in Ziegler (1976), U.S. Department of Interior (1977), and Bertram (1979).

b. Pressures applied to the test section during tests should normally be limited to 23 kilo Pascals (kP) per meter (1 psi per foot) of depth above and 13 kP/meter (0.57 psi/foot) of depth below the piezometric surface. The limit was established to avoid jacking and damage to rock formations. The limit is conservative for massive igneous and metamorphic rocks. However, it should be closely adhered to for tests in horizontally bedded sedimentary and other similar types of formations. Naturally occurring excess water pressures (artesian) should be taken into account in computations for limiting test pressures. Where the test intervals are large, a reduction in total pressure may be necessary to prevent jacking of the formation within the upper portion of the test section.

c. An important, but often unrecognized, phenomenon in pressure testing is joint dilation and contraction as pressure is applied and released. In the case of a dam project, it is desirable to use pressures that will correspond to future reservoir conditions. Joint dilation can frequently be observed by conducting a "holding" test. The fall in pressure is observed and a plot of pressure versus time is made. The pressure should quickly drop to near the surrounding piezometric level if the joint openings remain the same width. The common observation of a slow pressure decay in pressure holding tests indicates joint closure with reduction in pressure.

d. Qualitative evaluations of leakage and grout requirements can be made from raw pressure test data (Ziegler 1976, U.S. Department of Interior 1977, Bertram 1979). Most analyses of this type assume laminar flow rather than turbulent flow. This assumption can be verified by conducting pressure tests on the same interval at several different pressures. If the water take is directly proportional to the total applied pressure, laminar flow can be assumed. If pressure test data are converted into values of equivalent permeability or transmissivity, calculations can be performed to estimate seepage quantities. Wherever possible, such results should be compared with data from completed projects where similar geologic conditions exist.

Section VI
In situ Testing to Determine Geotechnical Properties

5-22. In Situ Testing

In situ tests are often the best means for determining the engineering properties of subsurface materials and, in some cases, may be the only way to obtain meaningful results. Table 5-2 lists in situ tests and their purposes. In situ rock tests are performed to determine in situ stresses and deformation properties (moduli) of the jointed rock mass, shear strength of jointed rock masses or critically weak seams within the rock mass, and residual stresses along discontinuities or weak seams in the rock mass. Pressure tests have been discussed in Section V (paragraph 5-20) of this manual.

Table 5-2
In Situ Tests for Rock and Soil

		Applicability to	
Purpose of Test	**Type of Test**	**Soil**	**Rock**
Shear strength	Standard penetration test (SPT)	X	
	Field vane shear	X	
	Cone penetrometer test (CPT)	X	
	Direct shear	X	
	Plate bearing or jacking	X	X[1]
	Borehole direct shear[2]	X	
	Pressuremeter[2]		X
	Uniaxial compressive[2]		X
	Borehole jacking[2]		X
Bearing capacity	Plate bearing	X	X[1]
	Standard penetration	X	
Stress conditions	Hydraulic fracturing	X	X
	Pressuremeter	X	X[1]
	Overcoring		X
	Flatjack		X
	Uniaxial (tunnel) jacking	X	X
	Borehole jacking[2]		X
	Chamber (gallery) pressure[2]		X
Mass deformability	Geophysical (refraction)	X	X
	Pressuremeter or dilatometer	X	X[1]
	Plate bearing	X	X
	Standard penetration	X	
	Uniaxial (tunnel) jacking	X	X
	Borehole jacking[2]		X
	Chamber (gallery) pressure[2]		X
Relative density	Standard penetration	X	
	In situ sampling	X	
	Cone[2] penetrometer	X	
Liquefaction susceptibility	Standard penetration	X	
	Cone penetrometer test (CPT)[2]	X	

[1] Primarily for clay shales, badly decomposed, or moderately soft rocks, and rock with soft seams.
[2] Less frequently used.

a. Soils, clay shales, and moisture-sensitive rocks. Interpretation of in situ tests in soils, clay shales, and moisture-sensitive rocks requires consideration of the drainage that may occur during the test. Consolidation during testing makes it difficult to determine whether the test results correspond to unconsolidated-undrained, consolidated-undrained, consolidated-drained conditions, or intermediate conditions between these limiting states. The cone penetrometer test is very useful for detecting soft or weak layers and in quantifying undrained strength trends with depth. Interpretation of in situ test results requires complete evaluation of the test conditions and the limitations of the test procedure. ASTM D 3877-80 (ASTM 1996h) is the standard laboratory method for evaluating shrink/swell of soils due to subtraction or addition of water.

b. Rock. Rock formations are generally separated by natural joints, bedding planes, and other discontinuities resulting in a system of irregularly shaped blocks that respond as a discontinuum to various loading conditions. Response of a jointed rock mass to imposed loads involves a complex interaction of compression, sliding, wedging, rotation, and possibly fracturing of individual rock blocks. Individual blocks generally have relatively high strengths, whereas the strength along discontinuities is normally reduced and highly anisotropic. Commonly, little or no tensile strength exists across discontinuities. As a result, resolution of forces within the system generally cannot be accomplished by ordinary analytical methods. Large-scale, in situ tests tend to average out the effect of complex interactions. In situ tests in rock are generally expensive and should be reserved for projects with large, concentrated loads. Well-conducted tests, however, may be useful in reducing overly conservative, costly assumptions. Such tests should be located in the same general area as a proposed structure and test loading should be applied in the same direction as the proposed structural loading.

5-23. In situ Tests to Determine Shear Strength

Table 5-3 lists in situ tests that are useful for determining the shear strength of subsurface materials. In situ shear tests are discussed and compared by Nicholson (1983b) and Bowles (1996).

Table 5-3
In Situ Tests to Determine Shear Strength

	For			
Test	Soils	Rocks	Reference	Remarks
Standard penetration	X		EM 1110-2-1906 Appendix C	Use as index test only for strength. Develop local correlations. Unconfined compressive strength in tons/square foot is often 1/6 to 1/8 of N-value
Direct shear	X	X	RTH 321[1]	Expensive; use when representative undisturbed samples cannot be obtained
Field vane shear	X		EM 1110-2-1906, Appendix D, Al-Khafaji and Andersland (1992)	Use strength reduction factor
Plate bearing	X	X	ASTM[2] Designation D 1194 ASTM SPT 479[3]	Evaluate consolidation effects that may occur during test
Uniaxial compression		X	RTH 324[1]	Primarily for weak rock; expensive since several sizes of specimens must be tested
Cone penetrometer	X		Schmertmann (1978a);	Consolidated undrained strength of clays; requires
test (CPT)			Jamiolkowski et al. (1982)	estimate of bearing factor, N_c

[1] Rock Testing Handbook (USAEWES 1993).
[2] American Society for Testing and Materials (ASTM 1996a).
[3] Special Technical Publication 479 (ASTM 1970).

a. The Standard Penetration Test (SPT). The SPT is useful for preliminary appraisals of a site (Bowles 1996). The N-value has been empirically correlated with liquefaction susceptibility under seismic loadings (Seed 1979). The N-value is also useful for pile design. In cohesive soils, the N-value can be used to determine where undisturbed samples should be obtained. The N-value can also be used to estimate the bearing capacity (Meyerhof 1956; Parry 1977), the unconfined compressive strength of soils (Mitchell, Guzikowski, and Villet 1978), and settlement of footings in soil (Terzaghi, Peok, and Mesri 1996).

b. The Becker Penetration Test. The Becker drill (paragraph 5-5(2)) provides estimates of in situ soil strength and other properties similarly to the SPT, including coarse grained soils like gravel. The Becker penetration test was described in Harder and Seed (1986). The test consists of counting the number of hammer blows required to drive the casing 1 ft into the soil, for each foot of penetration. The test uses both open casing and plugged bits, commonly with a 14-cm (5.5-in.) or 17-cm (6.6-in.) OD casing and bits. Correlations of Becker blowcounts with SPT blowcounts have been developed to allow the use of Becker data in foundation investigations and in evaluation of liquefaction potential in coarse-grained soils under seismic loading.

c. Direct shear tests. In situ direct shear tests are expensive and are performed only where doubt exists about available shear strength data and where thin, soft, continuous layers exist within strong adjacent materials. The strength of most rock masses, and hence the stability of structures, is often controlled by the discontinuities separating two portions of the rock mass. Factors controlling the shear of a discontinuity include the loads imposed on the interface, the roughness of the discontinuity surfaces, the nature of the material between the rock blocks, and the pore water pressure within the discontinuity (Nicholson 1983b). In situ direct shear tests measure the shear strength along a discontinuity surface by isolating a block of rock and the discontinuity, subjecting the specimen to a normal load perpendicular to and another load (the shear load) parallel to the plane. The advantages of the direct shear test are: (1) its adaptability to field conditions, i.e., a trench, an adit, a tunnel, or in a calyx hole; (2) it is ideal for determining discontinuity shear strength because the failure plane and direction of failure are chosen before testing, accommodating anisotropic conditions; and (3) it allows for volume increases along the failure plane. Disadvantages of direct shear tests are their expense, the fact that they measure strength along only one potential failure plane, and the sometimes nonuniform application of normal stress during shearing. For the latter reasons, some engineers favor the triaxial compression test, which also can be performed in situ, for determination of shear strength (Ziegler 1972). The direct shear test measures peak and residual strength as a function of stress normal to the shear plane. Results are usually employed in limiting equilibrium analysis of slope stability problems or for stability analysis of foundations for large structures such as dams. Where field evidence suggests that only residual strengths can be relied on, either in a thin layer or in a mass, because of jointing, slickensiding, or old shear surfaces, in situ direct shear tests may be necessary. Few in situ direct shear tests are performed on soils, but they may be justified on clay shales, indurated clays, very soft rock, and on thin, continuous, weak seams that are difficult to sample. Ziegler (1972) and Nicholson (1983a,b) discuss the principles and methods of performing in situ direct shear tests. The Rock Testing Handbook (RTH) method RTH 321-80 (USAEWES 1993) provides the suggested method for determining in situ shear strength using the direct shear apparatus.

d. Field vane shear tests. Field vane tests performed in boreholes can be useful in soft, sensitive clays that are difficult to sample. The vane is attached to a rod and pushed into the soft soil at the bottom of the borehole. The assembly is rotated at a constant rate and the torque measured to provide the unconsolidated, undrained shear strength. The vane can be reactivated to measure the ultimate or residual strength (Hunt 1984). The vane shear test results are affected by soil anisotropy and by the presence of laminae of silt or sand (Terzaghi, Peck, and Mesri 1996). Shear is applied directionally. Failure of the soil occurs by shearing of horizontal and vertical surfaces. See Al-Khafaji and Andersland (1992) for a discussion of effects of soil anisotropy. The test may give results that are too high. Factors to correct the results are discussed in Bjerrum (1972) and Mitchell, Guzikewski, and Villet (1978). The test has been standardized as ASTM method D 2573 (ASTM 1996e).

e. Plate bearing tests. Plate bearing (plate-load) tests can be made on soil or soft rock. They are used to determine subgrade moduli and occasionally to determine strength. The usual procedure is to jack-load a 30- or 76-cm (12- or 30-in.) diam plate against a reaction to twice the design load and measure the deflection under each loading increment. The subgrade modulus is defined as the ratio of unit pressure to unit deflection, or force/length^3 (Hunt 1984). Because of their cost, such tests are normally performed during advanced design studies or during construction.

f. Cone penetrometer test or dutch cone. The Cone Penetrometer Test (CPT) can provide detailed information on soil stratigraphy and preliminary estimations of geotechnical properties. Based on the soil type as determined by the CPT (Douglas and Olsen 1981) or adjacent boring, the undrained strength can be estimated for clays (Jamiolkowski et al. 1982, Schmertmann 1970), and the relative density (and friction angle) estimated for sands (Durgunoglu and Mitchell 1975; Mitchell, Guzikewski, and Villet 1978; Schmertmann 1978b). For clays, a bearing factor, N_c, must be estimated to calculate the undrained strength from the CPT cone resistance and should be close or slightly greater than the CPT sleeve friction resistance if the soil is not sensitive or remolded (Douglas and Olsen 1981). The calculated undrained strength as well as the change of undrained strength with depth can both be used with several techniques to estimate the overconsolidation ratio (OCR) (Schmertmann 1978a). For sands, the relative density can be estimated if the overconsolidation conditions (i.e., lateral stress ratio) and vertical effective stress are known. The friction angle can also be estimated but also depends on the cone surface roughness and the assumed failure surface shape (Durgunoglu and Mitchell 1975). The mechanical (i.e., Dutch) cone is performed at a depth interval of 20 cm (8 in.) using hydraulic gages which measure the force from an inner rod directly in contact with the end of the probe. The electric (PQS or VGRO) cone is pushed at a constant speed for 1-m intervals while electronically measuring cone and friction resistance continuously.

5-24. Tests to Determine In Situ Stress

Table 5-4 lists the field tests that can be used to determine in situ stress conditions. The results are used in finite element analyses, estimating loading on tunnels, determining rock burst susceptibility in excavations, and identifying regional active and residual stresses. Stresses occur as a result of gravity forces, actively applied geologic forces such as regional tectonics, and from stored residual-strain energy. Stress is measured to determine the effect on foundations of changes in loading brought about by excavation or construction. Where a confining material has been removed by natural means or by excavation, the remaining material tends to approach a residual state of stress. In a majority of projects, the major principal stress is vertical, i.e., the weight of the overlying material. However, it has been found from measurements made throughout the world that horizontal stresses in the near-surface vicinity, defined as 30 m (100 ft) or less, can be one and one-half to three times higher than the vertical stress. Recognition of this condition during the design phase of investigations is very important. Where high horizontal stresses occur at a project site, the stability of cut slopes and tunnel excavations is affected. In situ testing is the most reliable method for obtaining the magnitude and direction of stresses. The three most common methods for determining in situ stresses are the overcoring, hydrofracture, and flatjack techniques.

Table 5-4
In Situ Tests to Determine Stress Conditions

Test	Soils	Rocks	Bibliographic Reference	Remarks
Hydraulic fracturing	X		Leach (1977) Mitchell, Guzikowski, and Villet (1978)	Only for normally consolidated or slightly consolidated soils
Hydraulic fracturing		X	RTH 344[1] Goodman (1981) Hamison (1978)	Stress measurements in deep holes for tunnels
Vane shear	X		Blight (1974)	Only for recently compacted clays, silts
Overcoring techniques		X	RTH 341[1] Goodman (1981) Rocha (1970)	Usually limited to shallow depth in rock
Flatjacks		X	RTH 343[1] Deklotz and Boisen (1970) Goodman (1981)	
Uniaxial (tunnel) jacking	X	X	RTH 365[1]	May be useful for measuring lateral stresses in clay shales and rocks, also in soils
Pressuremeter (Menard)	X		Al-Khafaji and Andersland (1992), Hunt (1984)	

[1] Rock Testing Handbook (USAEWES 1993).

a. Overcoring method. Possibly the most common method used for measuring in situ stresses in rock is overcoring, a stress-relief technique. An NW (75.7 mm (2.980 in.)) core hole is drilled, instrumented, and redrilled with a larger core barrel. The overcoring decouples the rock surrounding the instrument package from the natural stress field of the in-place formation. The change in strain recorded by the instruments is then converted to stress by using the elastic modulus of the rock determined from laboratory tests. At least three separate tests must be made in the rock mass in nonparallel boreholes. A detailed description of the field test is provided by RTH 341-80 (USAEWES 1993). The overcoring method is hampered by the necessity for many instrument lead wires that may be broken during testing. The practical maximum depth of testing is usually less than 45 m (150 ft).

b. Flatjack method. In the flatjack method, a slot is bored or cut into the rock wall midway between two inscribed points. Stresses present in the rock will tend to partially close the slot. A hydraulic flatjack is then inserted and grouted into the slot, and the rock is jacked back to its original position as determined by the inscribed points. The unit pressure required is a measure of the in situ stress. Flatjacks installed at different orientations provide a measure of anisotropy (Hunt 1984). The value recorded must be corrected for the influence of the tunnel excavation itself. Flatjack tests require an excavation or tunnel. The high cost for constructing the opening usually precludes this technique as an indexing tool except where the size of the structure and complexity of the site dictate its use.

c. Hydrofracture method. The hydrofracture method has been used in soils and rock. A section of hole is isolated with packers at depth, and an increasingly higher water pressure is applied to the zone. A point will be reached where the pressure begins to level off, and there is a marked increase in water take.

This indicates that a crack in the formation has opened, and the threshold pressure has been reached. The threshold pressure measures the minor principal stress component. The orientation is then obtained by an impression packer. This procedure then gives the intensity and direction of the minor principal stress, which is perpendicular to the crack. The hydrofracture method has no particular depth limitation, but drilling deep holes can be very expensive. This expense can often be circumvented by using holes that have been drilled for other purposes. Evidence indicates that stresses measured within 30 m (100 ft) or more of ground surface may not always reflect the actual stress magnitude or orientation at depth. This may be true particularly in areas where closely jointed and weathered surface rock formations are decoupled from the deeper, more intact rock.

5-25. Tests to Determine In Situ Deformation

Deformation characteristics of subsurface materials are of major importance in dynamic and seismic analyses for dams and other large structures, static design of concrete gravity and arch dams, tunnels, and certain military projects. Geotechnical investigations for such purposes should be planned jointly by geotechnical personnel and structural engineers. Deformation properties are normally expressed in terms of three interdependent parameters: Young's modulus, shear modulus, and Poisson's ratio. These parameters assume that materials are linear, elastic, homogeneous, and isotropic. In spite of this limitation, these parameters are often used to describe the deformation properties of soil and rock. Large-scale tests (e.g., tunnel jacking) are frequently used because they reduce the effect of nonhomogeneity. Multiple tests, with different orientations, can be used to determine the anisotropy of the deformation properties. Soils, in particular, tend to be nonlinear and inelastic. As a result, their properties are often strain dependent, i.e., moduli determined at low strain levels can be substantially different from those determined at high strain levels. The fact that sample disturbance, particularly in soils, can substantially affect the deformation properties serves as the primary reason for using in situ tests in soils. Table 5-5 lists the in situ tests used to determine one or more of the deformation parameters. Some test results are difficult to relate to the fundamental parameters but are used directly in empirical relationships (Table 5-6). Deformation properties of a jointed rock mass are very important if highly concentrated loadings are directed into the abutments of arch dams in directions that are tangent to the arches at the abutments. In these cases, the ratio of the deformation modulus of the abutment rock to that of the concrete in the dam must not be so low as to cause adverse tensile stresses to develop within the concrete dam. One problem often encountered in conducting in situ deformation tests is the need to include representative sizes of the jointed rock mass in the test, particularly if the joint spacing is moderately large (e.g., 0.6 to 0.9 m or 2 to 3 ft). This problem has been solved in some instances by excavating a chamber in rock, lining it with an impermeable membrane, and subjecting it to hydraulic pressure to load the rock over relatively large areas.

a. Chamber tests. Chamber tests are performed in large underground openings, such as exploratory tunnels. Preexisting openings, such as caves or mine chambers, can be used if available and applicable to project conditions. The opening is lined with an impermeable membrane and subjected to hydraulic pressure. Instrumented diametrical gages are used to record increases in tunnel diameter as the pressure load increases. The test is performed through several load-unload cycles. The data are subsequently analyzed to develop load-deformation curves from which a deformation modulus can be selected. The results are usually employed in the design of dam foundations and for the proportioning of pressure shaft and tunnel linings. The chamber test method is described by RTH 361-89 (USAEWES 1993).

Table 5-5
In Situ Tests to Determine Deformation Characteristics

Test	For Soils	For Rocks	Reference	Remarks
Geophysical refraction, cross-hole and downhole	X	X	EM 1110-1-1802	For determining dynamic Young's Modulus, E, at the small strain induced by test procedure. Test values for E must be reduced to values corresponding to strain levels induced by structure or seismic loads
Pressuremeter	X	X	RTH 362[1] Baguelin, Jezequel, and Shields (1978) Mitchell, Guzikowski, and Villet (1978)	Consider test as possibly useful but not fully evaluated. For soils and soft rocks, shales, etc.
Chamber test	X	X	Hall, Newmark, and Hendron (1974) Stagg and Zienkiewicz (1968)	
Uniaxial (tunnel) jacking	X	X	RTH 365[1] Stagg and Zienkiewicz (1968)	
Flatjacking		X	RTH 343[1] Deklotz and Boisen (1970) Goodman (1981)	
Borehole jack or dilatometer		X	RTH 363[1] Stagg and Zienkiewicz (1968)	
Plate bearing		X	RTH 364[1] ASTM STP 479[2] Stagg and Zienkiewicz (1968)	
Plate bearing	X		MIL-STD 621A, Method 104	
Standard penetration	X		Hall, Newmark, and Hendron (1974)	Correlation with static or effective shear modulus, in Mpa (psi), of sands; settlement of footings on clay. Static shear modulus of sand is approximately: $G_{eff} = 1960\ N^{0.51}$ in Mpa (psi); N is SPT value

[1] Rock Testing Handbook (USAEWES 1993).
[2] American Society for Testing and Materials, Special Technical Publication 479 (ASTM 1970).

Table 5-6
Correlations Between Field Tests for Soils, Material Characteristics, and Structural Behavior

Field Test	Correlation With	Remarks
1 x 1-ft plate load test	Modulus of subgrade reaction. Settlement of footings on sand	Mitchell, Guzikowski, and Villet (1978)
Load test for radar towers	Young's modulus of subgrade soils	MIL-STD-621A
Standard penetration N-value	Settlement of footings and mats on sand; shear modulus	TM 5-818-1; Hall, Newmark, and Hendron (1974) Meyerhof (1956) Parry (1977) U.S. Army Engineer Waterways Experience Station (1954)
Cone penetrometer test	ö of sands; settlement of footings on sand; relative density	Mitchell, Guzikowski, and Villet (1978) Mitchell and Lunne (1978) Schmertmann (1978b) Durgunoglu and Mitchell (1975) Schmertmann (1970) Schmertmann (1978a,b)

b. Uniaxial jacking test. An alternative to chamber tests is the uniaxial jacking test (RTH 365-80 (USAEWES 1993)). The test uses a set of diametrically opposed jacks to test large zones of soil and rock. This method produces nearly comparable results with chamber tests without incurring the much greater expense. The test determines how foundation materials will react to controlled loading and unloading cycles and provides data on deformation moduli, creep, and rebound. The uniaxial jacking test is the preferred method for determining deformation properties of rock masses for large projects.

c. Other deformation tests. Other methods for measuring deformation properties of in situ rock are anchored cable pull tests, flatjack tests, borehole jacking tests, and radial jacking tests. The anchored cable pull test uses cables, anchored at depth in boreholes, to provide a reaction to large slabs or beams on the surface of the rock. The test is expensive and difficult to define mathematically but offers the advantages of reduced shearing strains and larger volumes of rock being incorporated in the test. Flatjack tests are flexible, and numerous configurations may be adopted. In relation to other deformation tests, the flatjack test is relatively inexpensive and useful where direct access is available to the rock face. Limitations to the method involve the relatively small volume of rock tested and the difficulty in defining a model for calculation of deformation or failure parameters.

(1) The borehole jack ("Goodman" jack), or dilatometer, and the Menard pressuremeter (Terzaghi, Peck, and Mesri 1996; Al-Khafaji and Andersland 1992; and Hunt 1984), which are applied through a borehole, have the primary advantage that direct access to the rock or soil face is not required. The dilatometer determines the deformability of a rock mass by subjecting a section of a borehole to mechanical jack pressure and measuring the resultant wall displacements. Elastic and deformation moduli are calculated. The pressuremeter performs a similar operation in soils and soft rock. The development of a mathematical model for the methods has proved to be more difficult than with most deformation measurement techniques.

(2) Radial jacking tests (RTH 367-89 (USAEWES 1993)) are similar in principle to the borehole jacking tests except that larger volumes of rock are involved in the testing. Typically, steel rings are placed within a tunnel with flatjacks placed between the rings and the tunnel surfaces. The tunnel is loaded radially and deformations are measured. The method is expensive but useful and is in the same category as chamber tests. All methods of deformation measurements have inherent advantages and disadvantages, and thus selection of test methods must be dictated by the nature of the soil or rock mass, the purpose of the test, and the magnitude of the project. Care must be exercised and limitations recognized in the interpretation and use of measurements of deformation.

5-26. Determination of Dynamic Moduli by Seismic Methods

Seismic methods, both downhole and surface, are used on occasion to determine in-place moduli of soil and rock (see Table 5-2). The compressional wave velocity is mathematically combined with the mass density to estimate a dynamic Young's modulus, and the shear wave velocity is similarly used to estimate the dynamic rigidity modulus. However, because particle displacement is so small and loading is transitory during these seismic tests, the resulting modulus values tend to be too high. The seismic method of measuring modulus should not be used in cases where a reliable static modulus value can be obtained. Even where the dynamic modulus is to be used for earthquake analyses, the modulus derived from seismic methods is too high. The moduli and damping characteristics of rock are strain dependent, and the strains imposed on the rock during seismic testing are several orders of magnitude lower than those imposed by a significant earthquake. Generally, as the strain levels increase, the shear modulus and Young's modulus decrease and the damping increases. Consideration of these factors is necessary for earthquake analyses.

Section VII
Backfilling of Holes and Disposition of Samples and Cores

5-27. Backfilling Boreholes and Exploratory Excavations

Except where the hole is being preserved for future use, all boreholes and exploratory excavations should be backfilled. The reasons for backfilling holes are to: eliminate safety hazards for personnel and animals, prevent contamination of aquifers, minimize underseepage problems of dams and levees, and minimize adverse environmental impacts. Many states have requirements regarding backfilling boreholes; therefore, appropriate state officials should be consulted. Holes preserved for the installation of instrumentation, borehole examination, or downhole geophysical work should be backfilled when no longer needed. As a minimum, borings that are preserved for future use should be protected with a short section of surface casing, capped, and identified. Test pits, trenches, and shafts should be provided with suitable covers or barricades until they are backfilled. Where conditions permit, exploratory tunnels may be sealed in lieu of backfilling. Procedures for backfilling boreholes and exploratory excavations are discussed in Appendix F, Chapter 10, of this manual.

5-28. Disposition of Soil Samples

Soil samples may be discarded once the testing program for which they were taken is complete. Geotechnical samples of soil and/or rock collected from areas suspected of containing HTRW *should be properly disposed of.* Soil samples are not normally retained for long periods, because even the most careful sealing and storing procedures cannot prevent the physical and chemical changes that, in time, would invalidate any subsequent test results. Requirements for the disposition of soil samples from plant pest quarantined areas are specified in ER 1110-1-5.

5-29. Disposition of Rock Cores

All exploratory and other cores not used for test purposes shall be properly preserved, boxed, and stored in a protected storage facility until disposal. The following procedures govern the ultimate disposition of the cores.

a. Care and storage. Filled core boxes can be temporarily protected at the drilling site by wrapping them in plastic sheeting and preventing direct contact of the boxes with the ground. Exploratory or other cores, regardless of age, will be retained until the detailed logs, photographs, and test data have been made a matter of permanent record. Precautions shall be taken to ensure against the disposal, destruction, or loss of cores that may have a bearing on any unsettled claim. Such cores shall be retained until final settlement of all obligations and claims. They then will be disposed of in accordance with the procedures outlined in the following text.

b. Disposal. Cores over 15 cm (6 in.) in diameter may be discarded after they have served their special purpose. Geotechnical samples of soil and/or rock collected from areas suspected of containing HTRW *should be properly disposed of.* In a case where the project is deauthorized, all associated cores may be discarded. When a project has been completed and final settlement has been made with the contractors and others concerned, all cores, except those related to future construction, and a few selected cores representative of foundation and abutment conditions, may be discarded. Selected cores, retained after the completion of a project, and additions thereto, may be discarded or otherwise disposed of 5 years after final completion of the project, provided no unforeseen foundation or abutment conditions have developed. After cores are disposed of, core boxes should be salvaged for reuse if their condition permits.

Chapter 6
Large-scale, Prototype Investigations

6-1. Prototype Test Programs

Whenever the size and complexity of a project warrant, large-scale, prototype test programs can yield information unavailable by any other method. Because these investigations are expensive and require the services of a construction contractor in most cases, they are commonly included as part of a main contract to confirm design assumptions. However, if performed during the PED phase, they can provide a number of benefits that will result in an improved, more cost-effective design. These benefits include: confirmation of assumptions for new or innovative design, improved confidence level allowing reduced safety factors, proof of constructibility, confirmation of environmental compliance, and greater credibility in allaying public concerns. Evaluation of the potential for savings and benefits should be made by an experienced engineering geologist or geotechnical engineer. A major pit-fall of the large-scale prototype test is that it is commonly difficult, if not impossible, to precisely follow the same procedures in the main contract that were used in the test. This does not eliminate the benefits to be gained but should be kept in mind when deciding how to incorporate the information into a bid package.

Section I
Test Excavations and Fills

6-2. Accomplishments

In most cases, a test excavation or fill technique such as blasting and rippability accomplishes one or more of the following requirements: (a) evaluates the suitability of specialized construction equipment such as coal saws or vibratory rollers; (b) investigates the influence of material properties on construction products such as blasted rock gradations; (c) provides the opportunity for preconstruction monitoring of ground reactions to test design assumptions; (d) more completely discloses the geologic conditions; (e) investigates material placement properties and procedures; (f) investigates environmental impacts such as blasting vibrations or ground water lowering; and (g) provides access to install initial ground support and instrumentation.

6-3. Test Quarries

Test quarries are usually implemented in conjunction with test fill programs and in areas where large quantities of rock material will be needed from undeveloped sources. EM 1110-2-2301 discusses test quarry and test fill evaluation procedures. Test quarries are especially important where there are serious questions about the suitability of rock in required excavations for use in embankment rock-fill zones or for slope protection. In addition to providing material for rock test fills, test quarries provide information on cut slope design constraints resulting from adverse geologic structure, suitable blasting techniques, suitability of quarry-run rock, and the feasibility and best methods for processing materials. The results of quarry tests can provide designers and prospective bidders with a much better understanding of drilling and blasting characteristics of the rock. Although useful information is gained from a well conducted test quarry program, it is an expensive type of investigation. If possible, a test quarry should be located in an area of required excavation. Excess materials from the test quarry can be stockpiled for later use. Determining the optimum methods of precision slope development (e.g., best presplitting blasthole spacing and powder factors) can be an important part of the test quarry program. This determination ensures maximum side slope stability by minimizing overbreakage. Mapping of test quarry slopes, in conjunction with the use of slope stability analysis programs such as ROCKPACKII (Watts 1996) and Discontinuous Deformation Analysis Program (International Forum on Discontinuous Deformation Analysis and Simulations of Discontinuous Media 1996), can provide needed geologic data for use in design of permanent slopes. To be of maximum benefit, the test quarry should be located in a portion of the excavation area that is representative of the geologic conditions to be encountered.

a. Geologic study. Before a test quarry program is undertaken, a careful geologic study should be made of the test quarry site. The geologic study should include:

(1) Field reconnaissance and mapping of exposed rock jointing and discontinuities.

(2) Examination of boring logs, rock cores, and borehole survey results to determine depth of overburden and weathered rock, joint patterns, presence of filled solution joints or fault zones, and ground water conditions that could affect blasting operations.

(3) Consideration of regional stress fields and site-specific stress conditions that could affect stress relief in joints during quarrying operations.

(4) Development of geologic sections and profiles depicting rock type and stratum thickness, joint spacing, frequency and orientation, filled joint systems, and other discontinuities that would influence rock breakage and the amount of fines.

(5) Consideration of all other factors that may control size, quantity, and quality of blasted rock (e.g., proximity to structures or urban areas where blast size, airblast, ground vibrations, or fly rock may have to be rigidly controlled).

b. Test objectives. Once geologic studies indicate that a quarry source can produce an adequate quality and quantity of fill and construction material and the range of aggregate size has been determined, other data needed for test quarry design purposes include overall gradation, yield, quality, and production. Blasting techniques and modifications to fit geologic conditions are discussed in EM 1110-2-3800. The upper fragment size is determined by the geologic conditions and rock structure. Other sizes and gradations can be controlled partially by the blasting techniques.

c. Test program. In a well-planned test quarry program, many of the blasting variables, such as blasthole spacing, patterns, and firing sequences, powder brisance, powder factor, and bench height, are subjected to field experimentation. The program should be developed and supervised by an experienced geologist or geotechnical engineer. By dividing the test quarry into separate tests, each of the variables can be evaluated separately while the others are held constant. For each test, the blasted rock must be gathered, sieved, and weighed to obtain gradations. For aggregate studies, representative samples should be taken for processing, testing, and mix design. If blasted rock is also being used for rock test fills, it may be necessary to use representative truck loads for gradation processing. As individual test blasts are completed and gradations determined, modifications to the blasting technique can be made. When all individual test blasts and associated gradations are completed, the data should be reviewed to determine which set of blasting parameters best fulfills design requirements. Test quarry programs are discussed in EM 1110-2-2301, Bechtell (1975), Lutton (1976), and Bertram (1979).

d. Application. The results of test quarry programs are expressed in terms of optimum blasting patterns, powder factors, blasthole sizes, firing delay sequences, yields, and gradations. These results, combined with results of test fills, form a valuable source of data for the quarry designer. This information is equally valuable to prospective contractors and should be provided for information in the plans and specifications.

6-4. Exploratory Tunnels

Exploratory tunnels permit detailed examination of the composition and geometry of rock structures such as joints, fractures, faults, shear zones, other discontinuities, and solution channels. They are commonly used to explore conditions at the locations of large underground excavations and the foundations and abutments for large dams. They are particularly appropriate in defining the extent of marginal strength rock or adverse rock structure suspected from surface mapping and boring information. For major projects where high-intensity loads will be transmitted to foundations or abutments, tunnels afford the only practical means for testing in-place rock at locations and in directions corresponding to the structural loading. Although expensive, exploratory tunnels provide exceptionally good preconstruction information to perspective contractors on major underground projects and can reduce bid contingencies and/or potential for claims. Long horizontal exploratory drill holes can also be used in lieu of, or in combination with, pilot tunnels to gather information about tunneling conditions prior to mining.

a. In the case of planned underground construction, an exploratory tunnel is often used to gain access to crown and roof sections of future large underground excavations. The tunnel can then be used during construction for equipment access and removal of excavated rock. A small bore or exploratory "pilot" tunnel is sometimes driven along the entire length of a proposed larger-diameter tunnel where difficult and often unpredictable ground conditions are anticipated. A pilot tunnel may be the most feasible alternative for long deep tunnels where deep exploratory drilling and access for in situ testing from the ground surface is prohibitively expensive. The pilot tunnel can be positioned to allow installation of roof support and/or consolidation grouting for critical areas of the full tunnel, or in some cases, to provide relief or "burn cuts" to facilitate blasting. Exploratory tunnels that are strategically located commonly can be incorporated into the permanent structure. They can be used for drainage and postconstruction observations to determine seepage quantities and to confirm certain design assumptions. On nonwater related projects, exploratory tunnels may be used for permanent access or for utility conduits. One concern, however, is not to position the pilot tunnel too close to the projected crown excavation neat line of the full-sized tunnel. If this occurs, overbreak in the crown of the tunnel can have a negative impact on mining and stabilizing the crown of the full-sized tunnel.

b. The detailed geology of exploratory tunnels, regardless of their purpose, should be mapped in accordance with the procedures outlined in Appendix C. The cost of obtaining an accurate and reliable geologic map of a tunnel is insignificant compared to the cost of the tunnel and support system. The geologic information gained from such mapping provides a very useful dimension to interpretations of rock structure deduced from exposures in surface outcrops. A complete picture of the site geology can be achieved only if the geologic data and interpretations from surface mapping, borings, and pilot tunnels are combined and well correlated. Such analyses are best carried out using a GIS.

6-5. Test Fills and Trial Embankments

a. Test fills. Test fills are generally recommended where unusual soils or rockfill materials are to be compacted or if newly developed and unproven compaction equipment is to be employed. Test fills are valuable for training earthwork inspectors on large projects, especially if materials vary widely or if compaction control procedures are complex. Test fills constructed solely to evaluate new or different compaction equipment are ordinarily performed by the contractor at his expense. Rock test fills are most frequently required to determine optimum placement and compaction operations. Test quarries are often associated with a rock test fill program to determine blasting requirements and to establish any required preplacement material processing. Test fills must be constructed ahead of contract advertisement as they are necessary to establish specifications. It is most economical if such test fills can be located in low stressed regions of the embankment and incorporated within the final embankment section. If cofferdams of compacted fill are required on the project, these can be utilized as test fills but only if their serviceability is not affected. In the past, COE Districts have found it most satisfactory to construct precontract test fills themselves by renting the necessary equipment or even letting a separate contract. The following two considerations are most important in execution of a test fill program:

(1) Plan of tests. Usually several different parameters are to be evaluated from a test fill program. Therefore, the test program should be thoroughly planned so that each parameter is properly isolated for evaluation. All aspects of the program must be treated in detail, particularly the means of measurements and controls and data reduction.

(2) Representative materials and procedures. The test fill operations must be representative with regard to prototype materials and placement and compaction procedures. This is especially critical in rock test fill programs associated with test quarries. For additional information, see EM 1110-2-1911 and Hammer and Torrey (1973).

b. Trial embankments. Trial embankments are infrequently used but may be the only reliable means for resolving uncertainties about the probable behavior of complex subsurface conditions or of poor quality embankment materials. For example, trial embankments were constructed at Laneport, TX (Parry 1976), Warm Springs, CA (Fagerburg, Price, and Howington 1989), and R. D. Bailey, WV Dams (Hite 1984).

(1) Where subsurface shear strengths are so low that the gain in strength from consolidation during construction must be relied upon, or if it is economical to do so, a trial embankment is desirable, especially where long embankments are to be constructed. A trial embankment affords the most reliable means for determining the field rate of consolidation and efficacy of methods to accelerate consolidation.

(2) Clay shale foundations are often jointed and slickensided and may contain continuous relict shear surfaces. Laboratory shear tests on undisturbed samples generally give too high shear strengths and may be badly misleading. The in situ mass strength of clay shales can best and frequently only be determined by analyzing existing slopes or constructing trial embankments. If trial embankments are incorporated within the final section, their height and slopes must be designed to result in desired shear stresses in the foundation. Where natural slopes are flat, suggesting that residual shear strengths govern stability of slopes and cuts, trial embankments can be useful in resolving uncertainties about available shear strengths; i.e., are natural slopes flat because residual shear strengths have developed or because of natural erosion processes and a mature landscape?

(3) If special circumstances indicate the desirability of using wet, soft clay borrow that cannot economically be reduced to conventional compaction water contents, a trial embankment should be strongly considered.

Section II
Test Grouting

6-6. Purpose

Test grouting operations are performed at projects where complex geological conditions or unusually severe project requirements make it necessary to acquire a knowledge of grouting performance prior to the letting of major contracts. Test grouting programs provide information necessary to formulate procedures and determine design specifications, costs, and appropriate equipment. Test grouting consists of performing experimental grouting operations on exploratory boreholes to determine the extent to which the subsurface materials are groutable. A well-conceived and well-conducted grout program can provide cost-effective data for the preparation of contract plans and specifications. This can reduce the potential for construction claims. Grouting procedures necessary for development of a satisfactory test grouting program are discussed in detail in TM 5-818-6, and EM 1110-2-3506.

6-7. Test Grouting Practices

In test grouting, the methods used should be guided by the geologic conditions at the site. For example, stage grouting is preferable in rock formations where joint permeability prevails and the weight of increasing overburden with depth tends to close and tighten joint passageways. In solutioned limestone formations or pervious lava flows, major water passageways may not decrease in size with depth. Consequently, stop grouting is preferable because this procedure is initially directed at the source of water seepage. Circle grouting is a more comprehensive test procedure. Multiple line grouting is also a comprehensive grout testing procedure but does not require as many grout holes as the circle grouting test.

6-8. Test Grouting Program

A test grouting program is nearly always performed in a small area. Where the ground water table is located in the limits of the curtain, it may be necessary to construct the grout curtains in closed circular or rectangular arrays. In this manner, the effectiveness of the grout curtain can be evaluated by performing pumping tests before and after grouting. The test well is usually located at the center of the grout curtain enclosure. Observation wells are positioned to radiate from one or more directions outward from the well and through the test curtain. Comparison of the reduction of water pumped from the well before and after grouting is a direct measure of the efficiency of the grout curtain. Where the water table is low or nonexistent, a multiple, linearly aligned curtain is sufficient for test purposes. Comparison of pre- and postgrouting pressure tests should be made to evaluate the effectiveness of the test grouting schemes. Some of the important variables a test grouting program should resolve are basic grouting methods, hole spacing, grout consistency and additives, and injection pressures. Some projects may require the testing and applicability of using chemical grout to solve difficult seepage or foundation competency problems.

6-9. Record Keeping

Meaningful evaluation of a grouting program is impossible without adequate record keeping. Variables that are pertinent in grouting are discussed in detail in TM 5-818-6 and EM 1110-1-3500 and will not be repeated here. They are numerous and unless the record keeping aspect of grouting is well-organized and given top priority, the value of the test grouting program will be lost. Records have traditionally been kept by hand on specially prepared forms. The Grouting Database Package (Vanadit-Ellis et al. 1995), a

PC-based, menu-driven program that stores and displays hole information, drilling activities, water pressure tests, and field grouting data is an improvement over manual record keeping methods and is available through USAEWES.

Section III
Piling Investigations

6-10. Piling Investigation Benefits

Piling investigations may be conducted prior to construction or, more commonly, as part of a construction project just prior to the start of production pile driving. Predicting the performance of driven piles has been extensively studied, and various methods, including some very sophisticated computer analyses, are available to the designer. While these are useful and adequate for predicting piling performance for small jobs, projects requiring large numbers of piles generally benefit from a preconstruction piling investigation. This is because the methods of analysis that are commonly used are conservative and may significantly underestimate the actual working capacity of pilings. In addition to providing reliable design load capacities, field tests can help answer questions such as: how much penetration into rock can be expected, what time-dependent effects can be expected, and do hard layers exist that will cause early refusal or make driving points a necessity? As in all aspects of geotechnical engineering, pile testing is subject to a margin of error which is dependent on the heterogeneity of the site being investigated. Where the site geology is very complex, the data from one pile load test will probably not be repeatable across the project site. In addition, the performance and efficiency of a pile driving hammer can vary tremendously, even for hammers with the same energy rating. Because the capacity of a pile is heavily influenced by the hammer, the results from a test program cannot be translated to the production piles on the final project without some reservations. All of these considerations aside, there are times when a pile testing program will result in a significant savings to a project or, at least, a high confidence level in the piles capacity. EM 1110-2-2906 provides detailed information of pile foundation design.

6-11. Driving Records

As with grouting, adequate evaluation of a pile driving job cannot be performed without adequate records. A sample form for recording pile driving data is shown in Figure 6-1. This form can be modified to meet particular designer's needs but generally lists the information that is of most interest.

6-12. Load Testing

The correct procedures for conducting a pile load test are given in numerous references including EM 1110-2-2906 and are not repeated here. Generally, a pile will be tested for axial load capacity, both in compression and tension, and for lateral resistance.

DAILY PILE RECORD FOR LARGE- AND SMALL-DIAMETER BORED PILES

PILE RECORDS TO BE SUBMITTED TO OFFICE DAILY
A SEPARATE SHEET TO BE USED FOR EACH PILE

BLOCK NUMBER	DRAWING NUMBER / /

1. General	Pile reference number		Pile diameter		Level of base	
			Underream diameter			
	Ground level		Cutoff level		Concreted level	
2. Drilling	Date started		Date completed			
	Error in position on plan					
3. Obstructions[1] Natural/ Unnatural	Type		Depth encountered		Penetration time	
	Type		Depth encountered		Penetration time	
4. Steel[1] main steel links or helix	Number of bars		Diameter		Length	
	Centers of bars/pitch		Diameter		Cover to all steel	
5. Concrete	Date started		Date completed		Concrete temperature	Quantity Actual: Theoretical:
	Mix		Slump		Supplier	
6. Borehole log and rock excavation	Depth of soil	Description of soil	Depth of rock	Description of rock	Depth of rock augered	Depth of rock chiseled
7. Casing[1]	Depth of temporary casing		Depth of permanent casing		Reason for use of permanent casing	
8. Water[1]	Depth encountered		Details of strong flow		Details of remedial measures	
	Depth to strong flow					

[1] If there are no changes to be recorded, items 3, 4, 7, and 8 need be completed for the first pile only in each block.

Remarks:

Signed: **Site Contract Engineer:**

Figure 6-1. Example of a piling log. Modified from Tomlinson (1994)

Chapter 7
Laboratory Investigations

7-1. Purpose

The purpose of laboratory tests is to investigate the physical and hydrological properties of natural materials such as soil and rock, determine index values for identification and correlation by means of classification tests, and define the engineering properties in parameters usable for design of foundations. The engineering geologist and/or geotechnical engineer, using the test data and calling upon experience, can then accomplish safe and economical designs for engineering structures. Procedures to assure quality in laboratory testing are outlined in ER 1110-1-261 and -8100. This chapter is divided into four sections that discuss selection of testing methods and samples, index and classification tests, engineering properties of soil, engineering properties of rock, and engineering properties of shales and moisture-sensitive rocks. No attempt has been made to describe the techniques for performing individual tests; references are provided for that purpose. A wide range of soil and rock tests are identified, and the appropriate applications are discussed.

Section I
Test and Sample Selection

7-2. Needs for Test and Sample Selection

The selection of samples and the number and type of tests are largely influenced by local subsurface conditions and the size and type of structure. Table 7-1 lists references that provide guidance for assigning laboratory tests for various types of structures. As a minimum, all soil samples should be classified according to the USCS (paragraph 5-8), and moisture contents should be determined on cohesive soils and on unsaturated granular soils that have 12 percent or more fines. Rock cores should be visually classified and logged prior to laboratory testing. The geologic model (paragraph 3-1) can be further developed using the results of basic indexing of soils and rock cores, together with other geotechnical data obtained from field reconnaissance and preliminary investigations. The geologic model, in the form of profiles and sections, can be used to indicate where additional indexing of soils and rock is needed, as well as the type and number of tests required to determine the engineering properties of all materials influencing the project. As more data become available, the testing requirements should be reviewed and modified as necessary.

a. Selection of samples for testing. Most index testing of soil and rock is performed on disturbed samples, i.e., samples that have not had special handling to preserve structural integrity. However, to determine natural water content the sample must be protected from drying. For soils, protection can be accomplished by using sealed metal tubes or plastic or glass jars. For rock, samples are normally waxed to prevent drying. Because many laboratory tests, particularly those to determine engineering properties, require "undisturbed" samples, great care must be exercised in storing, selecting, shipping, and preparing these materials. The geologist and/or geotechnical engineer responsible for applying test data to project requirements should have positive control of sampling and shipping of soil and rock samples. Table 7-2 lists some of the factors that may cause undisturbed samples to be less representative of the conditions encountered on the project.

b. Distribution and size of samples. The distribution of locations of soil and rock tests should be evaluated periodically. Within the project requirements, a suitable suite of index and engineering

Table 7-1
Guidance for Assigning Laboratory Tests

Type of Structure or Work	Reference
Embankment dams	EM 1110-2-2300 EM 1110-2-1902
Concrete gravity dams	EM 1110-2-2300 EM 1110-2-1902
Buildings and other structures	TM 5-818-1
Deep excavations	TM 5-818-5/AFM 88-5, Chapter 6/NAVFAC P-418
Tunnels and shafts in rocks	EM 1110-2-2901
Breakwaters	EM 1110-2-2904
Pile structures and foundations	EM 1110-2-2906
Levees	EM 1110-2-1913

Table 7-2
Factors Causing Undisturbed Samples to be Less Representative of Subsurface Materials

	Effect on	
Factor	**Soils**	**Rocks**
Physical disturbance from sampling and transportation	Effect on shear strength: a. Reduces Q and UC strength. b. Increases R strength. c. Little effect on S strength. d. Decreases cyclic shear resistance. Effect on consolidation test results: a. Reduces P b. Reduces $C^{c\bullet}$ c. Reduces c_v in vicinity of σ_p and at lower stresses. d. Reduces $C_{\alpha\bullet}$	Cause breaks in core; may be difficult to obtain intact specimens suitable for testing May seriously affect weakly cemented materials, e.g., for sandstones, may destroy evidence of significant cementation. Foundation may appear to be more fractured than it is May prevent testing of some materials May reduce deformation modulus, E
Changed stress conditions from in situ to ground surface locations	Similar to physical disturbance but less severe	Stress relief may cause physical disturbance similar to that from sampling and transportation. Deformation modulus reduces with decreasing stress field
Contamination of sands from drilling mud	Greatly reduces permeability of undisturbed samples	

Note: Q = Unconsolidated-undrained triaxial test; UC = Unconfined compression test; σ_p = Preconsolidation pressure; C_c = Compression index; c_v = Coefficient of consolidation; = C_α= Coefficient of secondary compression; R = Consolidation-undrained triaxial test; S = Drained direct shear test.

property tests should be planned both in the vertical as well as lateral direction. Duplication of costly, complex tests should be avoided except where statistical balance is required. If it becomes apparent in the application of the test data that coverage of field conditions is irregular, or missing in certain stratigraphic units, field sampling procedures should be revised. Undisturbed sample sizes for soils should conform to those given in Appendix F. Rock sample sizes can range from 4.763 to 20 cm (1.875 to 6.0 in.). Large-diameter cores are obtained in lieu of the smaller core sizes in cases where rock defects make core recovery and sample quality difficult to attain. In some cases, the test procedure may dictate sample size. Rock tests and procedures are presented in the Rock Testing Handbook (USAEWES 1993).

Section II
Index and Classification Tests

7-3. Soils

Types of index and classification tests that are typically required are listed in Table 7-3 together with their reporting requirements. Initially, disturbed samples of soils are classified according to the USCS. Upon visual verification of the samples, Atterberg limits, mechanical analyses, and moisture content tests will be performed (Schroeder 1984: Gillott 1987). Table 7-3 also presents two other index tests relating to durability under cyclic weather conditions (slaking tests), and shear strength (torvane and penetrometer). The torvane and penetrometer shear tests are simple and relatively inexpensive; however, the test results can be widely variable and should be used with caution. These shear tests can be helpful as a guide to more comprehensive tests. Slaking tests are valuable if the project is located in moisture sensitive clays and clay shales, and foundation design requirements indicate that the foundation and cut slope areas will be exposed temporarily to wetting and drying conditions.

Table 7-3
Index and Classification Tests for Soils

Test	Remarks
Water content[1]	Required for every sample except clean sands and gravels
Liquid limit and plastic limit[1]	Required for every stratum of cohesive material; always have associated natural water content of material tested (compute liquidity index) [2]
Sieve	Generally performed on silts, sands, and gravels (> 200 mesh)
Hydrometer analysis	Generally performed on soils finer than the No. 10 sieve size (medium sand and finer). Correlate with Atterberg limit tests to determine the plasticity of the soil
Slaking test	Performed on highly preconsolidated clays and clay shales where deep excavations are to be made or foundations will be near-surface. Wet and dry cycles should be used
Pocket penetrometer and torvane	Performed on cohesive materials, undisturbed samples, and intact chunks or disturbed samples. Regard results with caution; use mainly for consistency classification and as guide for assigning shear tests
X-ray diffraction	Generally performed on clays and clay shales to determine clay mineralogy which is a principal indicator of soil properties

[1] See EM 1110-2-1906 for procedures.
[2] Liquidity index = LI = $\frac{w_n - PL}{LL - PL}$. (w_n=natural water content).

7-4. Rock

All rock cores will be logged in the field and the log verified by the project geologist or geotechnical engineer prior to selection of samples for index and classification tests. Types of index and classification tests which are frequently used for rock are listed in Table 7-4. Water content, unit weight, total porosity, and unconfined (uniaxial) compression tests will be performed on representative cores from each major lithological unit to characterize the range of properties. The RQD values (TM 5-818-1), as developed by Deere (1964), may be assigned to rock cores as a guide prior to testing. Additional tests for bulk specific gravity, apparent specific gravity, absorption, elastic constants, pulse velocity, and permeability, as well as a petrographic examination, may be dictated by the nature of the samples or by the project requirements (Das 1994). Samples of riprap and aggregate materials will be tested for durability and resistance to abrasion, and the specific gravity of the solids should be determined. Data from laboratory index tests and core quality conditions may be used for rock classification systems such as those developed by Bieniawski (1979) and Barton, Lien, and Lunde (1974).

Table 7-4
Laboratory Classification and Index Tests for Rock

Test	Test Method	Remarks
Unconfined (uniaxial) compression	RTH 111[1]	Primary index test for strength and deformability of intact rock
Specific gravity of solids	RTH 108[1]	Indirect indication of soundness of rock intended for use as riprap and drainage aggregate
Water content	RTH 106[1]	Indirect indication of porosity of rock or clay content of sedimentary rock
Pulse velocities and elastic constants	RTH 110[1]	Index of compressional wave velocity and ultrasonic elastic constants for correlation with in situ geophysical test results
Rebound number	RTH 105[1]	Index of relative hardness of intact rock cores
Permeability	RTH 114[1]	Intact rock (no joints or major defects)
Petrographic examination	RTH 102[1]	Performed on representative cores of each significant lithologic unit
Specific gravity and absorption	RTH 107[1]	Indirect indication of soundness and deformability
Unit weight and total porosity	RTH 109[1]	Indirect indication of weathering and soundness
Durability	TM 5-818-1, Federal Highway Administration (1978) Morgenstern and Eigenbrod (1974)	Index of weatherability of rock exposed in excavations and durability of rock for rockfill and riprap
Resistance to abrasion	RTH 115[1]	Los Angeles abrasion test; limited usefulness for evaluating weatherability of riprap
Point load testing	RTH 325[1]	May be used to predict other strength parameters with which it is correlated

[1] Rock Testing Handbook (USAEWES 1993).

Section III
Engineering Property Tests - Soils

7-5. Background

Reference should be made to EM 1110-2-1906 for current soil testing procedures and EM 1110-1-1904 for methods of settlement analysis.

a. Shear strength. Shear strength values are generally based on laboratory tests performed under three conditions of test specimen drainage. Tests corresponding to these drainage conditions are: unconsolidated-undrained Q tests in which the water content is kept constant during the test; consolidated-undrained R tests in which consolidation or swelling is allowed under initial stress conditions, but the water content is kept constant during application of shearing stresses; and consolidated-drained S tests in which full consolidation or swelling is permitted under the initial stress conditions and also for each increment of loading during shear. The appropriate Q, R, and S tests should be selected to reflect the various prototype loading cases and drainage conditions. Normally, strength tests will be made with the triaxial compression apparatus except S tests on fine-grained (relatively impervious) soils, which generally are tested with the direct shear apparatus because of time constraints using the triaxial apparatus. Where impervious soils contain significant quantities of gravel sizes, S tests should be performed on triaxial compression apparatus using large-diameter specimens.

(1) Q test. The shear strength resulting from a Q test corresponds to a constant water content condition, which means that a water content change is not permitted prior to or during shear. The Q test conditions approximate the shear strength for short-term conditions, e.g., the end-of-construction case. In cases where a foundation soil exists that is unsaturated but will become saturated during construction, it is advisable to saturate undisturbed specimens prior to axial loading in the Q test.

(2) R test. The shear strength resulting from an R test is obtained by inducing complete saturation in specimens using backpressure methods, consolidating these specimens under confining stresses that bracket estimated field conditions, and then shearing the specimens at constant water content. The R test applies to conditions in which impervious or semipervious soils that have been fully consolidated under one set of stresses are subject to a stress change without time for consolidation to take place.

(3) S test. The shear strength resulting from an S test is obtained by consolidating a sample under an initial confining stress and applying shear stresses slowly enough to permit excess pore water pressures to dissipate under each loading increment. Results of S tests are applicable to free-draining soils in which pore pressures do not develop. In cohesive soils, S tests are used for evaluating the shear strength of long-term conditions, e.g.,"normal operating" case. The R-bar test, a consolidated, undrained triaxial test in which pore pressures are measured during shear to determine the effective stress, has sometimes been used by a USACE district in lieu of the S test.

(4) Selection of design shear strengths. When selecting design shear strengths, the shape of the stress-strain curves for individual soil tests should be considered. Where undisturbed foundation soils and compacted soils do not show a significant drop in shear or deviator stress after peak stresses are reached, the design shear strength can be chosen as the peak shear stress in S direct shear tests, the peak deviator stress, or the deviator stress at 15 percent strain where the shear resistance increases with strain. For each soil layer, design shear strengths should be selected such that two-thirds of the test values exceed the assigned design values.

b. Permeability. To evaluate seepage conditions, reasonable estimates of permeability of pervious soils are required. Field pumping tests (TM 5-818-5) or correlations between a grain-sized parameter (such as D_{10}) and the coefficient of permeability, as in Figure 3-5, EM 1110-2-1913, are generally used for coarse-grained materials below the water table. The permeability of compacted cohesive soils for embankments and backfills and for soils modified in place is generally estimated from consolidation tests. Laboratory permeability tests are also being used more frequently for these materials.

c. Consolidation and swell. The parameters required to perform settlement and rebound analyses are obtained from consolidation tests on highly compressible clays or on compressible soils subjected to high stresses. Swell tests are also performed to identify, confirm, and quantify swelling ground conditions in tunneling. The sequence and magnitude of test loading should approximate the various prototype loading cases for which settlement and rebound analysis are to be performed. For expansive soils, the standard consolidation test or a modification of the test (Johnson, Sherman, and Al-Hussaini 1979) may be used to estimate both swell and settlement. Consolidometer swell tests tend to predict minimal levels of heave. Soil suction tests (Johnson, Sherman, and Al-Hussaini 1979) can be used to estimate swell. However, this test tends to overestimate heave compared with field observations. Gillott (1987) describes various tests to evaluate expansive soils.

Section IV
Engineering Property Tests - Rock

7-6. Background

Table 7-5 lists the laboratory tests frequently performed to determine the engineering properties of rock. These and other rock tests are presented in the Rock Testing Handbook (USAEWES 1993) and Nicholson (1983b).

Table 7-5
Laboratory Tests for Engineering Properties of Rock

Test	Reference	Remarks
Elastic moduli from uniaxial compression test	RTH 201[1]	Intact rock cores
Triaxial compressive strength	RTH 202[1]	Deformation and shear strength of core containing inclined joints
Direct shear strength	RTH 203[1]	Strength along planes of weakness (joints) or rock-concrete contact
Creep in compression	RTH 205[1]	Intact rock from foundation where time-dependent compression is an important factor in design
Thermal diffusivity	RTH 207[1]	Intact rock subjected to elevated temperatures such as adjacent to mass concrete where heat conductance is a factor

[1] Rock Testing Handbook (USAEWES 1993).

a. Unconfined uniaxial compression test. The unconfined uniaxial compression test is performed primarily to obtain the elastic modulus and unconfined compressive strength of a rock sample. Poisson's ratio can be determined if longitudinal and lateral strain measurements are made on the sample during the test. The value of Poisson's ratio is required for describing the deformation characteristics of a rock mass. Occasionally, design requirements dictate the need for testing of samples at different orientations to describe 3-D anisotropy. This test is economical, adequate for most foundation testing, and therefore a useful test for smaller projects. See USAEWES (1993), methods RTH 201-89 (ASTM D 3148-86 (ASTM 1996g)) and RTH 205-93 (ASTM D 4405-84 (ASTM 1996i)) for standard test methods.

b. Point load tests. Although the point load test is, strictly speaking, an index test for rock, it can be equated with the unconfined uniaxial compression strength. Its advantage lies in the ease with which the test can be conducted. The testing apparatus is portable so that it can be used in the field to test cores as they are retrieved from the ground. In this way, a statistically significant amount of data can be collected economically and with minimal effects from aging and handling of the cores. In addition, the tests can be run both perpendicular and parallel to the axis on the same piece of core, cut block, or irregular lump to provide a measure of the anisotropy of the rock strength. RTH 325-89 (USAEWES 1993) presents the suggested method for conducting point load strength tests.

c. Tensile strength. The tensile strength of rock is normally determined by the Brazilian method (RTH 113-93 (USAEWES 1993)) in which a piece of core is split along its axis. In some cases, direct pull tensile tests are conducted, but these samples are much harder to prepare. Results of the tensile strength tests provide input that can be used in the design of underground openings.

d. Unit weight. Determination of the unit weight of the various lithologies at a site is an important piece of engineering data. It is used for input into blast performance and muck handling, among other things. Since the unit weight is a nondestructive test, the sample can be subjected to additional tests after the unit weight is determined.

e. Direct shear test. Laboratory triaxial and direct shear tests on intact rock cores and intact rock cores containing recognizable thin, weak planes are performed to determine approximate values of cohesion, c (shear strength intercept) and φ (angle of internal friction) of a rock type. Detailed procedures for making the laboratory direct shear test are presented in RTH 203-80 (USAEWES 1993). The test is performed on core samples ranging from 6.5 to 20 cm (2 to 6 in.) in diameter. The samples are trimmed to fit into a shear box or machine and oriented so that the normally applied force is perpendicular to the feature being tested. Results of tests on intact samples will give upper-bound strength values while tests on smooth surfaces give lower-bound values. Repetition of the shearing process on a sample, or continuing, displacement to a point where shear strength becomes constant can ultimately establish the residual shear strength value. Where natural discontinuities control the rock mass shear strength, tests should be performed to determine the friction angle of the discontinuity asperities as well as the smooth discontinuity plane. The direct shear test is not suited to the development of exact stress-strain relationships because of the nonuniform distribution of shearing stresses and displacements within the test specimen. The bonding strength of a rock/concrete contact can be determined by this test. Values of cohesion and angles of internal friction are used to determine strength parameters of foundation rock. These values are the principal parameters in the analytical procedures to define the factor of safety for sliding stability and for some bearing capacity analysis. They are appropriate in analyses of the stability of rock slopes and of structures subjected to nonvertical external loading. For the test results to have valid application, test conditions must be as close as possible to actual field conditions. This includes selection of the normal loads to be used. Since the failure envelope is nonlinear at the lower load ranges, testing performed with too little or too much normal load will not adequately model actual conditions and will yield inappropriate values of c and φ. The application of these values is discussed in detail in Corps of Engineers guidance on gravity dam design (EM 1110-1-2908), and in Ziegler (1972) and Nicholson (1983a).

f. Triaxial shear test. The triaxial shear test can be made on intact, cylindrical rock samples. The test provides the data for determination of rock strength in an undrained state under 3-D loading. Data from the test can provide, by calculation, the strength and elastic properties of the rock samples at various confining pressures, the angle of internal friction (shearing resistance), the cohesion intercept, and the deformation modulus. Strength values are in terms of total stress as pore water pressure is not measured, and corrections should be made accordingly. The standard test method is presented as RTH 202-89 (USAEWES 1993) (ASTM D 2664-86 (ASTM 1996f)). A variation of this test using multistage triaxial loading (RTH 204-80 (USAEWES 1993)) is sometimes used to evaluate the strength of joints, seams, and bedding planes at various confining pressures.

g. Other testing. There are numerous other engineering properties (e.g., toughness, abrasiveness) of rock that are of interest in different applications and all have different testing procedures. The designer is advised to search the literature, including the RTH, to determine which testing is appropriate.

Section V
Engineering Property Tests - Shales and Moisture-Sensitive Rocks

7-7. Index Testing

Most moisture-sensitive geomaterials are sedimentary or metamorphic in origin. These include clays, clay shales, poorly to moderately cemented sandstones, marl, and anhydrite. Most commonly, moisture-sensitive rock and sediment contain clay minerals, particularly smectites, which have the capacity to hold large volumes of interstitial water. In some cases, the weathered product of a rock type may be the sensitive material in the overall rock mass and can be the result of chemical weathering (saprolite) or rock movement (fault gouge, mylonite). As these rock forms have soil-like characteristics, the index properties (Atterberg limits, moisture content, etc.) of these materials should be determined prior to more comprehensive testing. The results of the index testing usually indicate the engineering sensitivity of the rock forms and should be used as a guide to further testing. Special procedures that may be necessary for index testing can be found in EM 1110-2-1906.

a. Direct and triaxial shear tests. Most direct and triaxial shear tests conducted on hard, brittle rock samples are of the undrained type. For these particular types of materials, pore pressures do not play a dominant role, and strength values are in terms of total stress. However, as softer rock types are encountered, with correspondingly higher absorption values (e.g., greater than 5 percent), the role of pore pressure buildup during the rock shearing process becomes more important. The same condition is true for many clay shales and other similar weak and weathered rock materials. For moisture-sensitive rocks, soil property test procedures should be used if possible. Critical pore pressures that may substantially reduce the net rock strength can then be monitored throughout the entire testing cycle. Where hydraulic concrete structures are to be constructed on clay shales or shales, shear testing should be conducted to determine the strength of the shale/concrete interface.

b. Test data interpretation. Laboratory test data on shales and moisture-sensitive rocks should be interpreted with caution. The laboratory undrained strength of intact specimens is rarely representative of in-place field shear strengths. Frequently, shales, clay shales, and highly overconsolidated clays are reduced to their residual shear strength with minor displacements. The geotechnical explorations, laboratory testing, and review of field experiences must establish whether residual or higher shear strengths are appropriate for design purposes. Results of laboratory tests should be confirmed by analysis of the field behavior of the material from prior construction experience in the area, analysis of existing slopes or structures, and correlation with similar geologic formations at sites where the field performance is known. For a general engineering evaluation of the behavioral characteristics of shales, see Table 3-7, TM 5-818-1, Underwood (1967), and Townsend and Gilbert (1974); for physical properties of various shale formations, see Table 3-8, TM 5-818-1. Slope stability of shales can be analyzed using the PC-based, menu-driven program, UTEXAS3 (Edris 1993), ROCKPACK (Watts 1996) or the International Forum on Discontinuous Deformation Analysis Method (International Forum on Discontinuous Deformation Analysis and Simulations or Discontinuing Media 1996).

7-8. Swelling Properties

For many shales and moisture-sensitive rocks, swelling characteristics are a key consideration. Where used as fill, their physical properties can change significantly over time, and in response to the presence of water (Nelson and Miller 1992). In addition, swelling of in situ rock has caused heave in foundations, slope failures, distress in slope treatments such as shotcrete, and failure of tunnel linings (Olivier 1979). EM 1110-1-2908 has a thorough discussion of the testing procedures used to evaluate swelling potential of rock and soil. For the constant volume test, great care should be exercised in interpreting the results. The procedure currently in use calls for increasing applied load periodically during the test to return the specimen to its original dimensions. This load may exceed the actual swell pressure because it also must to overcome the elastic properties of the rock.