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Field Investigations for Foundations of Nuclear Power Facilities

U.S. Army Corps of Engineers

U.S. Nuclear Regulatory Commission
Office of Nuclear Regulatory Research
Washington, DC 20555-0001

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ABSTRACT

This document provides a technical basis for revision of the U.S. Nuclear Regulatory Commission Regulatory Guide 1.132, Site Investigations for Foundations of Nuclear Power Facilities, reflecting current and state-of-the-art techniques related to field site investigations. The report summarizes the processes of acquiring geological, geophysical, geotechnical, and other kinds of relevant information that may affect the construction or performance of a building or other engineered structure at selected sites. Guidance is presented for in situ site studies during the various stages of site characterization. Topics range from initial information gathering, literature review, and site reconnaissance investigations, to on-site testing and the collection and management of samples for laboratory testing. Specific laboratory tests and techniques for the engineering analyses of soils and specific requirements for liquefaction analysis are not addressed in this document but are covered in companion technical basis documents.

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PREFACE

The study covered by this report was performed by the U.S. Army Engineer Waterways Experiment Station (WES) for the U.S. Nuclear Regulatory Commission (NRC) under Inter-Agency Agreement RES-95-006 during the period June 1995 to January 1999. The study was directed by Mr. Robert Kornasiewicz, Office of Nuclear Regulatory Research, NRC.

The report was prepared by Ms. Nalini Torres, Dr. Joseph P. Koester, and Mr. José L. Llopis of the Earthquake Engineering and Geosciences Division (EEGD), Geotechnical Laboratory, WES. General supervision was provided by Dr. Mary E. Hynes, Chief, Earthquake Engineering and Geophysics Branch, and Dr. Lillian D. Wakeley, Acting Chief, EEGD.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Robin R. Cababa, EN.

OVERVIEW

This document provides a technical basis for revision of the U.S. Nuclear Regulatory Commission Regulatory Guide 1.132, "Site investigations for Foundations of Nuclear Power Facilities," reflecting current and state of the art techniques related to field site investigations. The report summarizes the processes of acquiring geological, geophysical, geotechnical, and other kinds of relevant information that may affect the construction or performance of a building or other engineered structure at selected sites. This includes the detection and assessment of potential foundation problems, geologic hazards and ground water conditions that may affect planned constructions. This document provides a description of commonly used methods for subsurface exploration and characterization, and of the types of subsurface materials for which they are generally applicable. However, foundation conditions at a specific site are unique, and thus the geologists/geotechnical engineers must develop an exploration program and methods that suit the conditions at their site.

This report is divided into sections covering procedures and requirements for a systematic field site characterization. Topics range from initial information gathering, literature review, and site reconnaissance investigations, to on-site testing and collection and management of samples for laboratory testing. The document is modeled from the Geotechnical Guide for Site Characterization produced by the U.S. Army Engineer Waterways Experiment Station (Engineer Manual (EM) 1110-1-1804) to assist those responsible for engineering projects of the Corps of Engineers. Specific laboratory tests and techniques for the engineering analyses of soils and specific requirements for liquefaction analysis are not addressed in this document but are covered in companion technical basis documents.

This report organizes the various activities that may be required for a site characterization in a logical sequence that will support a complete study. Section contents are summarized in the following paragraphs.

The **Geologic Site Assessment** section describes an office study, a walk-over study, and the development of a conceptual model and geotechnical data base as the first steps toward a site selection investigation. The office study consists of an intensive literature review to assure that all the previously available information of the study sites will be obtained and reviewed. This segment includes a section on general map studies. Recommendations are given for this process, as well as sources of published and unpublished information. The walk-over study is basically the "reality check" of all the remote and documentary information collected previously in the office study. The walk-over includes the review of the office data and on-site reconnaissance. A conceptual model and geotechnical data base are described; these should be developed from information acquired during the office and walk-over studies. The process of creating the conceptual model and data base starts with the information gathered in the office study, continues with the corroboration and collection of necessary information in the walk-over study, and culminates with the analysis of all the verified and customized information. The appropriate sampling requirements for a site characterization study are discussed, including such details as sample depth, layout, and spacing. Subsurface investigation methods for typical geologic conditions are included, as well as characterization needs and detailed explorations for unusual site conditions. The model and geotechnical database are continually updated as more information is acquired and performance data become available.

The **Groundwater Investigations** section covers the requirements of a site characterization program related to assessment of subsurface water conditions. Groundwater studies include: observations and measurements of flows from springs and of water levels in existing wells, boreholes, observation or monitoring wells, and piezometers. This information is used with site geologic information to determine piezometric surface elevations and profiles, fluctuations in piezometric surface 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. Descriptions of the information that can be provided by different types of piezometers and their specific installation requirements as well as other instruments and techniques and their advantages and disadvantages, are included in this section. Guidance and references are also included for instrument calibration, maintenance, and verification necessary to assure proper performance of the equipment and that reliable results are obtained.

The **Boring and Exploratory Excavations** section describes the many drilling techniques and equipment available for site investigation. Rotary and percussion drilling, power augering, hand driven techniques, and other options are described.

Sampling Methods Techniques for both disturbed and undisturbed sampling are discussed. Alternative sampling techniques are described and referenced for undisturbed sampling of hard-to-sample materials. Site-specific in situ sampling recommendations are presented according to soil type (e.g., cohesive, non-cohesive, fine-grained, coarse grained).

The **Geophysical Investigations** section introduces and tabulates various engineering geophysics techniques applied to interrogate large areas and amounts of the subsurface for determination of engineering properties and the spatial distribution of geologic materials at potential foundation sites. Advantages and disadvantages of each technique are presented.

The **In Situ Physical Testing** section includes discussions of the methods used in current practice for in situ testing of subsurface soils and rock. This section also describes in situ tests for specific site conditions, including penetration tests, in situ stress, compressibility tests, and deformation characteristics tests.

The **Handling, Field Storage, and Transporting of Samples** section offers guidance and references for procedures for handling, transporting, and preserving disturbed and undisturbed samples.

1 GEOLOGIC SITE ASSESSMENT

1.1 Development of a Conceptual Model

Nuclear power facilities require extraordinary assurance of public safety. Part of assuring this safety ensues from building nuclear facilities where geologic conditions are known and geologic hazards are minimal. Knowledge of the geologic setting that one gains through site investigation is a prerequisite to planning an engineering assessment of the site. That is, on-site and geotechnical engineering investigations can proceed in a cost-effective manner only after development of a conceptual model of the geologic setting. Most importantly, the exploration program and methods must be tailored to address the key foundation and geotechnical issues identified from the geologic conceptual model.

A geologic conceptual model begins as the 3-dimensional mental picture or best guess of geologic conditions of the subsurface and how the site under consideration fits in its regional geologic setting. This mental picture is revealed by a study of all available geologic information, and refined through on-site reconnaissance and sampling. Details of the concept are changed with additional data, resulting in a sound basis for in situ measurements of materials properties and for creation of a sampling matrix for subsequent laboratory testing of engineering properties as needed for appropriate design and construction. The purpose of a geologic site assessment and subsequent activities of site investigation is to create this conceptual model and populate the model with useful geotechnical data.

Several other types of information related to conditions in the shallow subsurface must be known about a site before it becomes a candidate for site investigation. The earthquake hazard potential of any site under consideration must be known. Regional information about earthquake hazards is available from published resources, and on-line as presented in later sections. Also, the likelihood of encountering cultural resources must by considered, to assure compliance with the Archeological Resources Protection Act of 1979 and the Native American Graves Protection and Repatriation Act of 1990.

Aspects of the Clean Water Act (33 U.S.C. 1344) also must be considered. Construction activities that involve placement of fill into wetlands are regulated at the national level. State and local wetland protection laws also may apply. Identification and delineation of wetlands involves sampling of soils, vegetation, and hydrology according to guidance given in the 1987 Corps of Engineers Wetlands Delineation Manual. Information about applications for Section 404 permits for modifying wetlands can be obtained from District Offices of the Corps. Additional information about District Regulatory Offices is available at:

http://www.usace.army.mil/inet/functions/cw/cecwo/reg/

Compiled and properly interpreted regional geologic data from the office study, coupled and corroborated with information obtained during the walk-over study, will provide the information necessary to identify suitable sites and to determine the scope of site investigations. A site model should involve the following:

- Tentative models should be developed for geologic conditions at the site.
- Establishment of micro-seismic monitoring network in the study area.
- Regional geologic conditions should be identified and incorporated into a regional geologic map.
- Preliminary assessment should be made of regional seismicity.
- Tentative sources of construction materials should be located.

1.2 Office Study

The office study consists of an extensive literature review with the goal of obtaining and studying the significant and previously available information on the site. 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, other federal, state and local agencies, and private industry. This information must be evaluated to determine its validity for use throughout the project's development. Deficiencies and problems must be identified early so that studies for obtaining needed information can be planned to assure economy of time and money. Some of the basic aspects that should be covered in an office study include geologic conditions, meteorological or climatic conditions, existing constructions, previous land uses, previous mining activity, and geomorphological changes. Especially in the case of larger projects, data are most effectively compiled and analyzed in a Geographical Information System (GIS) format. Table 1 summarizes the sources of topographic, geologic, and special maps and geologic reports. Table 2 outlines the types of geologic features or conditions that should be identified from the various sources of information.

1.2.1 Existing Literature

Understanding the geologic and hydrologic conditions is the goal of an early site characterization program. The remainder of the site characterization program will involve the collection and analysis of site data to improve this understanding and determine engineering properties of the geologic materials. Information on topography, geology and geologic hazards, surface and groundwater hydrology, seismology, and soil and rock properties is reviewed to determine the adequacy of available data. Additional information such as groundwater, other subsurface information, and seismicity data will be needed at specific sites. These data are critical to long-term studies, requiring advanced planning and early action.

General site topographic and geological characteristics should be obtained from existing topographic, geologic and geophysical maps. Available geologic and engineering geology maps, soil survey maps and records, geological publications, regional field trip guides, and journals should be reviewed for pertinent information. Geotechnical problems and conditions described in previous studies, journals, newspapers, and ground investigation reports should be identified.

Climatic or meteorological conditions of the area can be obtained from existing meteorological records. Ground water conditions can be assessed from topographic maps, aerial photographs, well records, water supply and previous site investigation reports.

Geomorphological changes through time can be observed as well as prior construction activities in the area. Knowledge of this information allows for a better understanding of the natural geologic-hydrologic system of the region and how it can be modified by the project. This information will allow better judgment for site selection and general project design and management.

Most states regulate well installation and operation and maintain water well databases that extend back many years. Wells may be municipal, industrial, domestic, or may have been drilled for exploration and/or production of petroleum and natural gas. The information that is commonly available includes date of installation, screened interval, installer's name or company, depth, location, owner, and abandonment data. Lithologic and geophysical logs may also be available, and in some cases, production and water quality information.

Table 1 Sources of Geologic Information (EM 1110-1-1804, Department of the Army, 1984)

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 series 1:20,000 (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, one for certain areas at 1:100,000 and 1:24,000 scales. Digital line graphs are available for some areas at 1:24,000 and 1:65,000 for: - Hydrograph - Transportation - U.S. Publication Survey - Boundaries - Hypsography	Orthophotoquad monocolor maps also produced in 7.5-minute and 15-minute series. New index of maps for each state started in 1976. Status of current mapping from USGS regional offices and in monthly USGS bulletin, "New publications of the U.S. Geological Survey"
USGS	Geology maps and reports	1:24,000 (1:20,000 Puerto Rico), 1:62,500, 1:100,00, 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 (conterminous U.S., 1974)	New index of geologic maps for each state started 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 groundwater	Miscellaneous Investigation Series and Miscellaneous Field Studies Series, maps and reports, not well cataloged; many included as open file
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 groundwater contours and location of wells

(Continued)

Table 1 (Continued)

Agency	Type of Information	Description	Remarks
USGS	Earthquake hazard	Seismic maps of each state (started 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).
USGS	Mineral resources	Bedrock and surface geologic mapping; engineering geologic investigations; map of power generating plants of U.S. (location of built, under construction, planned, and type); 7.5-minute 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" North American, Hawaiian Islands, and Guam	Published until 1972
Geological Society of America	Bibliography	"Bibliography and Index of Geology Exclusive of North America"	1934-1968
		"Bibliography and Index of Geology"	1969 to present, 12 monthly issues plus yearly cumulative index
NOAA	Earthquake hazards	National Geophysical Data Center in Colorado contains extensive earthquake hazard information	
NASA	Remote sensing data	Landsat, Skylab imagery	See Table 4-2 of EP 70-1-1 for detailed information
NOAA	Remote sensing data		
EOSAT	Remote sensing data		
USFWS	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	Series of 19 reports, 1973 to present

(Continued)

Table 1 (Continued)

Agency	Type of Information	Description	Remarks
International Union of Geological Societies	Worldwide mapping	Commission for the Geological Map of the World publishes periodic reports on worldwide mapping in "Geological Newsletter"	
NRCS	Soil survey reports	1:15,840 or 1:20,000 maps of soil information on photomosaic background for each country. 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, 1:250,000, 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.
FEMA	Earthquake hazard	NEHRP "Recommended provisions for Seismic Regulations for New Buildings and Older Structures," 1997, includes seismic maps.	
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, groundwater studies	List of maps and reports published annually, unpublished information by direct coordination with state geologist
National Imagery and Mapping Agency (NIMA)	Topographic Maps	Standard scales of 1:12,500, 1:50,000, 1:250,000 and 1:1,000,000 foreign and worldwide coverage including digital and photomaps	Index of available maps from NIMA
American Association of Petroleum Geologists	Geological highway map series	Scale approximately 1 in. equal to 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-minute 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

(Continued)

Table 1 (Concluded)

Agency	Type of Information	Description	Remarks
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 inter-library loan or from USAE Waterways Experiment Station for many dams
Agricultural Stabilization and Conservation Services Aerial Photography Field Office	Aerial photograph	The APFO offers aerial photographs across the U.S. typically a series of photographs taken at different times, as available for a given state	Information is available at 801-975-3503
USGS Earth Resources Observation Systems (EROS) Data 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 800 USAMAPS
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

Table 2 Special Geologic Features and Conditions Considered in Office Studies and Field Observations (EM 1110-1-1804, Department of the Army, 1984)

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
Landslides	Stability of natural and excavated slopes	Presence or age in project area or at construction sites should be determined	Estimate areal extent (length and width) and height of slope	Are landslides found off site in geologic formations of same type that will be affected by project construction?
		Compute shear strength at failure. Do failure strengths decrease with age of slopes-- especially for clays and clay shales?	Estimate ground slope before and after slide (may correspond to residual angle of friction)	What are probable previous and present groundwater levels?
			Check highway and railway cuts and deep excavations, quarries and steep slopes	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	Verify presence at site, if possible, from surface evidence; check potential fault traces located from aerial imagery	Are lineaments or possible fault traces apparent from regional aerial imagery?
		Examine existing boring logs for evidence of faulting from offset of strata	Make field check of structures, cellars, chimneys, roads, fences, pipelines, known faults, caves, inclination of trees, offset in fence lines	
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 groundwater levels outside valley	Examine wells and piezometers in valleys to determine if levels are lower than normal groundwater regime (indicates valley rebound not complete)	

(Continued)

Table 2 (Continued)

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
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?
Anhydrites or gypsum layers	Anhydrites in foundations beneath major structures may hydrate and cause expansion, upward thrust and buckling	Determine possible existence from available geologic information and delineate possible outcrop locations	Look for surface evidence of uplift; seek local information on existing structures	Are uplifts caused by possible hydrite expansion or "explosion"?
	Gypsum may cause settlement, subsidence, collapse or piping. Solution during life of structure may be damaging		Check area carefully for caves or other evidence of solution features	
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	Determines need for total or partial channel slope protection	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?

(Continued)

Table 2 (Continued)

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
Internal erosion	Affects stability of foundations and dam abutments. 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 groundwater 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	Determines need for removal of shallow foundation materials that would collapse upon wetting	Determines 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?
Locally lowered groundwater	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 groundwater 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 groundwater levels)	May indicate effective stresses are still increasing and may cause future slope instability in valley sites	Compare normal groundwater levels with piezometric levels if data is available		Is a possible cause the past reduction in vertical stresses (e.g. deep glacial valley or canal excavations such as Panama Canal in clay shales where pore water pressures were reduced by stress relief)?

(Continued)

Table 2 (Continued)

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
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 groundwater 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 flow 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 groundwater	Determine areas of possible varved clay deposits associated with prehis toric lakes. Determine settlement behavior of structures in the area	Check natural slopes and cuts for varved clays; check settlement behavior of structures	
Dispersive clays	A major factor in selecting soils for embankment dams and levees	Check with Soil Conservation Service and other agencies regarding behav ior 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	

(Continued)

Table 2 (Concluded)

Geologic Feature or Condition	Influence on Project	Office Studies	Field Observations	Questions to Answer
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

1. Geologic Site Assessment

All existing and previous construction activities should be investigated. Services such as water and gas, electric, and telephone lines in the area should be documented from construction (as-built) drawings, topographical maps, plans held by utilities, and mining records. The locations of power lines, pipelines, access routes, and ground conditions that could restrict the location of or access to borings should be noted. Local construction practices and the condition of existing structures and roads should be observed and potential problems also noted. State DOT's may have detailed record of highway and bridge boring programs in the area.

Previous land uses should also be documented. This information can be obtained from topographical maps and geological maps, air photographs, airborne remote sensing, archeological records, state historical offices, libraries, real estate agencies, and mining records. Historical and archaeological sites that may include cultural resources should be identified and noted. Their presence may influence construction practices or site location.

Current and historical records of human activities on the site can provide information such as withdrawal or addition fluids from or to the subsurface and the extraction of minerals. The location of abandoned mine workings such as adits, benches, shafts, and tailings embankments should be noted.

Remote sensing surveys can provide additional information in many aspects, including interpretation of large scale and regional geologic structures, regional lineaments, drainage patterns, rock types, soil characteristics, erosion features, availability of construction materials, vegetation, surface water and changes in land use. Geologic features can be enhanced by manipulating remote sensing imagery. Detailed topographic maps can be produced from aerial photographs. Any problem boundaries or areas adjacent to the project that will be affected or affect the project in a negative way need to be identified so they can be modified, eliminated, or protected.

1.2.2 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 an understanding of the regional geology prior to the field reconnaissance and exploration work. The types of available maps and their description are presented in Table 1. Their uses are described in this section.

1.2.2.1 Topographic Maps. Topographic and geologic maps can typically be found in the USGS 7-1/2 minute series (1:24,000 or 1:25,000 scales) with contours and elevations in feet or meters respectively. Topographic maps are also available as 1:50,000, 1:100,000 and 1:250,000 series. The 15-minute series (1:62,500 scale) is not being updated by the USGS. Many of the 1:24,000 are available in digital raster form as DRG's, or digital raster graphs, and as DEM's, or digital elevation maps from USGS or commercial distribution. The 1:250,500 maps are available as DEM's and can be downloaded from the USGS web page.

Availability information can be found at http://edcwww.cr.usgs.gov/nsdi/gendem.htm for Digital Elevation Models (DEM) and Digital Line Graphs (DLG), at ftp://www-nmd.usgs.gov/pub/doi_high_priority/html/doq_stat.html for Digital Orthophoto Quadrangles (DOQ), and at http://mcmcweb.er.usgs.gov/drg/avail.html for Digital Raster Graphics (DRG).

Topographic maps provide information about land forms, drainage patterns, slopes, locations of prominent springs and wet areas, quarries, man-made cuts (for field observation of geologic features and other man-made features), and mines. If older topographic maps are available, especially in mining regions, abandoned shafts, filled surface pits, and other features can be located by comparison with current maps.

Optimum use of topographic maps involves the examination of both large and small-scale maps. Certain features, such as large geologic structures, may be apparent only on small-scale maps. Conversely, the interpretation of locally 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 more specific.

Certain engineering geology information can be inferred from topographic maps by proper interpretation of land forms 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. Not all geologic features are equally apparent on topographic maps, and professional geologic knowledge is 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 engineering significance that may be obtained or inferred from aerial photographs and topographic maps includes physiography, general soil and rock types, bedrock structure, and geomorphic history.

1.2.2.2 Geologic Maps. Surficial and bedrock geologic maps can be used to identify geologic formations, formation contacts, gross structure, fault locations, and approximate depths to bedrock. 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 (see Dodd, Fuller, Clarke, 1989, for instructions to access USGS publications). State Geologic surveys, local universities, state and federal DOT, 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, estimates on depth to bedrock, and characteristics of surface and near-surface soil and rock.

1.2.2.3 Mineral Resource Maps. Mineral resource maps produced by the USGS and state geological survey are important sources of geologic information. The USGS coal resources evaluation program, for example, 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 geological survey 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).

1.2.2.4 Hydrologic and Hydrogeologic Maps. Maps showing hydrologic and hydrogeologic information provide a valuable source of data on surface drainage, well locations, ground-water quality, groundwater level contours, seepage patterns, and aquifer locations and characteristics. The USGS (Dodd, Fuller, and Clarke, 1989, is an access guide for USGS information), state geological surveys, local universities, and geotechnical and environmental firms may provide this information.

1.2.2.5 Seismic Maps. Krinitzsky (1995) discussed the distribution of seismic source areas in the U.S. and provided a potential magnitude of earthquakes within each zone. Several World Wide Web sites are available as resources for seismic activity and history, including, but not limited to:

http://wwwneic.cr.usgs.gov/
http://www.geophys.washington.edu/seismosurfing.html
http://www.usgs.gov/network/science/earth/earthquake.html

Peak acceleration and spectral acceleration (percent g) maps for various periods of ground motion have been generated by the USGS and others to assess seismic hazard. The Building Seismic Safety Council (BSSC) has published updated seismic hazard potential maps in the form of spectral values for periods 0.1, 0.3 and 1.0 seconds. Ordering information for BSSC and national Institute of Building Sciences publications may be found on the World Wide Web at http://nibs.org/cato7.htm

1.3 Walk-over Study

The walk-over study provides a field verification of all the remote and documentary information collected during office studies. A multidisciplinary team should be formed at this stage to identify problem areas and design site-specific testing programs. The team leader is responsible for assuring that all pertinent information required is obtained to the desired level of detail. In general, the walk-over study should proceed from a regional approach initially to a site specific approach.

1.3.1 Geologic Field Reconnaissance

The walk-over study should include surface reconnaissance of the immediate site and surrounding areas. Detailed topographic, hydrologic, hydrographic, and surface geologic feature information should also be corroborated, corrected, and mapped. The identification of surface geologic features should include detailed onsite mapping of soils, and all surface water features present at the study site. Site accessibility, ground condition, and right of entry problems that could affect the exploration work must be identified as well as construction practices and existing structural distress that could indicate problem soil or rock conditions. Approximate soil profiles, representative samples of the principal strata, rock transects, or limited geologic cross sections by indirect methods of exploration should also be produced at this stage. The study of areas remote to the site may be necessary to have a more complete and reliable evaluation of the site. On-site observations and a fly-over by plane or helicopter would be an advisable approach for regional site mapping. The corroboration of previous geological interpretation of aerial photographs and other remote sensing imagery will provide a better visualization of the area. This approach can identify the need for new mapping or aerial photographic coverage. Such coverage should be coordinated with planners early in the study process to insure sufficient and timely coverage.

An extensive photographic and video record taken by personnel with a background in geology or geotechnical engineering can serve as a reasonable proxy for cursory, preliminary investigations; however, a site visit by the site characterization team is essential. Potential environmental hazards such as former landfills, surface impoundments, mining activity, industrial sites, signs of underground storage tanks, distressed vegetation or dust should be recorded and assessed for Hazardous, Toxic, or Radioactive Waste (HTRW) potential.

Figure 1 shows a diagram for the characterization of regional geology. This information will be the basis for further, more site-specific investigations. Field reconnaissance should be conducted at all sites that are potential facility locations and should include examination of important geologic features and potential problem areas identified during office studies. Preliminary geologic, seismic, hydrologic, and economic studies should be used to indicate the most favorable sites before preliminary subsurface investigations are begun. Field observations have special value in planning subsequent investigations because adverse subsurface conditions often can be anticipated from surface evidence and regional geology. Proper coordination and timing of these studies, and incorporation into a GIS data base, can reduce costs and increase confidence in the results.

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 adversely dipping beds that would affect the stability of natural or excavated slopes should be included. Indications of slope instability such as scarps, toe bulges, leaning trees, etc. should also be recorded. The locations of sources of construction materials, such as sand and gravel deposits, borrow areas for soils, and active or abandoned quarries are important. Observable hydrologic features include surface drainage flow, springs and seeps in relation to formation members, and marshy or thick vegetation areas indicating high groundwater tables.

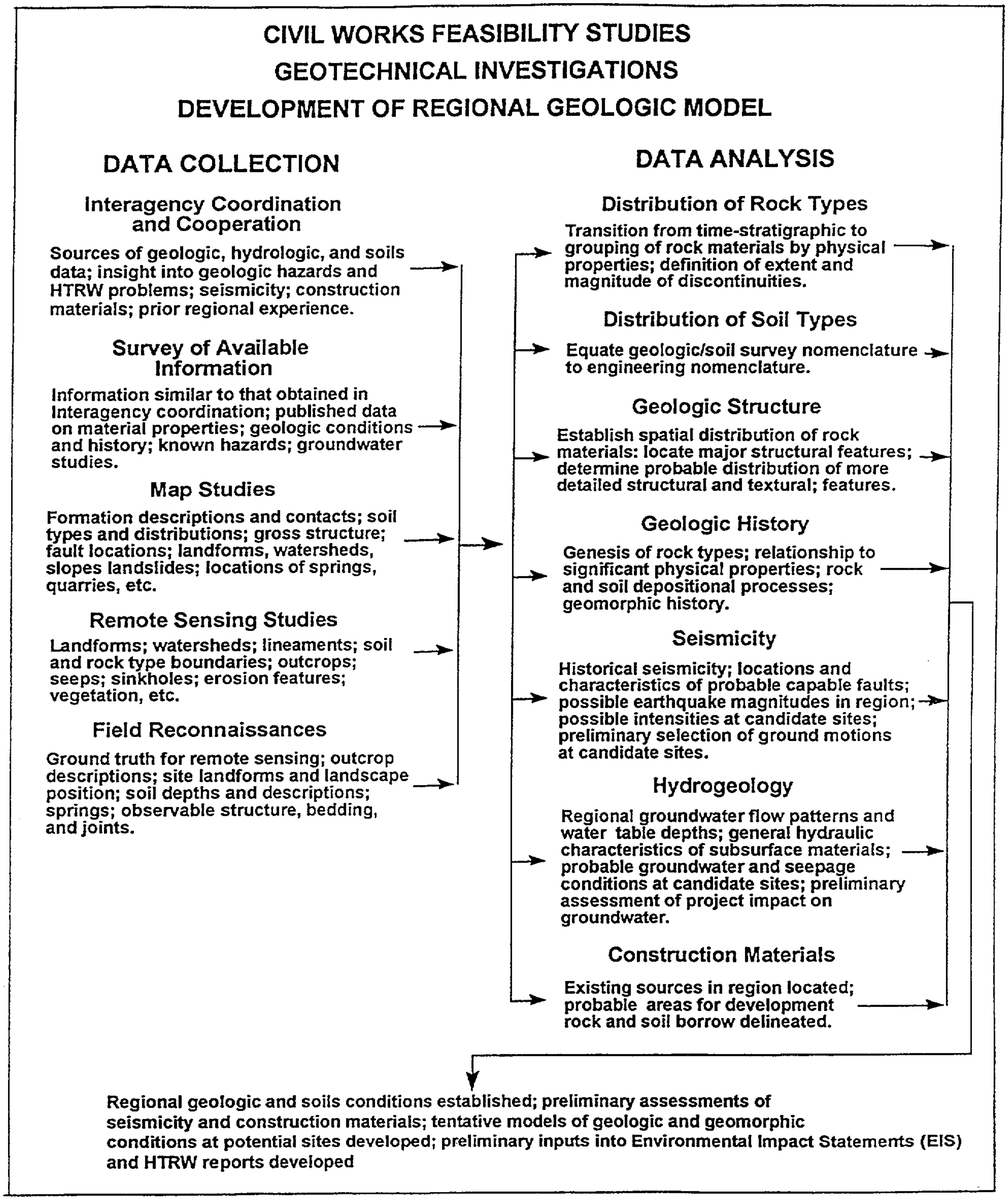
1.3.2 Geologic Field Mapping

1.3.2.1 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 project and on the regional geology. Map scales of 1:250,000, 1:62,500 or 1:50,000 may be appropriate for small scale regional mapping and 1:24,000, 1:25,000, or larger scales are appropriate for more detailed and specific mapping. The area to be mapped should include the project site(s) as well as the surrounding area that could influence or could be affected by the project.

Geologic and environmental features within the facilities' footprint and adjacent areas that should be studied and mapped include the following:

- Faults, joints, stratigraphy, and other significant geologic features.
- Karst topography or other features that indicate potential for ground collapse.
- Water well levels, springs, surface water, water-sensitive vegetation, or other evidence of the groundwater regime.
- Presence of soluble or swelling rocks such as gypsum, anhydrite, or expansive clays
- Potential or old landslide areas.
- Archeological sites.
- Valuable mineral resources.
- Mine shafts, tunnels, and gas and oil wells.
- Potential borrow and quarry areas and sources of construction materials.
- Shoreline erosion potential.
- Landfills, dumps, underground storage tanks, surface impoundments, and other potential environmental hazards.

Large-scale and detailed geologic maps should be prepared for specific sites of interest within the project area and should include proposed structure areas, borrow, and quarry sites. Investigation of the geologic features of overburden and bedrock materials is essential in site mapping and subsequent explorations.



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; groundwater studies.

Map Studies

Formation descriptions and contacts; soil types and distributions; gross structure; fault locations; landforms, watersheds, slopes 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 groundwater flow patterns and water table depths; general hydraulic characteristics of subsurface materials; probable groundwater 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 1. Diagram for the development of regional geology

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 bedrock, and the geotechnical engineer or engineering geologist should determine the engineering properties of the site foundation, potential construction materials, the manner of application to design, and the adaptation of proposed structures to foundation conditions.

1.3.2.2 Building Construction Sites. A preliminary map of geologic structure, lithology, and stratigraphy should be prepared prior to making any subsurface borings. For each proposed boring, an estimate should be made of the subsurface conditions anticipated, such as depths to critical contacts and to the water table. 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 for site characterization.

1.3.2.3 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 regional mapping. It is sometimes necessary to expand the field area in order 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 utilize materials closer to the project but lower in quality should be tentatively formulated and evaluated. A complete borrow and quarry source map should include all soil and rock types encountered and adequate descriptions of surficial weathering, hardness, and joint spacings.

Processed rock products are usually most economically acquired from commercial sources. Test results are often available on these sources through state or federal offices. Soil and/or rock sources should be selected based upon sampling and laboratory analysis. A geologic map may be used to determine material quantities available based on estimates of the depths and thicknesses of various deposits. Geologic maps can also be used to make a preliminary layout of haul and access roads in order to estimate haul distances. A GIS or similar system could be used to evaluate the quality and quantity of available quarry material, cost of excavation, and optimal transport routes.

1.3.3 Construction Mapping

Traditionally, these are geologic maps with details on structural, lithologic, and water-oriented features. They can represent planned structure foundations, cut slopes, and geologic features in tunnels or large chambers. These maps should be prepared to 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 excavation has been cleaned to grade and just prior to the placement of concrete or backfill, to permit the observation and recording of geologic details in the foundation. Photographic and videographic record should be made during foundation mapping.

The person in charge of foundation mapping for critical structures 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 clean-up. 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.

Appendix A 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 exposed surfaces and to allow a reasonable length of time for mapping to be carried out. Technical procedures for mapping tunnels are outlined in Appendix B and can be modified for large chambers.

1.3.4 Remote Sensing Methods

Conventional aerial photographs and various types of satellite and other aerial imagery can be used effectively for small-scale regional interpretation of geologic structure, analyses of regional lineaments, drainage patterns, rock types, soil characteristics, erosion features, and availability of construction materials. Geologic hazards, such as faults, fracture patterns, subsidence, and sink holes or slump topography, can be recognized from stereoscopic examinations of air photo pairs and imagery interpretation. Detailed topographic maps can be generated from aerial stereophotography that has 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 such as Landsat, Sky Lab, and French SPOT images are useful for regional studies.

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 (DA, 1995).

Spatial components typically used to prepare a GIS reference 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 or digital elevation models (DEM's) are generated through photogrammetric methods and can be displayed using a GIS. A DEM may be used to interpolate and plot a topographic contour map, generate 2-dimensional (contour or shaded relief) or 3-dimensional (perspective) views of the modeled surface, determine earthwork quantities, and produce cross-sections oriented along arbitrary alignments.

Geotechnical parameters resulting from surface and subsurface explorations can be referenced to a Digital Terrain Model (DTM), resulting in a spatial database capable of producing geologic cross-sections, and 2- and 3-dimensional 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 U.S.

A GIS can be used to streamline and enhance regional or site-specific geotechnical investigations by: (1) Verifying which information is currently available and what new data must be obtained or generated to fulfill requirements for the desired level of study. (2) Sorting and combining layers of information to evaluate the commonality of critical parameters and compatibility of proposed alternatives/sites, and (3) Assigning quantitative values and relational aspects of data combinations and classifications. In this respect, a geotechnically augmented GIS database can be used to quantify reliability and uncertainty for specific design applications and assumptions. Burrough (1986), Intergraph (1993), and Kilgore, Krolac, and Mistichelli (1993) provide further explanations of GIS uses and capabilities.

2 GROUNDWATER INVESTIGATIONS

2.1 General

Groundwater investigations produce information that may be vital to selection of foundation sites and should be required in site characterization programs for large, complex facilities. The objectives and level of detail required should be determined early in the study to identify the data requirements and the resources needed to produce the information. Guidance for development of a conceptual model of a field site and selection of an analytical test method for determination of hydraulic properties is provided in ASTM D 4043-96, 1996, "Guide for Selection of Aquifer Field Test and Analytical Procedures in Determination of Hydraulic Properties by Well Techniques."

For engineering foundation projects, records of groundwater levels, pressures, seasonal variations, and the relationship of groundwater with surface waters or tides should be evaluated. Borings, wells, and piezometers must be correctly located and accurately surveyed for reliable groundwater investigations. Table 3 describes instruments to measure groundwater pressure (Dunnicliff, 1988). The kinds of information that can be obtained from groundwater studies include: position and thickness of aquifers and confining beds (aquicludes), transmissivity, storage coefficient, location and nature of aquifer boundaries, and hydraulic characteristics of aquicludes. Piezometers, when correctly chosen and installed, can monitor groundwater flow patterns, directions, and velocities. Instrument calibration, maintenance, and recalibration are necessary to assure proper performance of the systems and reliable results.

The scope of groundwater studies is determined by the size and nature of the proposed project. The type of studies 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. ASTM D 5474-93, 1993, "Guide for Selection of Data Elements for Groundwater Investigations," explains the logic of why individual and combinations of data elements are selected to meet the requirements of the study (with examples of specific groundwater investigations). Groundwater studies include observations and measurements of flows from springs and of water levels in existing wells, boreholes, selected observation wells, and piezometers. This information is used with site and regional geologic information to determine water table 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. Results from groundwater 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 groundwater resources due to

Table 3 Instruments for Measuring Groundwater Pressure (Dunnicliff, 1988, reprinted by permission of John Wiley & Sons, Inc.)

Instrument Type	Advantages	Limitations[a]
Observation well	Can be installed by drillers without participation of geotechnical personnel.	Provides undesirable vertical connection between strata and is therefore often misleading; should rarely be used.
Open standpipe piezometer	Reliable. Long successful performance record. Self-de-airing if inside diameter of standpipe is adequate. Integrity of seal can be checked after installation. Can be converted to diaphragm piezometer. Can be used for sampling groundwater. Can be used to measure permeability.	Long time lag. Subject to damage by construction equipment and by vertical compression of soil around standpipe. Extension of standpipe through embankment fill interrupts construction and causes inferior compaction. Porous filter can plug owing to repeated water inflow and outflow. Push-in versions subject to several potential errors.
Twin-tube hydraulic piezometer	Inaccessible components have no moving parts. Reliable. Long successful performance record. When installed in fill, integrity can be checked after installation. Piezometer cavity can be flushed. Can be used to measure permeability.	Application generally limited to long-term monitoring of pore water pressure in embankment dams. Elaborate terminal arrangements needed. Tubing must not be significantly above minimum piezometric elevation. periodic flushing may be required. Attention to many details is necessary.
Pneumatic piezometer	Short time lag. Calibrated part of system accessible. Minimum interference to construction: level of tubes and readout independent of level of tip. No freezing problems.	Attention must be paid to many details when making selection. Push-in versions subject to several potential errors.
Vibrating wire piezometer	Easy to read. Short time lag. Minimum interference to construction: level of lead wires and readout independent of level of tip. Lead wire effects minimal. Can be used to read negative pore water pressures. no freezing problems.	Special manufacturing techniques required to minimize zero drift. Need for lightning protection should be evaluated. push-in version subject to several potential errors.
Unbonded electrical resistance piezometer	Easy to read. Short time lag. Minimum interference to construction: level of lead wires and readout independent of level of tip. Can be used to read negative pore water pressures. No freezing problems. Provides temperature measurement. Some types suitable for dynamic measurements.	Low electrical output. Lead wire effects. Errors caused by moisture and electrical connections are possible. Need for lightning protection should be evaluated.

(Continued)

Table 3 (Concluded)

Instrument Type	Advantages	Limitations[a]
Bonded electrical resistance piezometer	Easy to read. Short time lag. Minimum interference to construction: level of lead wires and readout independent of level of tip Suitable for dynamic measurements. Can be used to read negative pore water pressures. No freezing problems.	Low electrical output. Lead wire effects. Errors caused by moisture, temperature, and electrical connections are possible. Long-term stability uncertain. Need for lightning protection should be evaluated. Push-in version subject to several potential errors.
Multipoint piezometer, with packers	Provides detailed pressure-depth measurements. Can be installed in horizontal or upward boreholes. Other advantages depend on type of piezometer: see above in table.	Limited number of measurement points. Other limitations depend on type of piezometer: see above in table.
Multipoint piezometer, surrounded with grout	Provides detailed pressure-depth measurements. Simple installation procedure. Other advantages depend on type of piezometer: see above in table.	Limited number of measurement points. Applicable only in uniform clay of known properties. Difficult to ensure in-place grout of known properties. Other limitations depend on type of piezometer: see above in table.
Multipoint push-in piezometer	Provides detailed pressure-depth measurements. Simple installation procedure. Other advantages depend on type of piezometer: see above in table.	Limited number of measurement points. Subject to several potential errors. Other limitations depend on type of piezometer: see above in table.
Multipoint piezometer, with movable probe	Provides detailed pressure-depth measurements. Unlimited number of measurement points. Allows determination of permeability. Calibrated part of system accessible. Great depth capability. Westbay Instruments system can be used for sampling groundwater and can be combined with inclinometer casing.	Complex installation procedure. Periodic manual readings only.

[a] Diaphragm piezometer readings indicate the head above the piezometer, and the elevation of the piezometer must be measured or estimated if piezometric elevation is required. All diaphragm piezometers, except those provided with a vent to the atmosphere, are sensitive to barometric pressure changes.

project operation, show potential for interference to aquifers by the construction of a project, and determine the chemical and biological quality of groundwater and its relationship to project requirements. Investigation and continued monitoring of groundwater fluctuations are key issues for retention ponds and dam safety. ASTM D 5447-93, 1993, "Guide for Application of Ground Water Flow Model to a Site Specific Problem," covers the application and documentation of a groundwater flow model to a particular site or problem, simulating or reproducing an aquifer studied in the field. The concepts are applicable to a wide range of models designed to simulate subsurface processes.

2.2 Methods of Observation

The following is a list of available tools for assessment of local and regional groundwater investigations.

2.2.1 Wells

Existing wells located during field geologic reconnaissance should be sounded, or water levels obtained from the well owners. The well location and the elevation of the top of the well should be checked for accuracy. Pumping quantities, seasonal variations in groundwater 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 groundwater 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. ASTM D 5092-95, 1995, "Practices for Design and Installation of Groundwater Monitoring Well in Aquifers," provides guidance on the design and installation of groundwater monitoring wells to obtain representative and reliable information, once a site specific conceptual model has been developed.

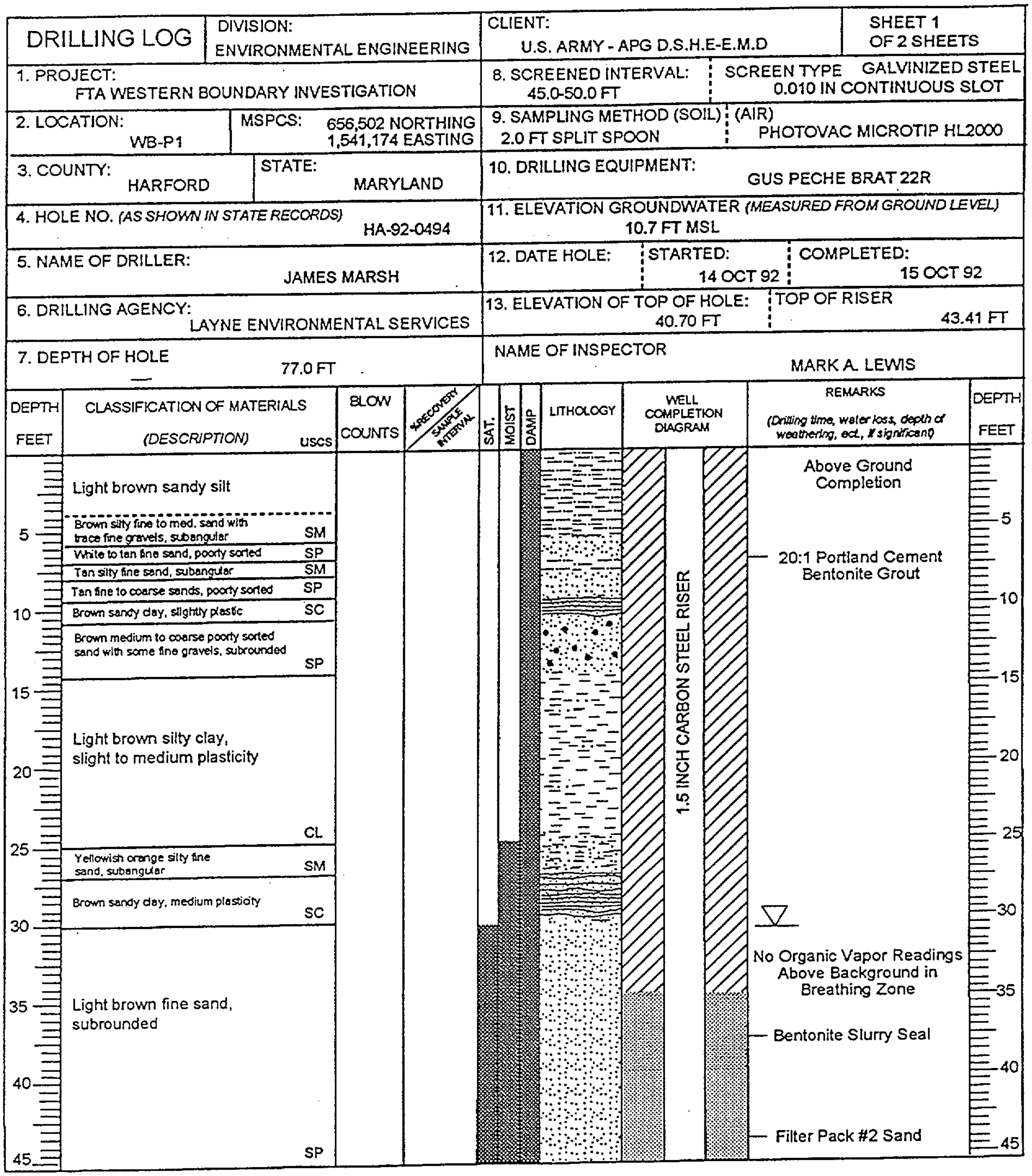
2.2.2 Borings

Water levels recorded on drilling logs are another source of information. They may not reflect true water levels, however 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 groundwater level information is needed, installation of piezometers or observation wells in borings should be considered.

2.2.3 Piezometers and Observation Wells

The most reliable means for determining groundwater levels is to install piezometers or observation wells. All information developed during preliminary studies on the regional groundwater regime should be considered in selecting locations for piezometers and observation wells. Useful references for types of piezometers, construction details, and sounding devices include: EM 1110-2-1908, Part 1 (Department of the Army (DA), 1995) and U.S. Departments of the Army, Navy, and Air Force (1983) . All piezometer borings should be logged carefully, and "as built" sketches prepared that show all construction and backfill details (e.g., Figure 2).

The selection of the screened interval is critical to the information produced since the water level recorded will be that associated with 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 selection of the screened interval and interpretation of the data. One of the greatest benefits of a piezometer or observation well is that it permits measurement of fluctuations in piezometric levels



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 GROUNDWATER *(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
0	Light brown sandy silt	
	Brown silty fine to med. sand with trace fine gravels, subangular	SM
	White to tan fine sand, poorly sorted	SP
	Tan silty fine sand, subangular	SM
	Tan fine to coarse sands, poorly sorted	SP
10	Brown sandy clay, slightly plastic	SC
	Brown medium to coarse poorly sorted sand with some fine gravels, subrounded	SP
15	Light brown silty clay, slight to medium plasticity	CL
25	Yellowish orange silty fine sand, subangular	SM
	Brown sandy clay, medium plasticity	SC
30	Light brown fine sand, subrounded	SP

REMARKS *(Drilling time, water loss, depth of weathering, ect., if significant)*

Above Ground Completion

20:1 Portland Cement Bentonite Grout

No Organic Vapor Readings Above Background in Breathing Zone

Bentonite Slurry Seal

Filter Pack #2 Sand

Figure 2. Example of report quality log with lithologic, blow count, moisture, and well completion information.

over time. To take advantage of this benefit, it is necessary to provide for periodic readings. This can be accomplished through manual reading or with an automated system, depending on the location and critical importance of the area being monitored.

Time Domain Reflectometry (TDR) offers a tool for groundwater elevation measurement within very small boreholes. TDR refers to techniques wherein short pulses of energy generated by an electromagnetic source are reflected back to the source by an anomaly (radar is an early example of TDR technology; TDR technology used for monitoring groundwater elevations is essentially a closed circuit radar). An electrical pulse is sent down a coaxial cable and is reflected by a change in cable capacitance caused by the air/water interface. The distance between the pulse source and the water is calculated from the average of the measured pulse travel time and the pulse velocity. Borehole diameter required to accommodate the diameter of the cable may be quite small, and the cable may be left in place, eliminating the need for decontamination of the equipment between readings on sites where this feature is desired.

Other information that can derived from observation wells and piezometers are temperature and water quality data. Tracer tests can sometimes be done, if there is enough time and the aquifer is sufficiently transmissive, to determine the direction and rate of groundwater flow.

2.2.4 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 base 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.

2.2.5 Tracer Testing

In some areas, especially karst terrains, it is of particular interest to determine preferential flow paths in the groundwater system. Flow paths 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 aquifer access points and monitored at exit points such as springs. The travel time from introduction to detection is recorded. Numerous tests can be run at different locations and times to interpret the overall groundwater flow regime. Tracer tests can be either qualitative or quantitative.

2.2.6 Geophysical Methods

Geophysical methods, such as seismic refraction, can be used to determine the depth to saturated material. Surface resistivity and ground penetrating radar surveys can also indicate the presence of and depth to water (Annan, 1992). Fetter (1988) discusses the application of these and other geophysical methods to characterize the hydrology and hydrogeology of a site. Geophysical survey techniques applicable to geotechnical site characterization are presented in Section V.

2.3 Permeability Testing

Permeabilities and transmissivities of foundation materials can be determined from tests in piezometers and wells (pumping tests and slug tests), laboratory tests on undisturbed samples, and pressure tests in rock foundations. The permeability of sands can be roughly estimated based on (the D_{10} size) the size of particle in a particular gradation for which 10 percent, by weight, of particles are finer (TM 5-818-5). Fracture and joint analysis is important in evaluating permeability of rock foundations. In situ tests for permeability are preferred over laboratory tests since it is very difficult to obtain accurate or representative formation permeability values from laboratory samples and testing. Permeability tests can be performed for rocks or soils, in boreholes, piezometers, or by sealing drillhole sections. The most commonly used tests are the rising and falling head tests, the constant head tests, and the packer or Lugeon tests, which sample the borehole point and the immediate surroundings. To evaluate the groundwater behavior of a larger area in a comprehensive way, pumping tests addressing the whole formation are performed and analyzed using the time-drawdown and distance-drawdown pumping methods. Permeability in soils is an anisotropic property; vertical and horizontal permeabilities may differ greatly and must be independently measured. Groundwater flow models must account for this anisotropy. General reviews of methods to evaluate permeability of soil and rock in the subsurface are given by Heath (1983) and Fetter (1988).

2.3.1 Tests in Piezometers or Wells

Permeability tests can 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. In a slug test a "slug," or fixed quality, of water or of a solid is either added to or removed from the well. The information obtained is representative of a smaller volume of material than that tested in well pumping tests. However, procedures are simple, costs are low, and results may be useful if interpreted with discretion and if tests are performed in several wells. Test details are discussed in EM 1110-2-1908, Part 1 (DA, 1995); TM 5-818-5 (U.S. Departments of the Army, the Navy, and the Air Force, 1983); U.S. Department of Interior (1977), Mitchell, Guzikowski, and Villet (1978); and Bennett and Anderson (1982).

The rising or falling head (slug test) tests are performed on relatively permeable soils in either a cased borehole or a piezometer. To analyze the results of these tests, the Hvorslev basic time lag concept and method maybe used (Hvorslev, 1949). There are also other methods available for different well geometries.

The constant head testing technique is used for soils susceptible to consolidation or swelling with stress changes. In-situ permeability tests performed in clays sometimes will render higher permeability values than actually exist due to the swelling of certain clays when exposed to increased pore water pressure. The purpose of a constant head permeability test is to determine the flow rate under steady seepage conditions after swelling has occurred. Hydraulic fractures may result from the use of high pressures during this kind of test; vertically in normally consolidated soils, and horizontally in overconsolidated soils. The development of these cracks will produce a sudden rise in permeability rates, several orders of magnitude higher than that of the intact material.

The packer or "Lugeon" test is the rock equivalent of the constant head permeability tests for soils. The test can be performed by sealing the test section at the bottom of a drill hole with a single packer (Figure 3-a), or at any depth by sealing the section with a double packer (Figure 3-b). Generally these tests are performed in vertical boreholes, but tests in other borehole orientation can be used to estimate

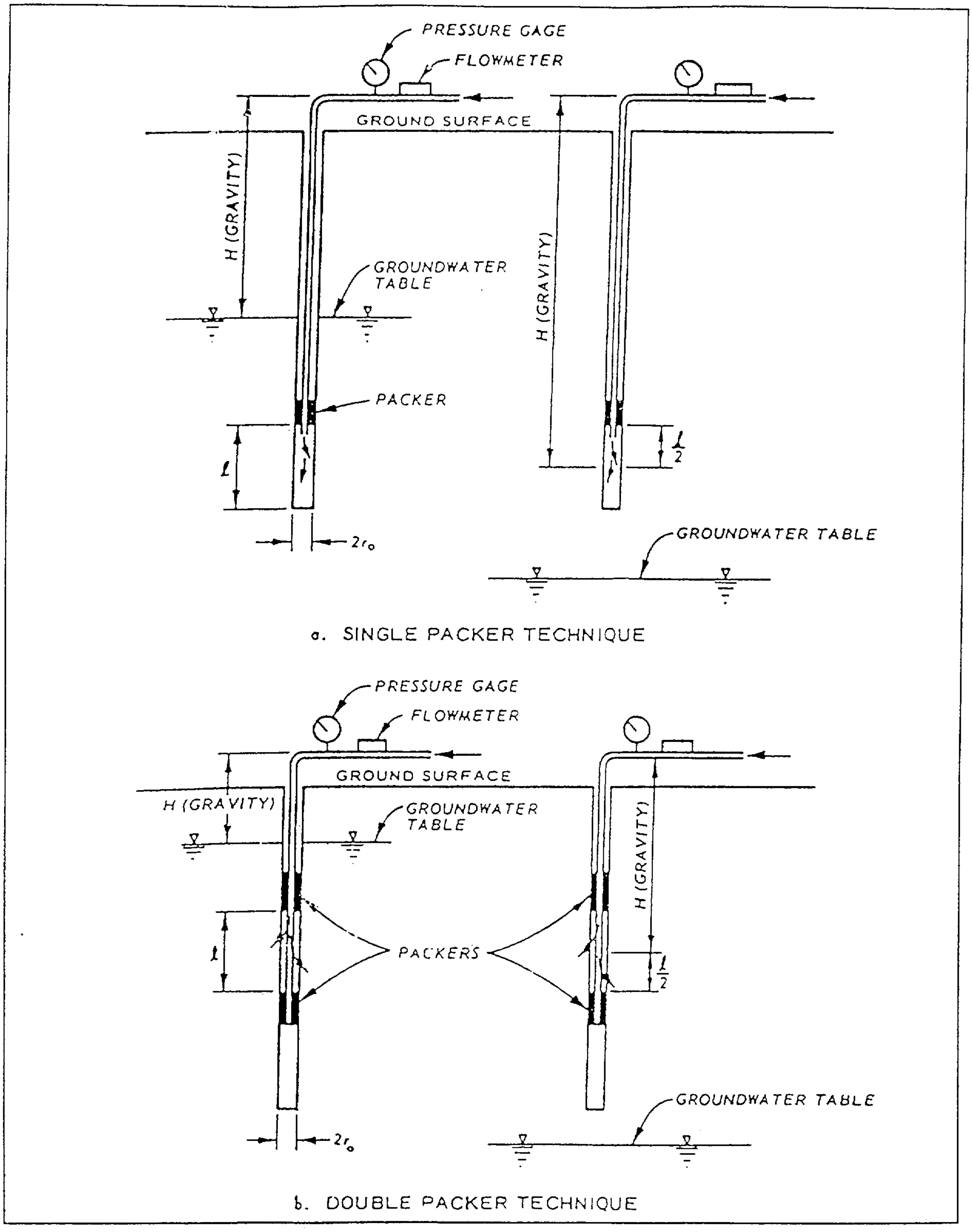


Figure 3. Single packer and double packer techniques (RTH).

rock mass anisotropy. A number of factors need to be taken into consideration when performing these tests, such as the possibility of packer leakage, material disturbance from the drilling process, erosion or distension of fractures, turbulent flow, siltation due to the use of dirty water, and the effect of performing the test above the groundwater level. Suggested method, test protocol and data reduction for these tests can be obtained in RTH 381-80, "Suggested Method for in Situ Determination of Rock Mass Permeability Using Water Pressure Tests."

2.3.2 Pumping Tests

Pumping tests are the traditional method for determining permeability of sand, gravels, or rock aquifers below the water table. The test consists of pumping a well at a constant rate for a period of time ranging from several hours to several days and measuring the change in water level in observation wells located at different distances from the pumped well. The drawdown is the difference between the water level at any time during the tests and the position of the static water level. For example, a pump test for an unconfined aquifer is typically run for 72 hours; confined aquifers may require 24-hour tests; and 8 to 24 hours can suffice for low capacity tests (less than 10 gpm). Piezometers or monitoring wells should be installed to measure the initial and lowered groundwater levels at various distances from the pumped well. Piezometers should have a point intake, no more than a few inches in length, through a porous tip. Monitoring wells may have screened intakes, several feet in length. 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:

- Large or complex projects requiring dewatering.
- Design of underseepage systems for dams or levees.
- Special aquifer studies.
- Projects where water supply will be obtained from wells.
- Projects immediately downstream from existing embankments.

Pumping tests evaluate the groundwater aquifer over an extensive area. The kinds of information that can be obtained from pumping tests include: position and thickness of aquifers and confining beds (from boring information), hydraulic conductivity, transmissivity, storage coefficient, yield, location and nature of aquifer boundaries, hydraulic characteristics of confining beds, and identification of the types of aquifer (confined or unconfined). They are essential to characterizing groundwater behavior in a formation in a three-dimensional and comprehensive way. A pumping test monitors change in water levels in a well or observation wells over time in an aquifer caused by withdrawal from (drawdown test) or recharge to (recovery test) a well.

Several methods are available to analyze pumping test data. Theis (1935) analyzed radial flow to a pumped well in confined aquifers using heat flow analogies; curves developed from Theis' work can be used to analyze the data obtained from the drawdown test. The Theis method permits prediction of the drawdown curve in non-equilibrium (non-steady) conditions by taking into account the effect of pumping time on the amount of water removed, or yield. Modifications of the Theis method have been developed to allow a simple analytical determination of aquifer properties without the tedious curve-matching of the Theis method, although computer assisted analyses are now available that make the curve matching techniques much easier. Some examples follow:

2.3.2.1 Time-drawdown Test. This test involves a modification of the Theis equation whereby under certain simplifying assumptions the time-drawdown curve (the shape of the drawdown phreatic surface) is used to calculate aquifer coefficients. This approach, called the Jacob method (Cooper and Jacob, 1946), works with both time-drawdown and distance-drawdown procedure. It works under the same assumptions of the Theis equation and is sensitive to the changes in shape that occur in the cone of depression during an aquifer tests and changes in the drawdown rate. The Jacob method is applicable only to the portion of the drawdown surface in which steady-shape conditions prevail or to the entire cone of depression only after steady-state conditions have developed (Heath, 1983).

2.3.2.2 Distance-drawdown Test. To perform this kind of test at least three observation wells at different distances from the pumping well are necessary. Drawdown measurements are taken at the same time from the different wells and then analyzed with the Theis equation or the Jacob method to determine the aquifer transmissivity and storage coefficient. The Jacob method can be utilized with a time-drawdown graph, using data from an individual well, or with a distance-drawdown graph using multiple "simultaneous" measurements from all the wells (Heath, 1983).

2.3.3 Permeability of Rock

Most rock formations contain complex interconnecting systems of joints, fractures, bedding planes, and fault zones that, collectively, are capable of transmitting groundwater. This 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 those in 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, like soil deposits, are anisotropic with regard to permeability. The influence of anisotropy on a practical level is that it is easy to over- or underestimate the groundwater 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 preliminary fracture and joint analysis should be conducted prior to laying out a pump test. 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 which 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 groundwater flow should be assessed.

2.3.4 Pressure Tests

Pressure tests are performed to measure the permeability of 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 5 to10 ft but may be varied to fit specific geological conditions observed during the core drilling operations. Zones to be tested should be determined by (a) examining freshly extracted cores, (b) noting depths where drill water was lost or gained, (c) noting drill rod drop, (d) performing borehole or TV camera surveys, and (e) 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).

Pressures applied to the test section during tests should normally be limited to 1 psi/ft of depth above the piezometric level and 0.57 psi/ft of depth below the piezometric level. 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.

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.

Qualitative evaluations of leakage and grout requirements can be made from raw pressure test data (Zeigler, 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.

3 BORINGS AND EXPLORATORY EXCAVATIONS

3.1 Borings

Borings are required to characterize the geologic materials and subsurface characteristics beneath planned foundations. Borings and borehole samples are used to identify and classify soil type, determine engineering properties, describe soil strata or rock, and assess solution and void conditions. The drilling method used to advance the borehole will depend on its applicability to the specific ground conditions and the purpose of the boring. Boring types might be rotary, percussion, and drive (or push). Sampling methods are classified broadly as disturbed, undisturbed, and core and are discussed in Section IV. Some major uses for borings are as follows.

- Define geologic stratigraphy and structure.

3. Borings and Exploratory Excavations

- Obtain samples for index testing.
- Obtain groundwater data.
- Perform in-situ tests (see Section V for geophysical tests and Section VI for penetration and other in situ tests).
- Obtain samples to determine engineering properties.
- Install instrumentation.
- Establish foundation elevations for structures.
- Determine the engineering characteristics of existing structures.

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, and 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 sizes for 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.

A major part of field investigations is the compilation of accurate borehole logs on which subsequent geologic and geotechnical information is 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 together with any 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:

- Observing and describing drilling tools and procedures.
- Observing, classifying, and describing geologic materials and their discontinuities.
- Selecting and preserving samples.
- Performing field tests on core samples (hand penetrometer, torvane).
- Photographing site conditions and cores.
- Observing and recording drilling activities and groundwater measurements.
- Overseeing and recording instrument installation activities.
- Completing the drilling log and/or entering information into a cataloging system.
- 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 on 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.

3.1.1 Boring Methods

Many methods are used to make borings; some of the more common methods are discussed in the following paragraphs. Many of these are also discussed in detail in EM 1110-1-1906 (DA, 1996) and Das (1994). Some factors that affect the choice of methods are:

- Purpose and information required.
- Equipment availability.
- Depth of hole.

- Experience and training of available personnel.
- Type of materials anticipated.
- Terrain and accessibility.
- Cost.
- Environmental impacts.
- Disruption of existing structures.

3.1.1.1 Auger Borings. Auger borings provide disturbed samples that are suitable for determining soil type, Atterberg limits, Proctor compaction 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 bedrock, and installing wells. Auger borings can be made using posthole, helical, barrel, hollow-stem, or bucket augers. Auger samples are difficult to obtain below the groundwater table, except in clays. However, hollow-stem augers with continuous split barrel sampler can retrieve some soils from below the water table.

Truck-mounted auger rigs currently come equipped with high yield and high tensile strength steel augers. New hydraulics technology can now apply torque upwards of 20,000 foot pounds. With this amount of torque, augers are capable of boring large size holes and of being used in soft bedrock foundation investigations. Because augers use no drilling fluids, they are advantageous for avoiding environmental impacts.

Currently, many drilling rigs are actually a combination auger/core/downhole hammer unit. 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. Another advantage of using augers is the ability (using hollow stems) for soil sampling, i.e., taking samples below the bit. Hollow stem augers use 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. However, the soil below the bottom of the auger may become disturbed by the process of advancing the auger or by the removal of the plug at the bottom of the auger. The latter cause of disturbance is particularly serious for sandy soils below the water level.

3.1.1.2 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 non-rotating, impact driven method for making a hole by continuous sampling utilizing a heavy wall drive barrel. Where larger samples are required, the most suitable drill for this method is the cable 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.

3.1.1.3 Wash Borings. A wash boring is advanced by a combination of water circulation and a loosening of the soils using various types of bits. The circulation of water (or other fluid) brings the soil cuttings to the surface. Water is injected from the bottom of the drilling rod assembly, with the water discharge being upwards to prevent disturbance below the bottom of the borehole. The borehole is kept open with a casing that is advanced separately, and/or with the use of a heavy bentonitic mud. When advancing boreholes below the water table, it is important to maintain the level of the fluid in the borehole above the water table to prevent the development of a quick condition at the bottom of the borehole. Wash boring is the preferred method of advancing a borehole for obtaining good quality samples or obtaining reliable measurements of penetration resistance (see Section IV).

3.1.1.4 Rock Core Boring. 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.

3.1.1.4.1 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.

3.1.1.4.1.1 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.

3.1.1.4.1.2 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 drill 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.

3.1.1.4.2 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; amount of core actually recovered; amount of core loss or gain; and amount of core left in the hole (tape check). From these data the unaccountable loss, i.e. the core that is missing and unaccounted for, should be computed. This loss should be shown on the graphic log at its most likely depth of occurrence based upon the drill action and close examination of the core.

The "N" size hole (3.0 in., or 75 mm O.D.) 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 (2.5 in., or 65 mm O.D.) and H (4.0 in., or 100 mm O.D.), 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.

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 does provide a statistically better size sample for laboratory testing.

Although the majority of soil and rock core borings are drilled vertically, inclined and horizontally oriented borings may be required to adequately define stratification and jointing. Inclined borings should be used to investigate soils beneath existing structures or steeply inclined rock jointing in abutments and valley sections for dams, along spillway and tunnel 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.

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 discontinuity 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 that are angled to this bedding. Once oriented, the attitudes of discontinuities can be measured directly from the core.

3.1.2 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 identifies the type of soil (clay, sand, etc.), places it within established groupings, and includes 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, "Description of Soils (Visual-Manual Procedure)." The most widely used classification scheme is the Uniform 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 TM No. 3-357 (DA, 1960), used all over the Corps, and by Schroeder (1984). Cernica (1993) provides detailed procedures to evaluate the physical properties of soils. 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 are shown in Appendix C.

3.1.3 Rock Core Logging

Each feature logged should 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. Readers should also be able to obtain some idea of the appearance of the core and an indication of its physical characteristics. A complete photo log of recovered core is very helpful in recording core appearance. The log should contain all the information obtainable from the core pertaining to the rock as well as discontinuities. Examples for presenting core logging data are shown in Appendix C.

Proper classification of subsurface materials can allow engineers to anticipate the soil or rock behavior during and after construction. The description of the site soils and rocks will be done first by visual inspection and simple field tests, and later corroborated in the laboratory. The engineering behavior of the sampled materials can be assessed by their index properties, obtained from established classification tests.

Generally, the characteristics by which soils and rocks are described and classified include grain size, constituency, mineralogical composition, strength, degree of induration, plasticity, porosity, fractures, faults, deformations, and degree of weathering. Guidance in the description and engineering classification of intact rock and rock masses for USCE is available in EM 1110-1-2908 as an example.

3.1.3.1 Lithologic Description. Each lithologic unit in the core should be logged. The classification and description of each unit should be as complete as possible. A recommended order of descriptions is as follows:

- Unit designation (Miami oolite, Clayton formation, Chattanooga shale).
- Rock type and lithology.
- Hardness.
- Degree of weathering.
- Texture.
- Structure.
- Discontinuities (faults, fractures, joints, seams).
- Orientation with respect to core axis.
- Asperity (roughness).
- Nature of infilling or coating, if present.
- Staining, if present.
- Tightness.
- Color.
- Solution and void conditions.
- Swelling properties.
- Slaking properties.
- Additional descriptions such as mineralization, inclusions and fossils.

Criteria for these descriptive elements are contained in Table 2. Murphy (1985) provides guidelines to geotechnical descriptions of rock and rock masses. Geological Society Engineering Group Working Party Report (1995) provides a description and classification scheme of weathered rocks for engineering purposes. TL 1110-2-282 outlines rock mass classification data requirements for rippability.

Variation from the general description of the soils and features not included in the general description should be indicated at the depth and the interval in the soil sample sequence or rock core where the feature exists. These variations and features should be identified by terms that will adequately describe the feature or variation so as to delineate it from the general description. They 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 amount of core left in the bottom of the hole after the final pull.

3.1.3.2 Rock Quality Designation. An easily done and widely used measure of the quality of the rock mass is provided by the Rock Quality Designation (RQD). This system, developed by Deere and Deere (1989) is a modified core recovery percentage which incorporates only intact pieces 4 inches or longer. In practice the RQD is measured for each core run and reported on an appropriate form for record. Many of the rock mass classification systems in use today are based, in part, on the RQD. Its wide usage 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 any effect. 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.

3.1.3.3 Solution and Void Conditions. Solution and void conditions should be described in detail because these features can affect the strength of the rock and can indicate potential groundwater seepage paths. Where cavities are detected by drill 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 drill fluid. If drill action indicates material is present, i.e., 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 drill action indicates the cavity was open, i.e., no resistance to the drill tools and/or loss of drill fluid, this should be noted on the drill 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 drill site, spacers (ER 1110-1-1802) should be placed in the proper position in core boxes to record voids and losses.

3.1.3.4 Rock Joint Description. Detailed descriptions of joint orientation, degree of fracturing, joint wall weathering and separation, and frequency in the rock mass or core, are necessary and useful for assessing sliding stability, bearing capacity, and deformation/settlement. Strike and dip angles of discontinuities, as well as thickness, and joint aperture should be recorded.

3.1.3.5 Photographic and Video Record. A color photographic record of all core samples should be made. Digital color photographs should be taken as soon as possible after retrieving the core samples. 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.

3.2 Borehole Examination

3.2.1 Visual Records

3.2.1.1 Borehole 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 alignment survey devices.

3.2.1.2 Borehole Camera and Borescope. Borehole cameras that have limited focus capability are satisfactory for examining bedrock features on the sidewalls of the borehole. However, the small viewing area and limited focus reduce their usefulness in borings that have caved or have cavities. They are best used for examining soft zones for which cores may not have been recovered in drilling and for determination of the dip and strike of important structural features of the rock formation. Borescopes have limited use because of their small viewing area, limited depth, and cumbersome operation. They are relatively inexpensive to use.

3.2.1.3 Borehole Video 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 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 and mines, tunnels, and shafts. 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 images or televiewers can be used to determine the strike and dip of discontinuities in the borehole wall.

3.2.2 Alignment Surveys

Precise determination of travel distance is required for proper interpretation of results from geophysical tests (presented in a later section) between two boreholes. Alignment, or deviation, surveys are required to account for plumb and/or orientation of a borehole if subsequent geophysical surveys are to be performed. Older methods employed a compass and photograph system which was relatively easy to use. More modern systems are electronic. Alignment surveys are particularly necessary in deep holes, and where instrumentation packages are to be installed, or where precise determinations of structural features in the rock formation are required. Borehole deviation survey services are available from commercial logging or drilling firms.

3.3 Exploratory Excavations

3.3.1 Test Pits, Shafts, 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 20 to 30 feet, and sides may require shoring if personnel must work in the excavations. Test pits, however, that are hand dug with pneumatic jackhammers and shored with steel cribbing can be dug to depths exceeding 60 feet. Test pits and trenches generally are used only above the water table. Test pits that extend below the water table can be kept open with air or electric powered dewatering pumps. Test shafts may be excavated using large augers or other caisson drilling equipment; these shafts are maintained open using steel casing and can extend deeper than 60 feet. Dewatering from inside the excavation may prove practical in a cased shaft until inflow either exceeds the capacity of the pumping equipment or uplift seepage pressures disturb the sampling surface at the bottom of the shaft. Otherwise a dewatering system must be installed outside the cased excavation capable of maintaining the water level below the maximum depth of the intended shaft exploration. One or more of these excavations may be desired to permit access to in situ soils for in-place density determination and high-quality sampling.

Exploratory trench excavations are often used in fault evaluation studies. An extension of a bedrock 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, determine the geomorphic history, and assessing cultural resource potential.

3.3.2 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 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, can not be achieved reasonably by any other means, the calyx hole may be the procedure of choice. The method involves circulation of a sludge of steel shot that aids in the abrasion of rock at the leading edge of an internal, coring bit (Krynine and Judd, 1957).

4 SAMPLING METHODS

The depth, layout, spacing of sampling borings, and sampling requirements for a site study depends on the subsurface requirements of the foundation, which are a function of the intended use of the site, and the complexity of the subsurface. Complex areas with evidence of adverse anomalies or discontinuities will require supplementary borings or soundings which would be distributed according to the features found and at a spacing small enough to detect them. Subsurface investigation methods for favorable or uniform typical geologic conditions are tabulated in Appendix D. Guidelines for spacing and depth of borings for safety-related structures are included in Appendix E.

The number of samples required to adequately characterize the engineering features of a foundation site will depend on the complexity of the study area. Where the geology of the area is known to be relatively uniform, a lower frequency of sampling will suffice. In more complex areas, further sampling will be needed to guarantee an adequate and representative coverage. The variation, size, and periodicity of features that need to be detected in a complex study site must be considered to design and customize the sampling plan. Probabilistic considerations for drilling explorations are presented as an option for the correct sampling coverage of a complex site (Franklin et al., 1981), using geometric probability and search theory. Exploration grids can be used when there is no available information on the deposition and location, size, or periodicity of target features, to assure uniform coverage.

4.1 Techniques for Recovery of Undisturbed Samples

Borings for undisturbed sampling of soils - It is physically impossible to obtain truly "undisturbed" samples because of the adverse effects resulting from the sampling process itself (e.g., unloading due to removal from confinement), 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 careful measurements are made to document volume changes that occur during each step in the sampling process. 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.

There are no standard or generally accepted methods for undisturbed sampling of cohesionless soils. Care is necessary in transporting any undisturbed sample; sands and silts are particularly vulnerable to vibration disturbance. One method to prevent handling disturbance is to obtain 3-inches. Shelby 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. For both methods, disturbance by cryogenic effects must be taken into account. Singh, Seed, and Chan (1982) examined in situ freezing techniques for undisturbed sampling of saturated sands. They found that strength behavior of clean sands was essentially unaltered by freezing, either in situ or once sampled in thin-walled tubes removed from the ground, as long as free water was allowed to drain out the sample ahead of the freezing front. Tani and Yasunaka (1988) studied the effects of in situ freezing to sample sands with up to

6 percent particles finer than 74 micrometers (i.e., passing the U.S. Standard No. 200 sieve). Their results indicated that there was no change in cyclic triaxial laboratory strengths for alternately frozen and thawed specimens. The authors claimed that cyclic strengths measured in samples taken by in situ freezing were thus representative of "true liquefaction resistance." Soils with higher fines contents, particularly low-plasticity silty or clayey soils, may be highly susceptible to frost heave (e.g., Lambe and Whitman, 1969), thus in situ freezing may be inadvisable in fine-grained soils.

So-called "undisturbed" samples are normally obtained 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 drill rig then enlarging the diameter of the sampled interval by some "clean out" method before beginning to sample again. Commonly used systems for push samples include the Hvorslev fixed-position sampler and the Osterberg hydraulic piston sampler. Rotary samplers are considered slightly more disruptive to soil structure and involve a double tube arrangement similar to a rock coring operation except that the inner barrel shoe is adjustable and generally extends beyond the front of the rotating outer bit. This reduces the disturbance caused to the sample from the drill fluid and bit rotation. Commonly used rotational samplers include the Denison barrel and the Pitcher Sampler.

4.1.1 Hand Trimming of Block Samples

Test pits and shafts offer the only effective access to collect high quality block samples and to obtain very detailed information on stratification, discontinuities, or preexisting shear surfaces in the ground. Cost increases with depth as the need of side wall support arises. Samples can be obtained by means of hand-carving oversized blocks of soil or hand-advancing of thin-walled tubes as performed using the GEI sampler (GEI Consultants, Winchester, MA) or other, similar device. The GEI tripod sampler is used to maintain a 3-inch diameter, 14-inch long, thin wall sampling tube in vertical alignment. The tube is advanced in increments of about 1/2 inch by using no more than light hand pressure. About 1/2 to 1 inch of soil around the periphery of the tube, below the cutting edge, is excavated prior to advancement of the tube. This pre-excavation allows soil to easily peel away from the tube as it is advanced and minimizes volume changes during sampling. Several hours of effort are required to obtain each sample. Detailed measurements of tube penetration and soil recovery are made during advancement of the tube. These measurements allow control and documentation of any volume changes that might occur during sampling. An example of the use of this sampler is described by Castro, Keller, and Boynton (1989).

4.1.2 Thin-wall Tube Samplers

Two early versions of samplers continuously pushed to recover high quality samples are shown in Figure 4. Check valves prevent washout of soil within the tubes on removal from the boring. Samples are extruded mechanically, or the tubes may be sawed in half. The tubes are advanced either by direct pushing (without rotation), or by means of a rotary core-barrel apparatus. Rotary core-barrel samplers include the Denison sampler and are best suited to sampling in stiff soils that would preclude smooth penetration of direct push devices, or soils which contain appreciable gravel particles.

4.1.3 Fixed-piston-type Thin-wall Tube Samplers

Two types of fixed-piston sampler are generally deployed to obtain high-quality samples in clays, silts and sands without boulders or gravel-sized particles: mechanically activated (e.g., the Hvorslev fixed-piston sampler, Figure 5, and the Butters sampler, which is a simplified version of the Hvorslev device); and hydraulically or pneumatically activated (e.g., the Osterberg sampler or similar devices). Detailed directions on assemblage and sampler operation for both types of fixed-piston devices can be found in

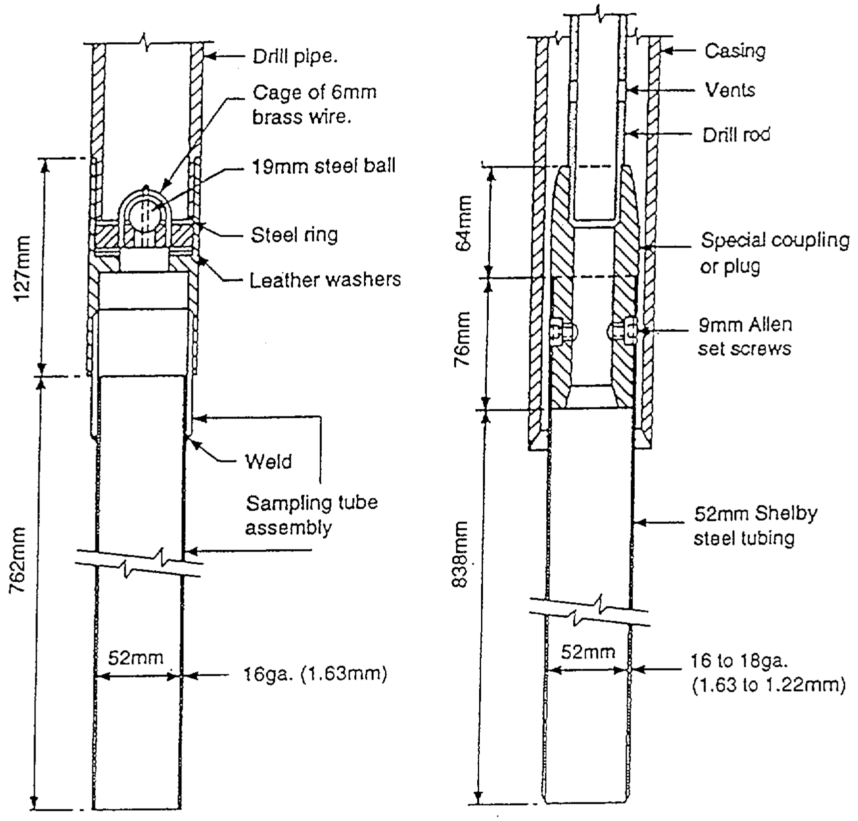


Figure 4. Early thin-walled open-drive samplers, Hvorslev 1940. (Clayton, Matthews, and Simons, 1995; reprinted with permission)

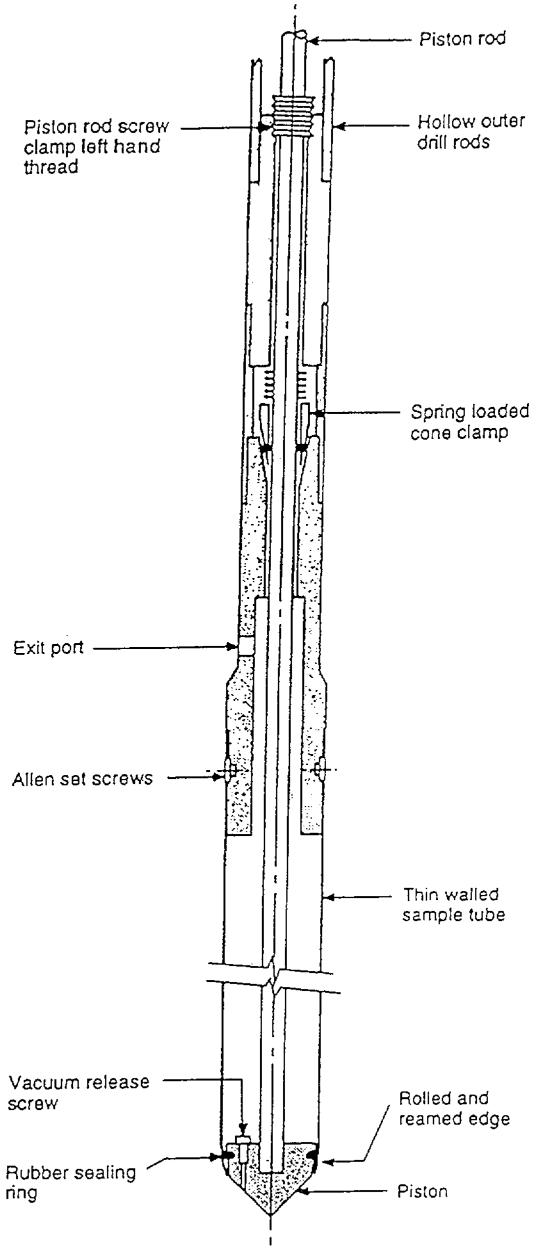


Figure 5. Hvorslev fixed-piston thin-walled tube sampler (Clayton, Matthews, and Simons, 1995; reprinted with permission).

EM 1110-1-1906 (DA, 1996). When fixity of the piston upon advancing the tube is important, the piston rods should be secured to a fixed frame attached to the ground or the boring casing, but not to the drilling rig. The drilling rig is the reaction for pushing the tube down, and thus it will move upwards as the tube is pushed. Practitioners have encountered problems maintaining fixity of the piston activation rods in holes at depths greater than about 150 feet, due to flexibility inherent in rod strings of that length. Hydraulically activated piston samplers can be modified so that the length of the push and the subsequent locking of the piston are accurately controlled to a predetermined value (Dr. Gonzalo Castro, GEI Consultants, Winchester, MA, personal communication).

4.1.4 Foil and Stockinette Samplers

These samplers were developed to obtain long, continuous samples in soft, cohesive soils, and are pushed with a fixed-piston operation. In each, as the sampler is pushed (not requiring a borehole), the internal piston retracts and a sliding liner (either an impervious metallic foil or a woven stocking) is unrolled from a storage magazine in the sampler head to surround the sample within the sampling tube as the drive progresses. Specific design and operational aspects of each device are detailed in EM 1110-1-1906 (DA, 1996).

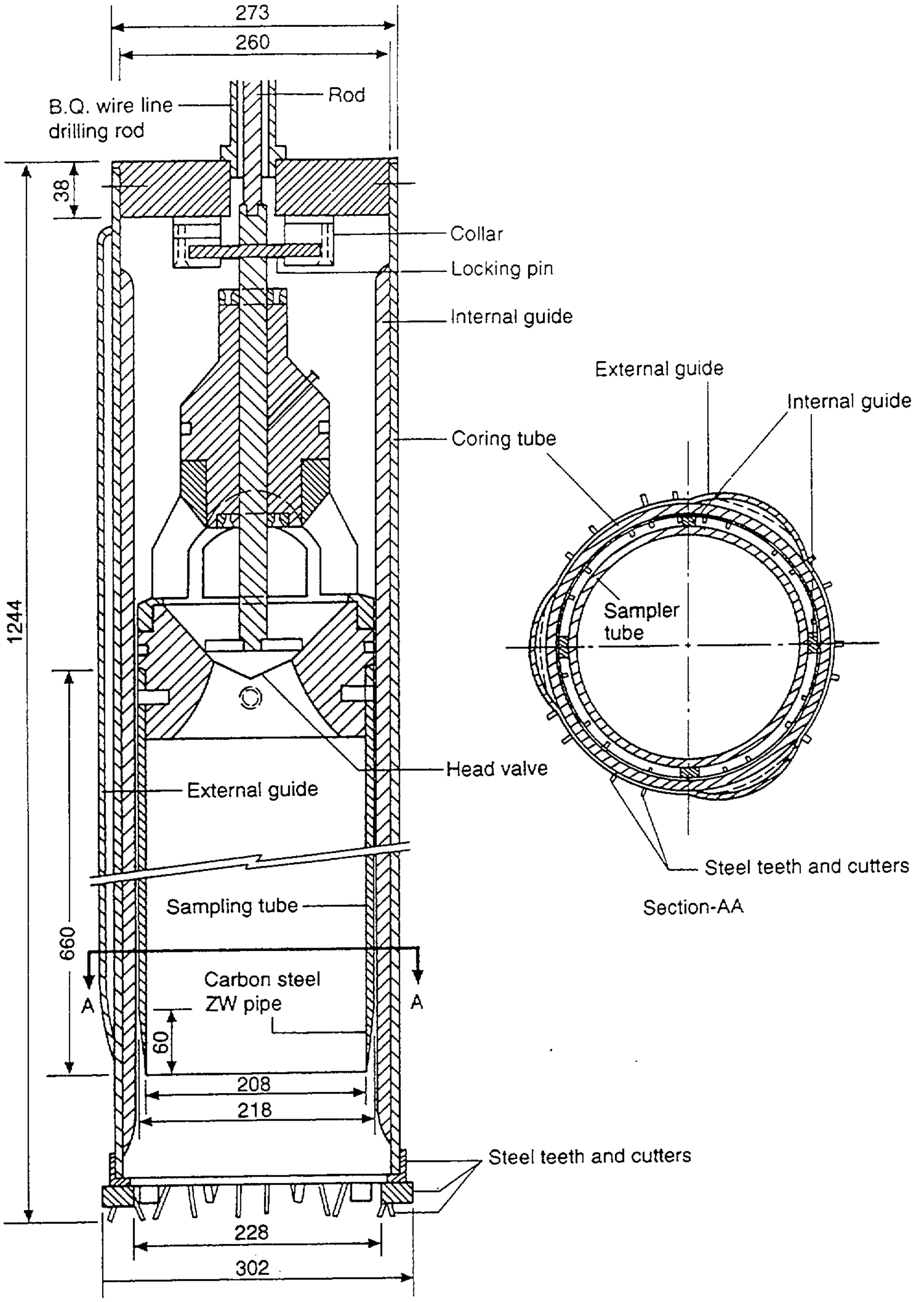
4.1.5 Large-diameter Laval Sampler

The large-diameter Laval sampler is not commonly used within the U.S. It was originally designed for sampling soft, sensitive marine clays (La Rochelle et al., 1981) and has been successfully used to sample saturated cohesionless soil deposits like fine-grained saturated sands (Konrad et al., 1995). This technique for fine-grained saturated sands is a less expensive alternative to in situ freezing and more efficient than thin-walled tubes for undisturbed sampling. Thin-walled tubes have a tendency to loosen dense sands and densify loose soils due to the small ratio of wall thickness to the diameter of sampling tube (Baligh, 1985). The large-diameter Laval sampler dimensions help to minimize the mechanical disturbance of the samples since the ratio effect has been taken into account. The Laval sampler has produced high quality undisturbed samples of sensitive clays that proved to be equivalent to block samples (La Rochelle et al., 1981).

The Laval sampler consists of a sampling tube fixed to a sampler head inside an overcoring tube (Figure 6) (Konrad et al., 1995). The sampling head has an opening in the center that allows the mud to flow from the sampling tube to the overcoring tube as it is pushed into the ground. The hole can be closed from the surface with a central rod avoiding the flow of mud into the sampler head as the sampler is removed. Details on the Laval sampler drilling equipment and sampling procedures can be found in Konrad et al., 1995 and La Rochelle et al., 1981. The sample is carefully retrieved to the surface, avoiding any kind of tilting to preserve sand structure. Once the sample is on the surface, unidirectional freezing in open system conditions is used to preserve and solidify the sample for transportation (Figure 7) (Konrad et al., 1995).

4.2 In Situ Soil Testing

The standard penetration test (Figure 9) is a soil index test used to obtain the resistance to penetration by counting the number of blows required to drive a steel tube of specific dimensions into the subsoil a specified distance using a hammer of a specified weight (EM 1110-1-1906, DA, 1996). The Standard Penetration Test (SPT) method of drive boring, described in ASTM D 1586, is probably the most commonly used method for advancing a hole by the drive method. Slight variations exist with this method, primarily concerning the sampling interval, clean out method, and the refusal criteria, but the



The Laval 200 mm diameter tube sampler (La Rochelle, Sarrailh, Tavenas, Roy and Leroueil 1981).

Figure 6. The Laval tube sampler (Clayton, Matthews, and Simons, 1995; reprinted with permission).

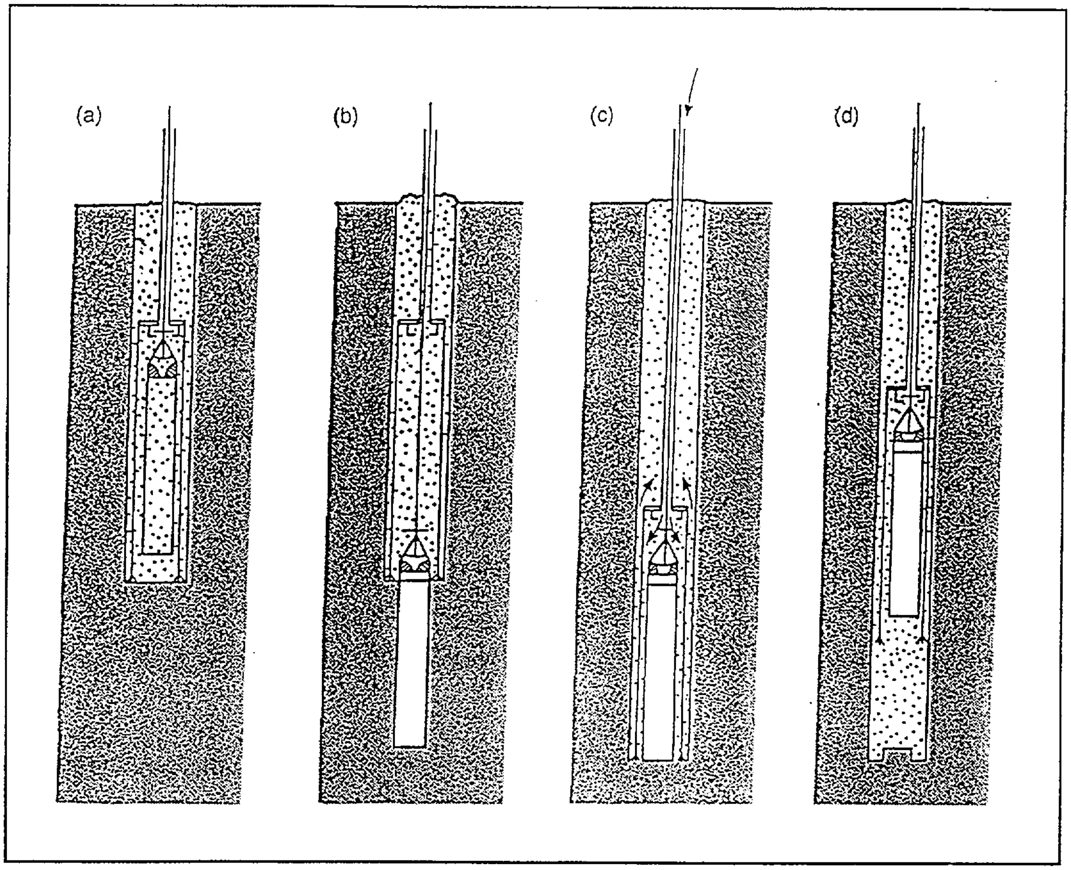


Figure 7. General operation of the Laval sampler (Clayton, Matthews, and Simons, 1995; reprinted with permission).

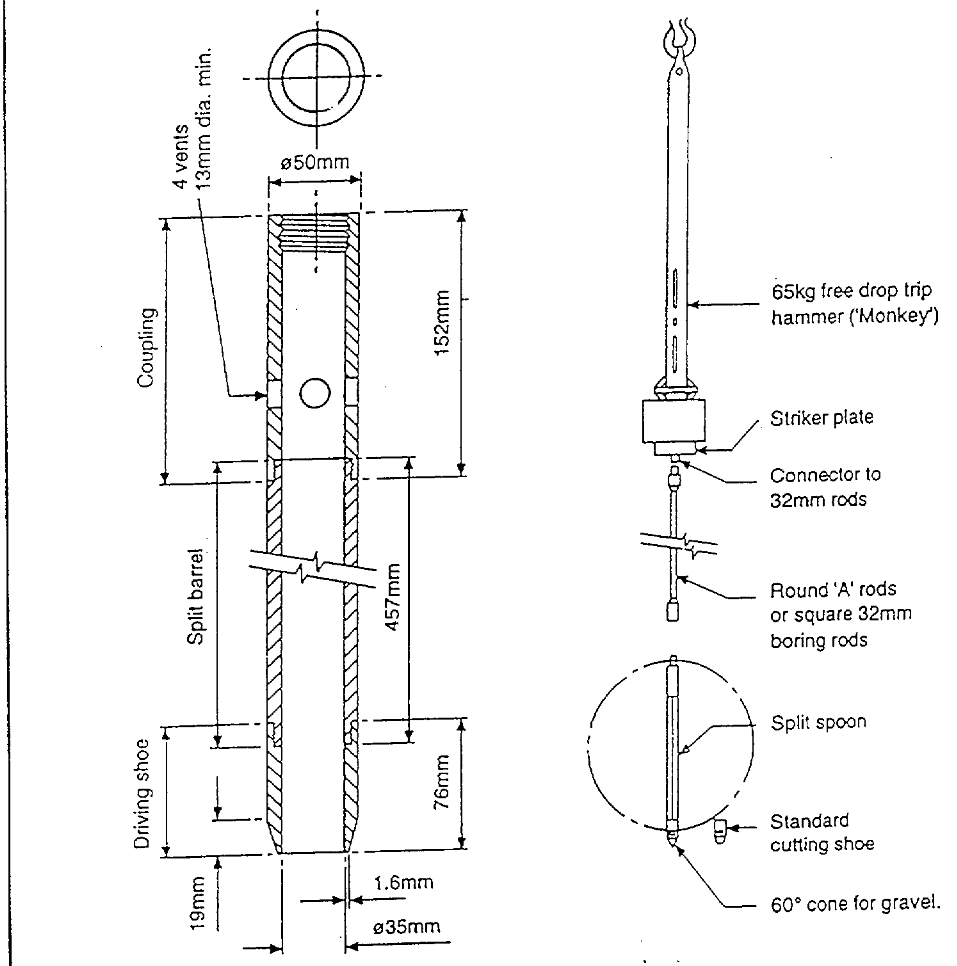


Figure 8. Standard Penetration Test equipment (Clayton, Matthews, and Simons, 1995; reprinted with permission).

fundamental procedure follows the ASTM standard. In this method a standard configuration 2-inch-OD split barrel sampler at the end of a solid string of drill rods is advanced for a 1.5-foot interval using a 140-pound hammer (the safety hammer used in the U.S. is not shown in Figure 8) dropped through a 30-inch free fall. The blows required to advance the hole for each 6-inch interval are recorded. The hole is then cleaned out 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 half foot of penetration. When used to define the top of rock, great care and close examination of samples are required to minimize uncertainties. 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 SPT can be used in relatively clean medium-to-coarse sands and fine gravels with a variable amount of moisture, and in saturated or nearly saturated cohesive soils. Non-saturated cohesive soils and saturated silty sands can produce misleading N-values (the standard penetration resistance, or number of blows required to penetrate the final 12 inches of an 18-inch drive). Typical SPT testing and sampling intervals are of 0.6 to 1.5 m (2 to 5 feet) in uniform strata and whenever there is a change of strata. Control of the hammer drop can be by manual, automatic, semiautomatic, or trip-hammer drop systems.

Borehole disturbance can affect SPT results dramatically, and may reduce N values to a fraction of the value that would be obtained in undisturbed ground. SPT blow count data can also be affected by the penetration interval, the sampling tube design, the number of turns of rope on the cathead winch used to raise the safety hammer between blows, variations on drop height, energy delivered to the sample tube , or split-spoon, in situ stress conditions, faulty equipment, and overboring. The suggested approach is to follow standard procedures, to be aware of the potential errors and error sources that can affect the technique, and take errors into account when collecting and interpreting SPT data. EM 1110-1-1906 (DA, 1996) describes the STP method and suggests modifications to the interpretation procedure.

The N-value has been empirically correlated with liquefaction susceptibility under seismic loadings (Seed, 1979). The use of SPT data in correlations developed for liquefaction potential assessment requires detailed documentation of field procedures. Energy measurements have been made to permit comparison of data obtained using different hammers, rigs, and operators, accounting for blow count variation due to these factors. For safety-related structures where earthquake effects need to be considered, it is recommended that driving energy be determined and recorded sufficiently often to calibrate the equipment used at a specific site. 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) and the unconfined compressive strength (Mitchell, Guzikowski, and Villet, 1978) of soils. The split-spoon sampler driven during an SPT recovers disturbed soil that may be used for visual classification and laboratory index testing, including determination of natural water content. Longer samplers are commercially available for recovery of coarse-grained soils.

4.3 Alternative Sampling Techniques

4.3.1 Hand sampling

Hand sampling in test pits, shafts, or other accessible excavations may be required to produce representative samples in boulders, gravels, or sand-gravel mixtures.

4.3.2 Freezing Sampling

In situ freezing followed by coring of frozen soil is an alternative technique to collect undisturbed samples of cohesionless materials like loose sands and gravelly soils. In situ freezing can be used to stabilize the soil before sampling or after sampling to avoid further disturbance of the sample. The quality of the samples obtained by this method will depend on the amount of density and soil structure changes the freezing and the sampling technique may provoke in the sample. The recommended freezing technique for high quality undisturbed samples consists of one-dimensional ground freezing followed by core sampling (Singh, Seed, and Chan, 1982). Relatively clean sands or gravels with good drainage are good materials to sample by freezing (Figure 8) since the freezing process can be performed and maintained one-dimensionally. Their inherently good drainage allows the freezing front to expand out through the ground without major alteration of the volume or soil structure as the water freezes. In situ freezing will be inhibited at sites with high groundwater gradient, as warmer groundwater flowing through the target zone may prevent the recirculating coolant from maintaining freezing temperatures in the soil to be sampled. The amount and nature of fine material present in the ground will also dramatically affect the applicability of freezing as a source of undisturbed samples. The presence of fines may inhibit the free drainage of the excess water created by the expansion of water into ice, thus causing void expansion and possibly ice lenses that will severely damage the structure of the soil. It may be necessary in the laboratory to observe a thaw protocol, such as providing confinement pressure near the in situ level during warm-up of the specimen in a test cell, to preserve original structure and strength behavior in frozen soils (Dr. Dave Sego, University of Alberta, personal communication).

4.3.3 Chemical Stabilization or Impregnation

Chemical stabilization or impregnation can be used to obtain samples for density determination from boreholes. Chemical impregnation provides an option to sample and preserve the natural structure of cohesionless granular material that would be hard to sample without disturbance, if at all. Agar has been used as an impregnation material for undisturbed sampling of sands below the water table as an alternative to freezing with positive results. The characteristics of the agar make its use simple, inexpensive, and free of contamination potential. The material is easy to handle and does not seem to appreciably change the soil's natural characteristics.

Chemical impregnation can be used either after sampling, to avoid further disturbance in transportation and handling of the samples, or by in situ impregnation before sampling. Both ways represent an alternative to freezing that is less expensive and produce samples that are easier to manage after collection. Removal of the impregnating material maybe accomplished once the sample is in the laboratory.

4.4 In Situ Sampling for Specific Site Conditions

4.4.1. Cohesive and Plastic Soils

Cohesive and plastic soils are relatively easier to study and sample than cohesionless soils, with a variety of techniques. These soils include all soils with some degree of cohesion and plasticity, from clayey, fine sand and silt, to plastic clays and firm peat. Depending on the soil consistency, several methods of boring and sampling can be used to obtain suitable results.

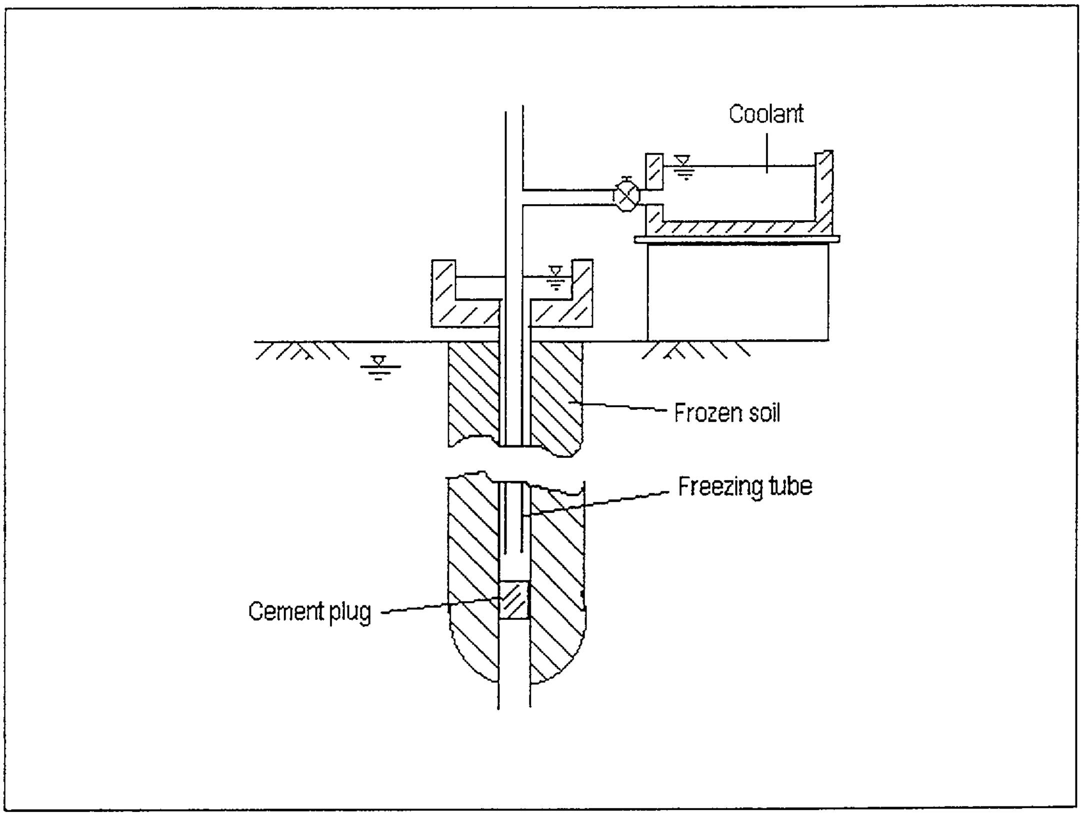


Figure 9. Method of sand sampling by freezing probe (after Yoshimi, Hatanaka, and Oh-Oka, 1977).

4. Sampling Methods

Uncased and dry bore holes are usually sufficient for sampling at shallow depths (less than 20 feet). Deeper boreholes should be stabilized with casing or drilling fluid to prevent caving when sampling. Open drive samplers can be used to obtain undisturbed samples, but thin-walled samplers of large diameter used with a stationary piston produce the longest, least disturbed samples. Core boring procedures can be used on stiffer soils. Advance trimming or box sampling are recommended for stiff and brittle soils. Since cohesive and plastic soils vary widely in their physical properties, the maximum depth of penetration and the diameter of the sampler will depend on the soil character. In general, sample loss is not as common when the sample diameter is less than 3 inches, but when retrieving larger samples a snare wire should be used to cut samples from the subsoil.

4.4.2 Slightly Cohesive and Brittle Soils

The soils included in this category include silt and fine sand with small amounts of clay or other cementation, such as loess, and partially saturated silt and loose sand with some apparent cohesion. Since these soils are easily disturbed, borings in partially saturated soils should be kept dry or if necessary filled with viscous drilling fluid instead of water. Casing, when used, should be advanced without vibrations and never ahead of the core. Generally, good samples can be obtained using open drive samplers or piston samplers. The sampling tubes should have very thin walls to avoid disturbance or displacement of the soil. Samplers with a stationary piston will produce the longest, least disturbed samples, with less risk of losing the sample. Even though core sampling can be used satisfactorily in these kinds of soils (e.g., using the Denison core barrel sampler), it is not clear that the vibrating disturbance of thus produced is not greater than the soil displacement disturbance produced by a thin-walled sampler.

4.4.3 Very Soft Soils

The soils included in this category include soft clays, soft peat, organic silt, and various mixtures of these soils or fine sand with decayed organic matter. These soils are often found in rivers, estuaries, tidal flats, and harbors and are commonly called mud or ooze.

Casing and drilling fluid are necessary to avoid caving of the boreholes when sampling these soils. Friction and adhesion develop quickly producing lateral deformation and internal friction that can be so high that the soil under the sampler is pushed aside instead of entering the tube. To avoid this, the pressure over the sample has to be reduced, or the friction and adhesion development prevented by a high speed of penetration.

Good quality samples are difficult to obtain in very soft or sensitive clays. The best procedure to obtain soft soil samples involves a piston sampler with stationary piston, as described earlier. This technique allows for consistent recovery of samples 2 inches in diameter and a length of 3 to 4 feet, even in extremely soft and sticky soils (longer samples are possible with the foil or stockinette samplers described in a previous section). Samples larger than 3 inches diameter are harder to recover even with increased pressure below the sample unless core retainers are used. Shallow surface samples can be obtained with short, thin-walled drive samplers of either open or stationary piston type. To sample at greater depth, stress changes and plastic deformations can be minimized by sampling from boreholes kept filled with water or drilling mud.

4.4.4 Saturated Silt and Loose Sand

The soils in this category include inorganic silt and loose to medium dense sand when found below groundwater level.

Casing and drilling mud or fluid are necessary to avoid caving of the boreholes when sampling these soils. When undisturbed samples are the goal, the casing should be advanced by rotation, and jacking. Vibrations should be avoided since the internal structure and the void ratio of these soils can be easily disturbed. The casing may be kept 1 to 2 feet above the bottom of the borehole prior to sampling, to minimize disturbance of the soil to be sampled caused by casing advancement. The bottom of the borehole must be carefully cleaned with special tools to remove cuttings and other disturbed materials. A firm clean bottom is essential for good quality sampling.

The preferred method to advance the borehole for sampling of saturated silts and sands is by washing techniques, with a bentonitic mud as drilling fluid. Truly undisturbed sampling of these materials in their in situ condition is not possible. Some volume changes are unavoidable. It is thus important for the sampling procedure to allow for the measurement of the volume changes so that the quality of the sample can be assessed and for making corrections to the soil properties measured in the samples. A fixed piston sampler has been successfully used for this purpose (Castro, Keller, and Boynton, 1989).

The tube sampling procedure consists of lowering the sampler to the bottom of the borehole with the actuating rods (attached to the fixed piston) inside the drill rods. The actuating rods are then fixed to an independent frame that is set up over the borehole. The frame can be supported on steel rods driven into the ground. The sampler is pushed hydraulically in a smooth continuous motion until a maximum allowable pressure or 24 inches of penetration is reached. Trial sampling is needed to select the maximum hydraulic pressure that can be used without deforming the cutting edge. Short samples are acceptable for measuring void ratio and laboratory testing. The penetration and any movements of the actuating rods and frame are carefully measured before and after each push. The tube is withdrawn from the bottom of the borehole in a smooth constant motion using hydraulic pressure to pull the tube at a rate of 1 inch per second or less for the first 2 feet. After the sampler is pulled free from the bottom of the borehole, withdrawal continues to the ground surface at a slow uniform rate no greater than 1 foot per second.

Special care must be taken to avoid jarring or disturbing the tube samples during sampling, storage, and transportation. Each tube must be held in a vertical position at all times from sampling in the field to arrival at the laboratory. Precise measurements are made of sample penetration, sample recovery length, and any change in sample length during storage and transportation.

The clearance ratio of the cutting edge of the sampling tube should be adjusted for the particular sand so that the clearance ratio (CR) has the minimum value necessary to retain samples. Typically, values of CR of about 0.5 percent work for most sands.

For sample diameters up to 3 inches, injection of compressed air under the sample or advance and cleaning before withdrawal may sometimes prevent loss of sample. Freezing the lower part of the sample before withdrawal has been proposed as a way of preventing sample loss in these kinds of soils. To retrieve undisturbed samples of these extremely loose soils, increasing the pore water viscosity by cooling it without actual freezing before sampling has been suggested. When undisturbed sampling is performed in test pits or other accessible excavation, the samples should be under capillary pressure so they acquire some apparent cohesion. This condition is attained by lowering the groundwater level below the bottom of the samples to be taken. Depending on the attained density of the soil, a thin-walled drive sampler or advance trimming may be used to take the samples. Careful field handling and transportation of these samples from the field to the laboratory are critical to avoid shock and vibrations, and preserve sample quality.

4.4.5 Compact or Stiff and Brittle Soils

The soils in this category include very dense sand; very stiff and brittle clays, and clayey, silty, and sandy soils that have been strongly compacted or have acquired considerable cohesion due to partial drying.

Bore hole stabilization with drilling fluid or mud is appropriate and casing can be used if required. Only viscous drilling fluid should be used, never clear water. When undisturbed samples of partially saturated soils or dried soils are to be taken, the bore hole should be kept as dry as possible. Open drive samplers and piston samplers can be used to obtain samples, but hammering or large pressures may be needed to thrust the sampler into the soil, especially when dense deep sands are present. Permeable and partially saturated soils may be compacted by this action and brittle soil samples can show shear failure disturbance. Core boring may provide less disturbed samples when the soil is relatively impervious and viscous drilling fluid and/or foam is used. Auger core barrels are used in dry bore holes to avoid drilling fluid contamination of permeable, partially saturated or dry soil samples. Cohesive soils can be retrieved easily but sand samples require the use of core retainers to prevent loss, especially in loose sand or silt. Advance trimming, box sampling or auger core boring should be used when obtaining undisturbed samples in test pits or other accessible excavations. Thin-walled drive samplers can be used to determine the natural density of the soil but not accurately when the soil is permeable or not fully saturated.

4.4.6 Hard and Partially Cemented Soils

This category includes hard and brittle clays, partially cemented soils, such as marl and hardpan, and highly compacted glacial till without significant amounts of gravel and stones.

Rotary or percussion drilling are generally necessary to advance a borehole in this material. Drilling fluid may be used to avoid gradual squeezing of deep bore holes. Drive samplers with thick walls can be hammered into these soils to obtain disturbed samples but the maximum penetration depth is usually small. Core boring will provide longer, less disturbed samples and, when sampling in accessible areas, box sampling methods are recommended.

4.4.7 Gravelly Soils

Representative samples of coarse-grained soils that include sands, gravels, and mixed soils can be extremely difficult to sample for laboratory analysis. Some may be impossible to sample since the sampler and the soils can both be destroyed or the soils completely disturbed by sampling operations. For these kinds of soils, in situ geophysical techniques and dynamic penetration techniques are potential alternatives. The Becker hammer drilling technique is specifically designed to provide a highly disturbed sample of coarse grained materials. The Becker hammer drill, marketed by the Layne Christensen Company of Mission Woods, Kansas, is a reverse circulation hammer drill that uses air and a double-walled drill casing to bring cuttings to a cyclone separator at the ground surface. The casing is driven using a double acting diesel pile driver and a toothed cutting bit. Driven with a plugged bit, the Becker hammer drill accommodates a penetration test to determine in situ strength in large-particled soils. The penetration test is discussed in a later section.

Undisturbed samples of these materials may be obtained by freezing the soil beneath the bottom of the borehole and then core boring. Techniques are also being developed whereby a larger volume of gravelly soil is frozen using a single probe and the resulting column or bulb of soil is removed, with the probe inside, to the ground surface for subsampling.

4.4.8 Tills

Gravel deposits and soils containing considerable amount of gravel and stones like glacial till are included in this category.

In these soils bore hole advance is slow and difficult, wash boring cannot be used and rotary drilling is inefficient. Borings may be advanced by displacement boring or with augers in loose gravelly soils. Compact deposits may be loosened with precision drilling and removed with bailers, sand pumps, or barrel augers. In stony soils, heavy-walled drive samplers are driven into the soil by a drop hammer, often damaging the cutting edge. Core boring cannot be used to sample loose gravel but it is an alternative for compacted or stony soils with some cohesive material. In both techniques the sample can be significantly disturbed since larger rocks get pushed either aside or into the sample. The Becker hammer drill, driven with open casing, is a good technique to obtain disturbed samples of coarse gravels; representative samples of clean gravels may also be obtained with barrel augers.

As is the case with gravelly soils, undisturbed samples of these materials may also be obtained by freezing the soil beneath the bottom of the borehole and then core boring.

4.4.9 Materials Unsuitable for Construction

Unsuitable materials generally need to be removed, wasted or greatly modified for construction due to undesirable characteristics. Some materials unsuitable for construction are: organic soil, peat, collapsible soils, clays with high swell and shrinkage potential, and sensitive or soft clays. Loess can be unsuitable for construction under certain conditions, but it has been used successfully as construction fill when properly remolded (Krinitzsky and Turnbull, 1967). Unsuitable soils become undesirable because of their low strength, low compressibility, low permeability, stickiness in construction, etc. These kinds of materials should be identified in the field, sampled, and tested, to establish their vertical and lateral extent and the specific characteristics of the material. The boundaries of unsuitable material should be determined with the appropriate accuracy and precision.

4.4.10 Collapsible Soils

Collapsible soils are those that suffer a sudden volume reduction when they become saturated and load is applied. Their soil structure is characterized by high porosity and low density. This includes soils composed of silt, fine sand-size particles and smectite (clay soils that can hold a quantity of water greater that the liquid limit). An in situ technique using a four-ton hydraulic jack with an attached microwave drying system was developed for collapsible soils as part of a Ph.D. dissertation (Bowers, 1986). The technique was successful in detecting collapsible soils and predicting the magnitude of additional settlement due to moisture increases in a relatively simple and reliable way.

The behavior of collapsible gravels is controlled by the fine matrix in the gravel. Small quantities of fines (as little as 5 to 20 percent) are enough to control collapse behavior. Relatively successful in situ methods for evaluating the properties of collapsible gravels include plate-load bearing tests and prebored pressuremeter tests. Both methods are conducted in the natural dry conditions and then after wetting.

Traditional investigative methods like the SPT can exhibit high blowcounts in gravelly soils, but once moisture is introduced or increased in the area the stiffness of the material may decrease drastically. Samples that are collected and reconstituted in the laboratory produce lower undrained cyclic stress results than measured from in situ frozen samples, indicating a change in the natural soil characteristics after reconstitution. Sampling by in situ freezing may be an alternative to sample these kinds of soils and transport them to the laboratory without disturbance.

5 GEOPHYSICAL INVESTIGATIONS

The three primary objectives in the use of geophysical methods are the location and correlation of geologic features, such as stratigraphy, lithology, discontinuities, groundwater; the in situ measurement of elastic moduli and densities, and detection of hidden cultural features. Perhaps the best reference available on theory and applications for a wide variety of geophysical survey techniques is EM 1110-1-1802, Geophysical Exploration for Engineering and Environmental Investigations (DA, 1995). The information presented in that manual is thorough and practical, and is useful to any individual or firm planning site reconnaissance or detailed geophysical characterization. The following discussion is general in nature; the reader is directed to EM 1110-1-1802 for specific guidance on contemporary geophysical survey practice. Internet users may find and download various manuals and other Corps of Engineers regulations, circulars, and Engineering technical letters at http://www.wes.army.mil/itl/cwgup/.

In locating and correlating geologic features, indirect geophysical techniques are intended to supplement direct methods insofar as is practical. There is no substitute for a direct assessment of site conditions, i.e., borings, pits, trenches, etc. Borings or test excavations should be used in conjunction with geophysical interpretations (or if necessary to calibrate or correct them). If used in this manner, geophysical methods offer both economic advantages and the ability to rapidly explore large subsurface volumes with adequate accuracy. By judicious planning, the number of borings required for subsurface definition can be greatly reduced if the proper geophysical methods are chosen to supplement the direct investigational program (Ballard and McLean 1975).

Geophysical methods may involve measurement of: seismic wave amplitude as a function of time, electrical conductivity or resistivity, electromagnetic radiation, radioactive radiation, magnetic flux density, and gravitational pull. These measurements can be obtained either by passive or by active techniques. Active techniques give more accurate information since the direct measurement or response of the subsurface materials to an introduced energy is collected. Passive measurements record the strength of various fields or changes in field strength which are always present, like gravity or magnetic fields. Assumptions are necessary for the interpretation of passive techniques and these assumptions can introduce ambiguity.

Geophysical survey methods can be divided in non-contacting techniques, surface techniques, and downhole survey methods. Non-contacting techniques are generally less expensive, faster, and easier to use. Some times these surveys can even be performed by a single person and provide variability information on shallow rock or soil deposits. Non-contacting methods include techniques like ground conductivity, magnetic techniques, and GPR (ground penetrating radar).

A single geophysical method rarely provides enough information about subsurface conditions to be used alone. Each method typically responds to several different physical characteristics of earth materials, and correlation of data from different methods has been found to provide the most meaningful results. Some degree of redundancy must be built into any well-planned site investigation. Certain methods may be useful under one set of subsurface conditions but offer little or no information in others. As with direct methods, planning, data acquisition, and interpretation phases of any investigational program using indirect methods are best performed by qualified personnel.

In situ values of the elastic moduli and densities are required to solve many foundation and soil-structure interaction problems. These values are particularly important for wave propagation analyses and where soil-structure interaction is a dominant factor in the response of a structure to dynamic loadings. Foundations for reactor containment vessels, subject to earthquake excitation, are examples of applications where in situ elastic moduli are sometimes required for design or safety analyses.

As a background for the use of geophysical methods, four general observations pertaining to these methods collectively should be made as listed in Schmidt et al., (1976):

> "(1) Geophysical surveys utilize both active and passive measurement techniques. In an active mode, some form of energy is introduced into the subsurface and the effect on the energy or the response of subsurface materials to energization is measured. Active measurement techniques usually provide the greatest accuracy. Passive measurements simply record the strengths of various fields or changes in field strength which are always present. Analytical assumptions that introduce ambiguity in the results are necessary for interpretation.
>
> "(2) Precision of measurements is high in all methods, but accuracy in the interpretation and inferences drawn from the measurements depends very much on the experience of the interpreter. All methods are inherently subject to lower accuracy due to interpretation as distance increases between the energy or field source of interest and the detecting sensors, especially in those methods based on field strength measurements (passive mode).
>
> "(3) Resolution capability of subsurface characteristics varies widely among geophysical methods when surveys are conducted conventionally. The parameters to be measured or inferred must be understood before a resolution dimension can be defined. Almost total resolution of any soil or strata parameter is possible if the survey is appropriately designed and time/cost requirements are not considered. One reasonable approximation is selection of measurement point separation on the order of the dimension of objects or strata changes to be resolved.
>
> "(4) Very few geophysical methods measure parameters directly used by the engineer (seismic and electrical methods may be exceptions), and all methods present the "averaged" effects of materials between and around the source and points of observation. Most results are based on interpretations that infer what kind of conditions would cause the measured parameter to have a certain value or to change in a certain way"

The interpretation of geophysical data is based on the assumption that the various earth materials have distinct subsurface boundaries and are both homogeneous and isotropic. These assumptions are in many cases at variance with reality. In addition, the practice of computing shear (secondary, or s-) wave velocity from measured compression (primary, or p-) wave velocity through the assumption of a Poisson's ratio is not recommended. Borehole seismic surveys accommodate procedures to induce polarized shear waves that may be readily interpreted in motion records; surface refraction is generally performed using an explosive charge or an impact source, such as a hammer blow, to generate elastic waves in the ground. It is often difficult to distinguish shear waves recorded in the latter case, thus surface refraction is not considered the best method for shear wave velocity measurement.

Table 4 is intended for use as a quick reference tool for the planners of foundation investigations. It is a tabulation of geophysical methods, their basic measurement parameters, engineering applications, advantages and primary limitations.

To further simplify the selection of a geophysical method to fit a given engineering application, Table 5 can be used as an aid in choosing the most effective geophysical method(s) to fit a specific engineering application. The matrix in the table is constructed in such a manner as to provide a subjective numerical rating system for the effectiveness of each method for a particular rating system. The five numerical ratings of 0, 1, 2, 3, and 4, which are explained in the note in Table 5, can then be used as an aid for planning an effective and economical site investigation. A rating of 4 for the combination of a given method indicates that the method is well developed, practical for use, and likely to give good results.

5.1 Spectral Analysis of Surface Waves

The Spectral Analysis of Surface Waves technique is widely applied to determine shear wave velocity profiles in a variety of applications (Stokoe and Nazarian, 1985; Stokoe et al., 1988). The SASW method is used for in situ evaluation of elastic moduli and layer thicknesses of soil and pavement profiles (Gucunski and Woods, 1991) and for large areal coverage in terrain with minimal relief. The technique has the advantage in that it is performed from the surface, and a simple procedure and test setup are required.

The SASW method consists of several phases: 1. collection of data in the field, 2. evaluation of the field or experimental Raleigh wave dispersion curve, and 3. inversion of the dispersion curve to obtain the shear wave velocity profile. The common receivers midpoint (CRMP) geometry and the common source (CS) geometry are the two most common source-receivers setups for SASW practice. In the CRMP method, the source and the receiver are located at the same distance from an imaginary line at opposite sides and in CS the source has a fixed location.

The type of energy source and motion transducer needed for the SASW will depend on the objective for which they are deployed. Layered deep soil deposits require large, low frequency energy sources (such as heavy vibrators) and sensitive, low frequency transducers; small, impact-type energy sources and high-frequency motion detectors would be used for investigations of layered pavements.

As the SASW technique has evolved through time, additional options have become available to collect and process the substantial quantities of test data that are typically generated. The original approach was based on data collection with a full-function, dual channel Fast Fourier Transform (FFT) analyzer. A computer was then used to compute a field dispersion curve from the phase data of the cross power spectrum and perform the required inversion process. Several software/hardware packages are now available for SASW applications. Commercial waveform processing software can be used with a PC to replace the FFT analyzer. Transient waveform digital recorders can be used to gather, digitize and store SASW field data and the data may be transferred later to a PC with the necessary software to finish the dispersion and inversion operations of the FFT. These packages are compatible with IBM or Apple Macintosh PC's and perform all functions including data acquisition and waveform processing. The user may more customize an approach and upgrade capabilities as more software versions become available. SASW techniques have advanced from using dual to multi-channel analyzers of up to 64 channels and equipment will likely accommodate more in the future.

Table 4 Applications of Selected Geophysical Methods for Determination of Engineering Parameters

Geophysical 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	Rapid, accurate, and relatively economical technique. Interpretation theory generally straightforward and equipment readily available. In saturated soils, the compression wave velocity reflects mostly wave velocities in the water, and thus is not indicative of soil properties.
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	Rapid, thorough coverage of given site area. Data displays highly effective. In saturated soils, the compression wave velocity reflects mostly wave velocities in the water, and thus is not indicative of soil properties.
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	Rapid technique which uses conventional refraction seismographs
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	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
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 is 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

(Continued)

Table 4 (Continued)

Geophysical 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, groundwater, 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, i.e., 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	Must have access to some cavity opening. Still in experimental stage - limits not fully established
Ground penetrating radar(GPR)	Travel time and amplitude of a reflected electromagnetic wave	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. On line digital data processing can yield "on site" 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	Provided extreme care is exercised in establishing gravitational references, reasonably accurate results can be obtained	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 or ferrous 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 ferrous 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

(Continued)

Table 4 (Continued)

Geophysical 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 sub-surface 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. Shell's law of refraction must be applied to establish zoning. A borehole deviation survey must be run. Requires highly experienced personnel. Repeatable source required.
Borehole spontaneous potential	Natural earth potential	Correlates deposits, locates water resources, studies rock deformation, assesses permeability, and determines groundwater 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.

(Continued)

Table 4 (Continued)

Geophysical 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, 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, 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 3 diameters

(Continued)

Table 4 (Concluded)

Geophysical Method	Basic Measurement	Application	Advantages	Limitations
		Borehole (Continued)		
Acoustic caliper	Sonic ranging	Measures borehole diameter	Large range. Useful with highly irregular shapes	Requires fluid filled hole and accurate positioning
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 groundwater
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	Interpretation is simple	Impeller flowmeters usually cannot measure flows less than 2 - 3 ft/min
Borehole dipmeter	Sidewall resistivity	Provides strike and dip of bedding planes. Also used for fracture detection	Useful in determining information on the location and orientation of primary sedimentary structures over a wide variety of hole conditions	Expensive log to make. Computer analysis of information needed for maximum benefit
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 groundwater flow	A reliable, cost effective method to determine lateral foundation leakage under concrete structures	Assumes flow not influenced by emplacement of borehole.

Table 5 Numerical Rating of Geophysical Methods to Provide Specific Engineering Parameters[a]

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
Surface																										
Refraction (seismic)	4	4	2	2	4	2	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	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)	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
Ground penetrating radar[b]	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	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[b]	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
Borehole																										
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
Crosshole acoustic	4	4	4	4	4	0	1	3	4	0	0	0	1	0	3	0	0	0	0		3	3	3	3	0	0
Crosshole resistivity	3	0	0	0	0	0	1	3	1	0	0	0	1	0	3	0	3	0	0		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

(Continued)

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[a] 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

[b] Airborne or inhole survey capability not considered.

Table 5 (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
Borehole (Continued)																										
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	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 (3D) 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	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	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	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	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	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	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	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

[a] 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

[b] Airborne or inhole survey capability not considered.

5.2 Measurement of Strain-dependent Properties

Most of the geophysical survey techniques discussed or presented in tables thus far measure in situ values of low-strain dynamic response characteristics, such as modulus and damping, within a localized region around the energy source. Laboratory tests on undisturbed or reconstituted specimens have been used to determine strain-dependency of various parameters in controlled environments, and these trends are typically extrapolated to predict in situ response in natural deposits. For critical projects, such as nuclear power facility foundations, sophisticated numerical analyses may be applied and it may be justifiable to determine variability of dynamic response parameters over larger volumes of material that better represent in situ behavior. Large-scale in situ dynamic testing techniques have been developed that capture both low- and large-strain dynamic response data; two of the more well-developed are: the in situ impulse test (Troncoso, 1975) and the cylindrical in situ test (CIST) (Wilson, Brown, and Schwarz, 1978). Both are crosshole geophysical methods, requiring installation and instrumentation of several boreholes with the concomitant expense.

The in situ impulse test involves generation of controlled shear waves within a source borehole and several receiver boreholes spaced to take advantage of decreased strain with distance from the source. The method is capable of providing shear wave velocities over a range of shear strains from 10^{-4} to 10^{-1} percent. The energy source consists of a guided hammer which is dropped 30 cm onto an anchor that is clamped to couple the energy into the wall of the borehole; a spring receives the hammer blow and shapes the input shear wave (Troncoso, 1975). The method is reported to be effective to depths up to 200 feet (61 m) in most soils. Recent study of this procedure, popularly known as the large-strain seismic cross-hole test, is detailed by Salgado et al. (1997). This reference includes discussion of proper interpretation techniques, as well as a case history of its application at a Department of Energy site for determination of site-specific shear modulus degradation relationships. Low-strain results are favorably compared with those obtained using shear wave velocity measurements from seismic cone penetrometer tests.

An alternative method to the in situ impulse test, the cylindrical in situ test, was first developed to investigate soil response to blast loads. The main difference between the two large-scale in situ tests is the energy source; the CIST source is a continuous, vertical line charge of high explosive installed central to the array of radially spaced receiver holes. The charge may be detonated in a delayed, controlled fashion to invoke other responses besides that associated with blast effects, and numerical models of the site in question may be accurately calibrated by matching received and predicted response (Wilson, Brown, and Schwarz, 1978).

6. IN SITU PHYSICAL TESTING

6.1 In situ Tests - General

In situ tests are often the best means of determining the engineering properties of subsurface materials and, in some cases, may be the only way to obtain meaningful results. Some materials are hard to sample and transport, while keeping them representative of field conditions, because of their softness, their lack of cohesion, or their composition. In situ techniques offer an option for appropriate evaluation of soils and rock that cannot be sampled for laboratory analysis. Table 6 lists several of the most commonly practiced in situ tests and their purposes. Several techniques will be discussed in some detail below.

Table 6 In Situ Tests for Rock and Soil (adapted from EM 1110-1-1804, Department of the Army, 1984)

		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[a]
	Borehole direct shear[b]	X	
	Pressuremeter[b]		X
	Uniaxial compressive[b]		X
	Borehole jacking[b]		X
Bearing capacity	Plate bearing	X	X[a]
	Standard penetration	X	
Stress conditions	Hydraulic fracturing	X	X
	Pressuremeter	X	X[a]
	Overcoring		X
	Flatjack		X
	Uniaxial (tunnel) jacking	X	X
	Borehole jacking[b]		X
	Chamber (gallery) pressure[b]		X
Mass deformability	Geophysical (refraction)	X	X
	Pressuremeter or dilatometer	X	X[a]
	Plate bearing	X	X
	Standard penetration	X	
	Uniaxial (tunnel) jacking	X	X
	Borehole jacking[b]		X
	Chamber (gallery) pressure[b]		X
Relative density	Standard penetration	X	
	In situ sampling	X	
Liquefaction susceptibility	Standard penetration	X	
	Cone penetrometer test (CPT)	X	
	Shear wave velocity (v_s)		

[a] Primarily for clay shales, badly decomposed, or moderately soft rocks, and rock with soft seams.

[b] Less frequently used.

Interpretation of in situ test results in soils, clay shales, and moisture-sensitive rocks requires consideration of the drainage that may occur during the test. Consolidation during soil testing makes it difficult to determine whether the results correspond to unconsolidated- undrained, consolidated-undrained, consolidated-drained conditions, or intermediate conditions between these limiting states. Interpretation of in situ test results requires complete evaluation of the test conditions and the limitations of the test procedure.

Rock formations are generally separated by natural joints and bedding planes (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. In situ rock tests are performed to determine in situ stresses and deformation properties 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. In situ testing performed in weak, near surface rocks include: penetration testing, plate loading testing, field geophysical techniques, and pressuremeter testing.

Table 7 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 (1983) and Bowles (1996). Direct shear strength tests in rock measure peak and residual direct shear strength as a function of normal stress on the shear plane. Direct shear strength from intact rock can be measured in the laboratory if the specimen can be cut and transported without disturbance. The suggested in situ method for determining direct shear strength of rocks (test preparation, consolidation, shearing and calculation procedures) is described in the RTH 321-80, “Suggested Method for In Situ Determination of Direct Shear Strength,” (Handbook).

6.2 In Situ Soil Testing

6.2.1 The Standard Penetration Test (SPT)

The standard penetration test (Figure 9) is a soil index test used to obtain the resistance to penetration by counting the number of blows required to drive a steel tube of specific dimensions into the subsoil a specified distance using a hammer of a specified weight (EM 1110-1-1906, DA, 1996). The SPT can be used in relatively clean medium -to-coarse sands and fine gravels with a variable amount of moisture, and in saturated or nearly saturated cohesive soils. Non-saturated cohesive soils and saturated silty sands can produce misleading N-values (the standard penetration resistance, or number of blows required to penetrate the final 12 inches of an 18-inch drive). Typical SPT testing and sampling intervals are of 0.6 to 1.5 m (2 to 5 feet) in uniform strata and whenever there is a change of strata. Control of the hammer drop can be by manual, automatic, semiautomatic, or trip-hammer drop systems.

Borehole disturbance can affect SPT results dramatically, and may reduce N values to a fraction of the value that would be obtained in undisturbed ground. SPT blow count data can also be affected by the penetration interval, the sampling tube design, the number of turns of rope on the cathead winch used to

Table 7 In Situ Tests to Determine Shear Strength (adapted from EM 1110-1-1804, Department of the Army, 1984)

Test	For		Reference	Remarks
	Soils	Rocks		
Standard penetration	X		EM 1110-2-1906 Appendix B	Use as index test only for strength. Develop local correlations. Unconfined compressive strength in tsf is often 1/6 to 1/8 of N-value
Direct shear	X	X	RTH[a] 321-80	Expensive; use when representative undisturbed samples cannot be obtained
Field vane shear	X		ASTM D 2573-94	Use strength reduction factor
Plate bearing	X	X	ASTM[b] Designation D 1194 ASTM SPT 479[c]	Evaluate consolidation effects that may occur during test
Uniaxial compression		X	RTH[a] 324	Primarily for weak rock; expensive since several sizes of specimens must be tested
Cone penetrometer test (CPT)	X		Schmertmann (1978); Jamiolkowski et al. (1982)	Consolidated undrained strength of clays; requires estimate of bearing factor, N_c

[a] Rock Testing Handbook.

[b] American Society for Testing and Materials.

[c] Special Technical Publication 479.

raise the safety hammer between blows, variations on drop height, energy delivered to the sample tube , or split-spoon, in situ stress conditions, faulty equipment, and overboring. The suggested approach is to follow standard procedures, to be aware of the potential errors and error sources that can affect the technique, and take errors into account when collecting and interpreting SPT data. EM 1110-1-1906 (DA, 1996) describes the STP method and suggests modifications to the interpretation procedure.

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) and the unconfined compressive strength (Mitchell, Guzikowski, and Villet, 1978) of soils. The split-spoon sampler driven during an SPT recovers disturbed soil that may be used for visual classification and laboratory index testing, including determination of natural water content. Longer and larger diameter samplers are commercially available for recovery of coarse-grained soils.

6.2.2 Becker Penetration Test (BPT)

The Becker hammer drill (Harder and Seed, 1986) is essentially a double-acting diesel pile driver specifically designed for penetration of frozen or large-particled soils. The operation of the drill has been adapted to perform an in situ penetration test that has been compared (by empirical correlation of blowcounts, not by physical procedures and test conditions) to the SPT in sandy soils. When performed using double-walled casing and reverse circulation by air, the Becker hammer drill produces highly disturbed samples of subsurface materials by means of a cyclone separator. The Becker Penetration Test (BPT) is not a standard test but is an alternative method to drill and sample in gravelly, bouldery soils and obtain their engineering characteristics. The BPT is not listed in the summary tables, since it is necessary to empirically convert BPT blow counts (obtained per foot of continuous penetration using a plugged casing) to equivalent SPT blow counts for correlation to field performance data on liquefaction resistance and residual shear strength. Limitations include sample disturbance and possible significant alteration of the in situ stress conditions during drilling. Appendix H of EM 1110-1-1804 (DA, 1995) describes the application of the Becker Penetration Test.

6.2.3 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 direct shear test measures peak and residual strength as a function of stress normal to the shear plane. Results are usually employed in limit equilibrium analysis of slope stability problems or for stability analysis of foundations for large structures such as dams. In situ direct shear tests may be necessary 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. Few in situ direct shear tests are performed on soils; they may be justified in clay shales, very soft rock, and on thin, continuous, weak seams that are difficult to sample. Methods for performing in situ strength tests on rock are described in Zeigler (1976) and Das (1994).

6.2.4 Field Vane Shear Tests

The field vane test is an in situ test used to measure the undrained shear strength of clays. The technique was developed to avoid sample disturbance effects common to soft materials by performing the test in the field. The vane, which consists of four blades in a cross shape, is pushed into the soil to the desired depth for testing and the torque needed to rotate it is measured. The shear strength of the soil is calculated from the torque and the geometry of the vane. A review of field vane testing is given by Chandler (1988). The test can be performed in the field either from the ground surface or from the bottom of a borehole following the ASTM D 2573-94, "Standard test method for field vane shear tet in cohesive soil," protocol or, at a reduced scale, in the laboratory. The test may be affected by such factors as soil type, permeability, strength anisotropy, disturbance when inserting the vane, amount of time between the insertion of the vane and the beginning of the test, the rate of rotation or strain, and failure of soil around the vane. Technique and equipment improvements have developed correction factors or equipment modifications to deal with many of the above issues (Merrifield, 1980). Field vane tests performed in boreholes can be useful in soft, sensitive clays that are difficult to sample. However, they may give results that indicate strengths that are too high. Factors to correct the results are discussed in Mitchell, Guzikowski, and Villet (1978). The test has been standardized by ASTM as D 2573.

6.2.5 Plate Bearing Tests

Plate bearing tests can be made on soil or soft rock. Because of their cost, such tests are normally performed during advanced design studies or during construction. They are used to determine subgrade moduli and occasionally to determine strength. Plate loading tests are used to determine the stiffness, compressibility, and other properties of hard-to-sample soils, granular soils, and fractured weak rocks. The test is performed by applying successive load increments until the soil or rock fails or until the soil resists several times the proposed designed load. When testing shear strength or bearing capacity of cohesive soils, the plate test is performed by applying a constant load to give a constant rate of penetration from which the undrained shear strength is calculated. Small downhole plate-loading tests can also be performed (Wrench, 1984). The need for reliable engineering properties of rocks and the lack of other methods to obtain them make plate testing for rocks an efficient and cost-effective approach. It is recommended that they be performed at the end of a site characterization study or as part of a supplementary study. The location and number of tests will depend on the soil or rock mass variability and properties as determined by other tests and visual observations.

6.2.6 Pressuremeter Test

The pressuremeter test can be used in all types of soil and weak rocks to measure strength and compressibility parameters. The pressuremeter test is used in overconsolidated clays to measure undrained strength, shear modulus, and coefficient of earth pressure at rest, K_o. Pressuremeters designed for higher pressure application in hard soils or rocks can be called dilatometers. The pressuremeter derives information on stiffness and strength of weak materials by measuring the relationship between applied radial pressure and the resulting deformation. The pressuremeter test may be used to infer parameters such as undrained shear strength (Marchetti, 1980), unit weight (Marchetti and Crapps, 1981), effective angle of friction (Marchetti, 1980), drained constrained modulus (Marchetti, 1980), elastic modulus, and the very small-strain shear modulus, G_{max}. Determination of in situ pore pressure and horizontal stress can be obtained in sands (Lutenegger and Kabir, 1988; and Robertson, Campanella, and Gillespie, 1988), in normally consolidated young clays (Lunne et al., 1990), in soft and medium to stiff clays (Briaud and Miran, 1992)

6.2.6.1 Borehole Pressuremeter. The borehole pressuremeter can be used in cemented very hard soils, gravelly soils, or rocks. The borehole pressuremeter is lowered into a smooth-walled borehole to the desired position and then inflated. The radial displacement is then measured either by a fluid-filled measuring cell or using gas (e.g., Oyo Elastmeter 2™) and electronically measuring the displacement with two opposing measuring arms.

6.2.6.2 Self-boring Pressuremeter. This pressuremeter is used only for soft soils and cannot be used in cemented very hard soils, gravelly soils, or rocks. This pressuremeter includes an internal cutting mechanism at its base and is pushed hydraulically. Cuttings are flushed by fluid through the center of the probe as it advances. There are several versions of this penetrometer: The Cambridge self-boring penetrometer, the Rock self-boring penetrometer (Clarke and Allan, 1989), and the French Pressiometre Autoforeur (PAF) (Baguelin et al., 1972).

6.2.6.3 Displacement Pressuremeter. Two models of displacement pressuremeters are available: the push-in pressuremeter (PIP), mainly used in offshore investigations (Henderson, Smith, and St. John, 1979), and the cone-pressuremeter (Withers, Schaap, and Dalton, 1986) which is a displacement measuring mechanism mounted on a cone penetrometer, which will be described in a subsequent section.

6.2.6.4 Marchetti Dilatometer. As defined previously, the Marchetti dilatometer is a pressuremeter. This test (DMT) (Marchetti, 1980) is used to calculate horizontal stress, dilatometer modulus, and pore pressure indices of preferably normally consolidated clays and uncemmented sands. The Marchetti blade can be pushed from the surface without a borehole, avoiding drilling disturbance. A DMT sounding can be performed in a short time and its results can be processed on a portable computer in the field.

6.2.6.5 Cone Penetrometer Test. The Cone Penetration Test (CPT) is an in-situ testing method for evaluating detailed soil stratigraphy as well as estimating geotechnical engineering properties (Schmertmann, 1978). The basic, electrical CPT involves hydraulically pushing a 1.4-inch-diameter instrumented probe into the earth while performing two measurements: cone tip bearing resistance and sleeve friction resistance. The cone penetrometer test (CPT) is very useful for detecting soft or weak layers and in quantifying undrained strength trends with depth. The probe is normally pushed from a special heavy duty truck but can also be performed from a trailer or drill rig. Because of the weight of the truck or trailer needed to conduct CPT borings, access to soft ground sites may require a special , all-terrain tracked vehicle. Olsen (1994) gives a comprehensive description of the CPT in its various forms and applications.

Soil type may be determined by empirical correlation to CPT penetration and frictional resistances (Douglas and Olsen, 1981), which should be supported by direct evidence from adjacent borings. Once soil type is known, 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, Guzikowski, and Villet, 1978; Mitchell and Lunne, 1978; Schmertmann, 1978). For clays, a bearing factor, N_c, must be estimated in order to calculate the undrained strength from the cone resistance and should be close or slightly greater than the 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 be used by means of CPT techniques to estimate the overconsolidation ratio (OCR) (Schmertmann, 1978). 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). Empirical charts for estimating seismically induced pore pressures (liquefaction) have been developed based on cone penetration measurements performed at sites at which earthquakes have occurred. These charts are similar to those developed for SPT blowcounts.

The basic Cone Penetrometer Test (CPT) consists of pushing a cone of specific characteristics and dimensions into the ground at a constant speed of approximately 2 cm/s and measuring the amount of force required for penetration due to end bearing and side friction forces. Cone resistance and side friction can be measured either by electrical or mechanical means. Originally used to locate and characterize sand layers in soft clays, CPT methods can be used to acquire detailed subsurface stratigraphic information, soil type information, pore pressure measurements, undrained strength for clays, and relative density for sands. Current standards and procedures are detailed in ASTM D 3441-94, "Standard test method for deep, quasi static, cone and friction-cone penetration test of soil."

The addition of piezocones to the CPT to measure pore water pressure has been of great utility, especially when characterizing soft cohesive deposits. By being able to measure pore pressure, the CPT can also determine the following:

- Profiling - location of thin granular layers in soft cohesive deposits, which is very important to determine the consolidation rate of those kinds of deposits.

- Soil type identification - The ratio between net cone resistance and the excess pore pressure, can be correlated to soil type.

- Determining static pore pressure - this can be accomplished in granular soils and estimated in clays by waiting for full dissipation of the excess pore pressure provoked by penetration.

- Determination of in situ consolidation characteristics - the horizontal coefficient of consolidation (C_v) in clays can be obtained by measuring pore pressure dissipation as a function of time after stopping the cone.

Specialized CPT equipment has also been developed for environmental site assessment, for example the Tri-Service (Army, Navy, Air Force) Site Characterization and Analysis Penetrometer System (SCAPS) (Koester et al., 1993), which is used to detect various underground contaminants. On-site 3-dimensional visualization of the subsurface stratigraphy and regions of potential contamination are quickly delineated using the SCAPS rigs with custom software and onboard computers.

6.3 In Situ Rock Testing

In situ stress conditions of rocks 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 from changes in loading brought about by excavation or construction. Where confinement of a 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 area, defined as 100 feet (30.48 m) or less, can be 1.5 to 3 times higher than the vertical stress. Recognition of this condition during the design phase of investigations is very critical. 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 8 lists the field tests that can be used to determine in situ stress conditions.

Table 8 In Situ Tests to Determine Stress Conditions (adapted from EM 1110-1-1804, Department of the Army, 1984)

Test	Soils	Rocks	Bibliographic Reference	Remarks
Hydraulic fracturing	X		Leach (1977) Mitchell et al. (1978)	Only for normally consolidated or slightly consolidated soils
Hydraulic fracturing		X	RTH[a] 344 Goodman (1981) Hamisen (1978)	Stress measurements in deep holes for tunnels
Vane shear	X		Blight (1974)	Only for recently compacted clays, silts and fine sands (see Blight, 1974, for details and limitations)
Overcoring techniques		X	RTH 341-80 Goodman (1981) Rocha (1970)	Usually limited to shallow depth in rock
Flatjacks	X		Deklotz and Boisen (1970) Goodman (1981)	
Uniaxial (tunnel) jacking	X	X	RTH 365-80	May be useful for measuring lateral stresses in clay shales and rocks, also in soils

[a] Rock Testing Handbook.

6.3.1 Residual Stress

6.3.1.1 Overcoring Method. Possibly the most common method used for measuring in situ stresses in rock for depths up to 150 feet or less is overcoring. An NW (3-1/2-inch diameter) core hole is drilled, instrumented with strain gages, 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 is then converted to stress by using the elastic modulus of the intact 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 given in the Rock Testing Handbook (RTH 341-80, "Determination of In Situ Stress by the Overcoring Technique," DA, 1993). The overcoring method is hampered by the presence of numerous instrument lead wires, which may be broken during testing. The practical maximum depth of testing is usually less than 150 feet. The three-component borehole deformation gauge method (Hooker and Bickel, 1974) is described in detail in the RTH 341-80.

6.3.1.2 Photoelastic Inclusion Method. Another approach is the photoelastic inclusion method. A photoelastic inclusion permits the measurement of the maximum and minimum stresses in the plane normal to the drillhole axis. This method is relatively inexpensive but can be complicated or erratic if rock anisotropy and deformations caused by fine material are not considered. The photoelastic inclusion method is described in the RTH 342-89, "Suggested Method for Determining Stress by a Photoelastic Inclusion," (DA, 1993).

6.3.1.3 Flatjack Method. In the flatjack method, two points are inscribed on the rock walls of a tunnel. A slot is bored or cut into the rock wall midway between the inscribed points. Stresses present in the rock will tend to partially close the slot. A hydraulic flatjack is then inserted in 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. The value recorded must be corrected for the influence of the tunnel excavation itself. Flatjack tests obviously need an excavation or tunnel for the test. 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. The flatjack tests can also be used as deformation tests and due to their versatility and size, can be adapted to a number of configurations. However, the jack tests only a small volume of rock, making it difficult to calculate deformation or failure properties.

6.3.1.4 Hydrofracture Method. The hydrofracture method has been used in soils and rock and theoretically has no depth limitation, even though it loses accuracy after 100 ft or more. To conduct the test, a section of hole is isolated with packers at depth and an increasingly higher water pressure applied to the section. A point will be reached where the water 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 of the crack 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 100 feet or more of the 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.

6.3.2 In situ Deformability Tests

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 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 non-homogeneity. 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 8 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 9). Deformation properties of a jointed rock mass are very important if highly concentrated loadings are anticipated and tolerance for displacement under final loading is low. 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., 2 to 3 feet). 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.

6.3.2.1 Chamber Tests. One of the limitations of in situ deformation testing for jointed rock masses is the necessity of representative sizes. Chamber tests can be performed to overcome this problem. This test determines the deformability of a rock mass by subjecting the cylindrical wall of a tunnel or chamber to hydraulic pressure and measuring the resultant rock displacements. With this data, the elastic or deformation moduli are calculated (RTH 361-89, "Suggested Method for Determining Rock Mass Deformability Using a Pressure Chamber," DA, 1993). Chamber tests are performed in large underground openings. Generally these openings are test excavations such as exploratory tunnels. Pre-existing openings, such as caves or mine chambers, can be used if available and applicable to project conditions, or a chamber may be excavated in the rock for this purpose. 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 computed.

6.3.2.2 Uniaxial Jacking Test. An alternative to the chamber tests is the tunnel, or uniaxial jacking test (Rock Testing Handbook, RTH 365-80). This test uses a set of diametrically opposed jacks to test large zones of rock in the radial direction from the axis of the tunnel. This method produces nearly comparable results with chamber tests without incurring the much greater expense. The test determines how foundation rock will react to controlled loading and unloading cycles and provides data on deformation moduli, creep, and rebound. The jacking test is the preferred method for determining deformation properties of rock masses for large projects. It is a more economical alternative to the chamber tests with comparable results (Figure 10). Information on site selection, preparation of test site,

Table 9 In Situ Tests to Determine Deformation Characteristics (adapted from EM 1110-1-1804, Department of the Army, 1984)

Test	For Soils	 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[a] 362-89 Baguelin, et al. (1972), Mitchell et al. (1978)	Consider test as possibly useful but not fully evaluated. For soils and soft rocks, shales, etc.
Chamber test	X	X	Hall et al. (1974) Stagg and Zienkiewicz (1968)	
Uniaxial (tunnel) jacking	X	X	RTH 365-80 Stagg and Żienkiewicz (1968)	
Flatjacking		X	Deklotz and Boisen (1970) Goodman (1981)	
Borehole jack or dilatometer		X	RTH 363-89 Stagg and Zienkiewicz (1968)	
Plate bearing		X	RTH 364-89 Stagg and Zienkiewicz (1968)	
Plate bearing	X		MIL-STD 621A, Method 104	
Standard penetration	X		Hall et al. (1974)	Correlation with static or effective shear modulus, in psi, of sands; settlement of footings on clay. Static shear modulus of sand is approximately: $G_{eff} = 1960N^{0.51}$ in psi; N is SPT value

[a] Rock Testing Handbook.

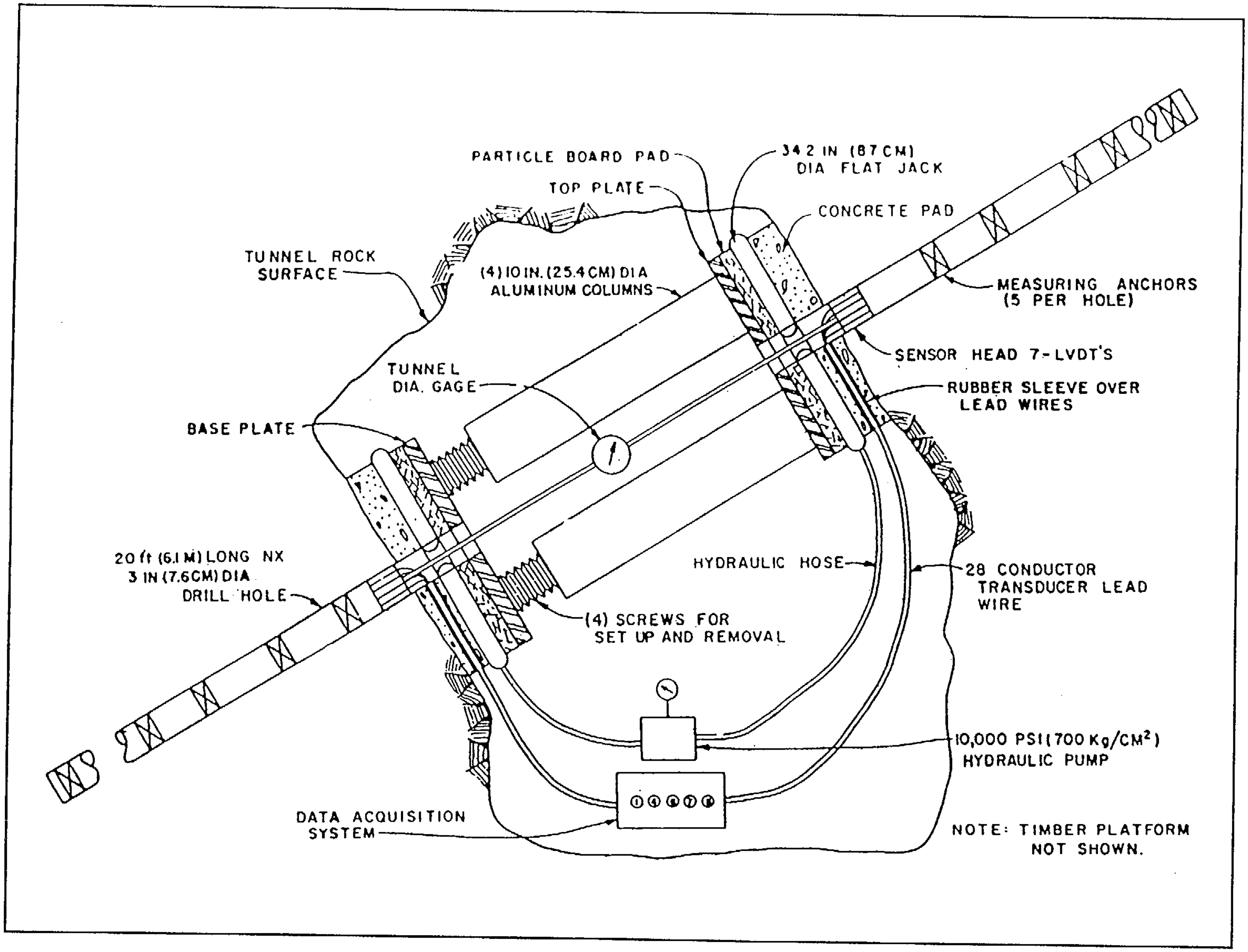


Figure 10. Uniaxial jacking test. (RTH)

equipment installation, testing and removal can be obtained in the RTH 365-80, "Bureau of Reclamation Procedures for Conducting Uniaxial Jacking Tests," (DA, 1993).

6.3.2.3 In Situ Uniaxial Compressive Test. This test is used to measure strength and deformability of large, in situ specimens of weak rocks such as coal. The strength of the rock mass depends on the sample size, so to find the value of engineering significance, samples of increasing size are tested until an asymptotic constant strength value is obtained (Bieniawski and Van Heerden, 1975). A block specimen is cut from the site to be tested and loaded until failure occurs. Preferably, a deformation-controlled loading system should be used. Details in testing arrangements, preparation, and calculations procedures are described in the RTH 324-80, "Suggested Method for Deformability and Strength Determination Using an In Situ Uniaxial Compressive Test."

6.3.2.4 Other Deformation Tests. Other methods for measuring deformation properties of in situ rock are anchored cable pull tests, flatjack tests, borehole jacking tests and tunnel 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 difficulty in defining a model for calculation of deformation or failure parameters. The borehole jack ("Goodman" jack) or dilatometer has the primary advantage that direct access to the rock face is not required. The development of a mathematical model for the methods, however, has proven to be more difficult than with most deformation measurement techniques.

Radial tunnel jacking tests 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. The small downhole plate-loading tests (Wrench, 1984) can also be used to determine deformability characteristics of a rock mass. The test is performed by loading a flat surface at the end of a drill hole or other recess and measuring the resultant displacement of that surface. Elastic or deformation moduli are calculated, as well as time dependent (creep) properties (RTH 364-89, "Method for Determining Rock Mass Deformability by Loading a Recessed Circular Plate"). (Figures 11 and 12).

Pressuremeter tests in soft rock for deformability consist of lowering an inflatable cylindrical probe into a predrilled borehole, expanding the probe laterally against the borehole wall, and recording the increase in size of the probe and associated pressure within the probe (RTH 362-89, "Pressuremeter Tests in Soft Rock"). The test can be performed at any desired depth and is terminated if yielding in the rock becomes large. From the recorded data, a pressure-volume curve is plotted and a pressuremeter modulus is calculated. The calibration of the probe, procedures, and calculations are described in RTH 362-89.

The point load strength test is a field or laboratory test for intact strength, an index test for strength classification of rock material. The test can be performed either in the laboratory or in the field with portable equipment, using core samples (diametral and axial tests), cut blocks (block tests), or irregular lumps (irregular lump test) (Haramy, Morgan, and De Waele, 1981). The test consists of a loading system, a loading measuring system, and a distance measuring system. Details of specimen selection, preparation, equipment calibration, and the different tests is available in the RTH 325-89.

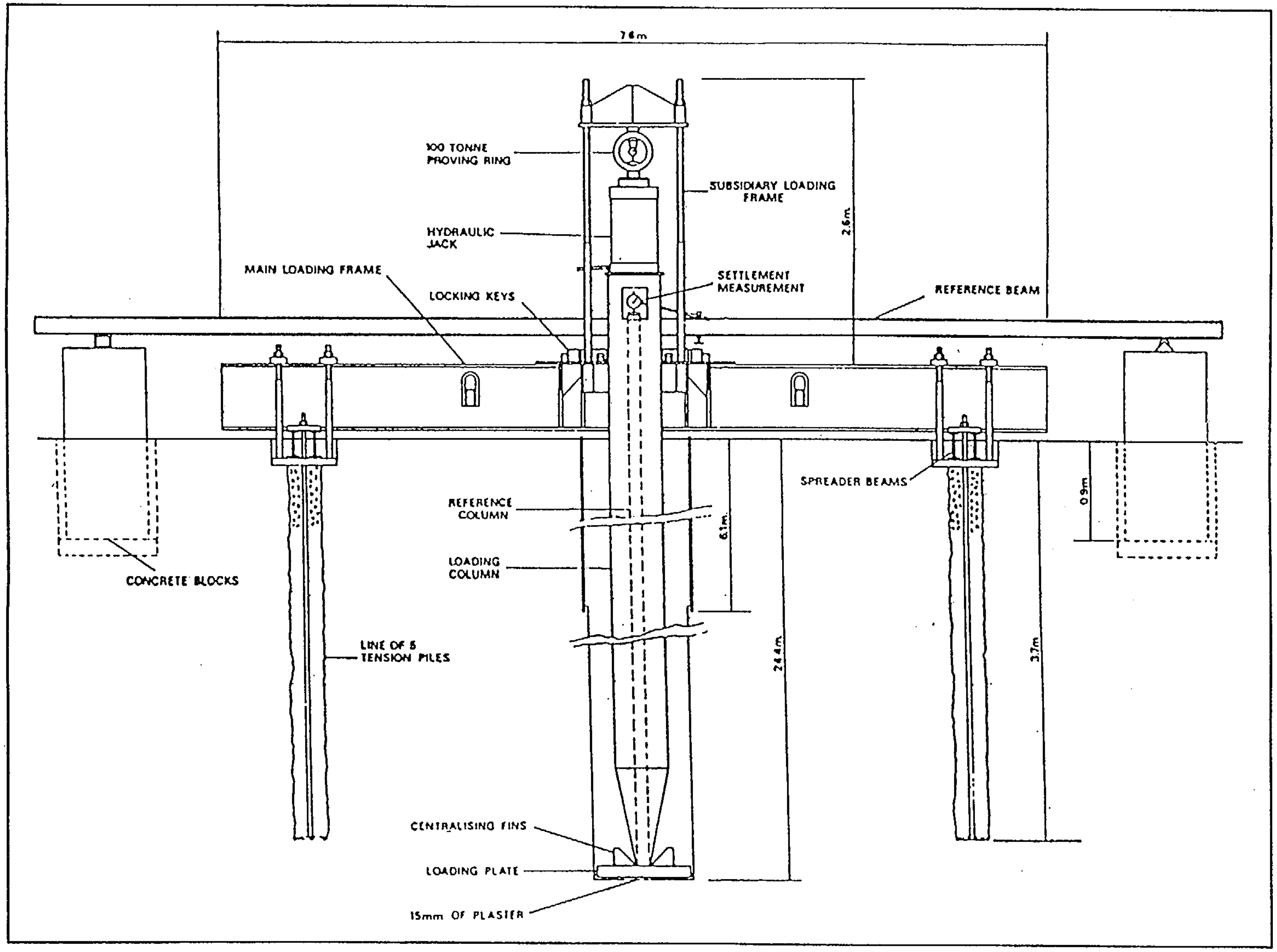


Figure 11. Plate-loading equipment. (RTH)

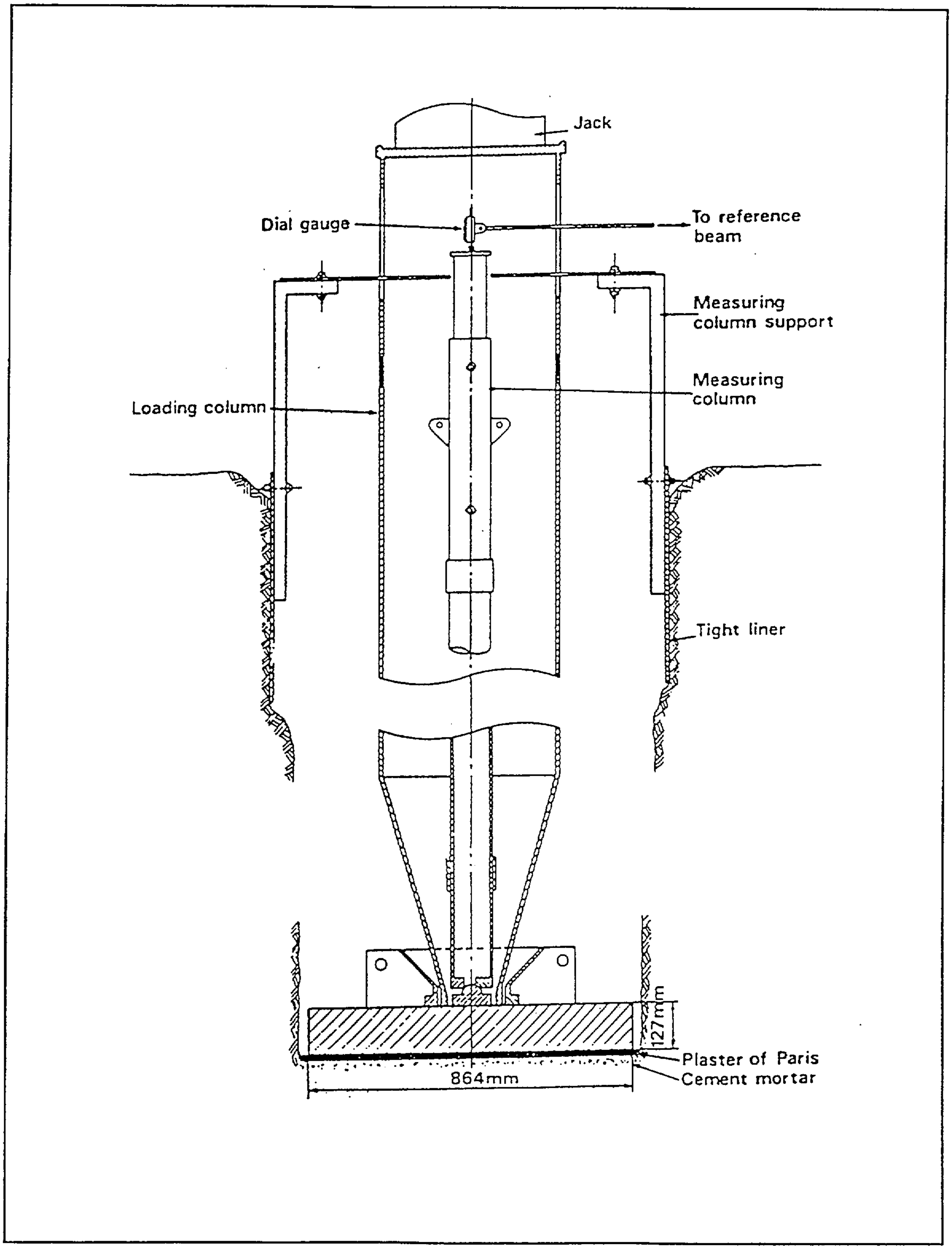


Figure 12. Details of plate tests equipment. (RTH)

6.3.3 Determination of Dynamic Moduli by Seismic Methods

Seismic methods, both downhole and surface, are used on occasion to determine in-place moduli of rock. The compressional wave velocity is mathematically combined with the rock's 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 rock particle displacement is so small and loading transitory during these seismic tests, the resulting modulus values tend to be too high. The seismic method of measuring rock 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 analysis, 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 analysis.

7 HANDLING, FIELD STORAGE, AND TRANSPORTATION OF SAMPLES

The handling, storage and transportation of samples are as critical for sample quality as the collecting procedures. Disturbance of samples after collection can happen in a variety of ways and transform samples from high quality, to slightly disturbed, to completely worthless. Soil samples can change dramatically due to moisture loss, moisture migration within the sample, freezing, vibration, shock, or chemical reactions.

Moisture loss may not be critical on representative examples, but it is preferable that it is kept to a minimum. Moisture migration within a sample provokes differential residual pore pressures to equalize with time. Water can move from one formation to another causing significant changes in the undrained strength and compressibility of the sample. Freezing of clay or silt samples can cause ice lenses to form and dramatically disturb the samples. Storage room temperatures for these kinds of samples should be kept above 4 deg C. Vibration or shock can provoke remolding, changes of strength, or density changes, specially in soft and sensitive clays or cohesionless samples. Transportation arrangements to avoid these effects need to be carefully designed. Chemical reactions between samples and samples container can occur during storage time and can provoke changes that can affect soil plasticity, compressibility, or shear strength characteristics. The correct selection of sample container material is important.

Cohesionless soil samples are particularly sensitive to disturbance due to impact and vibration during removal from borehole, from sampler and subsequent handling. Samples should be kept in the same orientation as that in which they were sampled at all times (e.g. vertical position if sampled in a vertical borehole), well padded for isolation from vibration and impact, and transported with extreme care if undisturbed samples are required.

7.1 Undisturbed Samples

Undisturbed samples must be handled and preserved in a manner to preserve stratification or structure, water content, and in situ stresses, to the extent possible (EM 1110-1-1906, DA, 1996). Once the material is brought to the surface, the sample itself should be sealed in a container or the soil sample removed from the sampler and sealed in an appropriate transporting container. If the sample will be stored in the containers for a period of time, precautions should be taken against chemical reactions between the sample and the container. Samples collected for water content information should be sealed to avoid soil moisture changes, and if glass jars are used, a good seal should be assured between the sealing edge and the cap. Guidance for handling, transportation, and preservation of samples can be obtained from EM 1110-1-1906 (DA, 1996), and ASTM D 4220-95, "Standard Practices for Preserving and Transporting Soil Samples."

7.1.1 Block Samples

Block samples need to be protected from moisture loss. The sample should be sealed by brushing with paraffin to develop a 2 mm coating, then wrapped in cheesecloth or clinging plastic wrap, and dipped again in paraffin to develop a 4 mm covering. If the specimen is too big for dipping it can be put in an oversized box and paraffin or polyurethane foam is poured on the sample to encase it. Wax seals can become ineffective after a few months and cracks may appear. Storage conditions should be kept cool and humid to minimize moisture loss after wax imperfections appear (Figure 13).

7.1.2 Sample Storage in Tubes

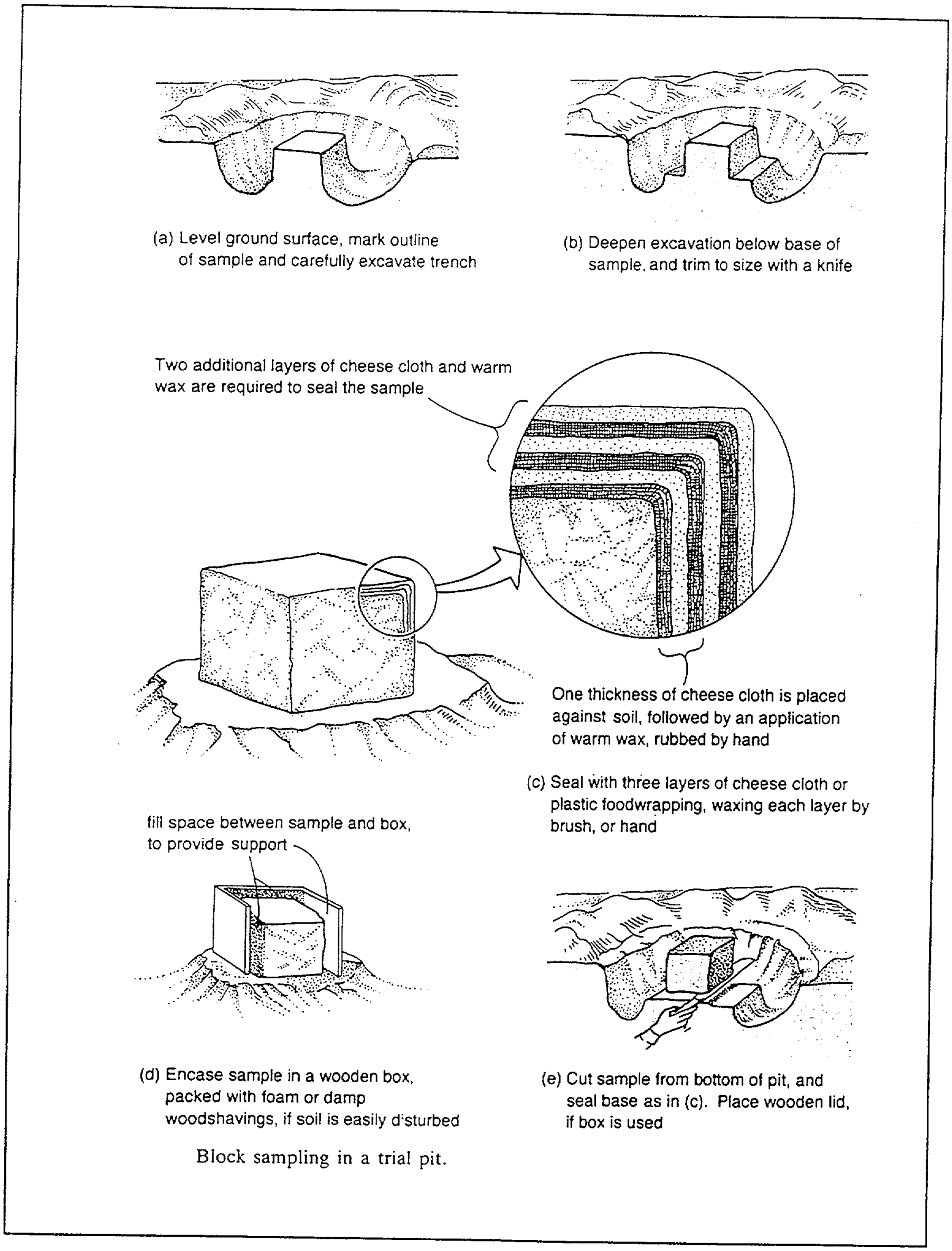
Using a cup cleanout auger, 5 cm (2 inches) of the material at the bottom of the sampling tube should be removed to be replaced with an expandable packer to seal the sample. The material removed from the tube should be stored in an appropriate container to be used for classification or water content determination afterwards.

7.1.2.1 Cohesionless Soils. Tubes containing cohesionless soils should be sealed with paper towel and a perforated expandable packer inserted at the bottom of the sampling tube. Once the sample is removed from the sampling apparatus, a cup cleanout auger should be used to remove the cuttings at the top. A metal disk (e.g., the bottom half of an expandable packer) is then placed on the top surface of the sample. The tube is subsequently kept in a vertical upward position. In this manner, any changes in the distance from the top of the tube to the metal disk would indicate that the sample has suffered changes in volume. Thus the distance should be measured immediately after cleanup of the top of the sample, at intermediate stages, and before the sample is extruded for testing.

The sample must be placed in a rack to drain. After drainage, the tube should be sealed with an expandable packer and filter paper. The excess moisture from the sample will be drained through the paper without losing sample material, minimizing sample handling disturbance and preventing liquefaction. The sample should never be allowed to dry completely since then it would be impossible to subsample.

Cohesionless sands with low content of fines and without fine material lenses can be frozen in the sampling tube and transported in this way, minimize transportation and handling disturbances (Figure 14). The sample should be drained prior to freezing to prevent structural changes due to water expansion.

7.1.2.2 Cohesive Soils. Undisturbed samples of cohesive soils can be removed from the sampling tube as they are collected, but if they are going to be preserved in the sampling tube, some material at the bottom and the top of the sampling tube should be removed. The cuttings at the top should be documented and measured, the material from the bottom of the tube should be appropriately stored, and the tube sample sealed with solid, impermeable packers.



(a) Level ground surface, mark outline of sample and carefully excavate trench

(b) Deepen excavation below base of sample, and trim to size with a knife

Two additional layers of cheese cloth and warm wax are required to seal the sample

One thickness of cheese cloth is placed against soil, followed by an application of warm wax, rubbed by hand

(c) Seal with three layers of cheese cloth or plastic foodwrapping, waxing each layer by brush, or hand

fill space between sample and box, to provide support

(d) Encase sample in a wooden box, packed with foam or damp woodshavings, if soil is easily disturbed

(e) Cut sample from bottom of pit, and seal base as in (c). Place wooden lid, if box is used

Block sampling in a trial pit.

Figure 13. Block sampling (Clayton, Matthews, and Simons, 1995; reprinted with permission).

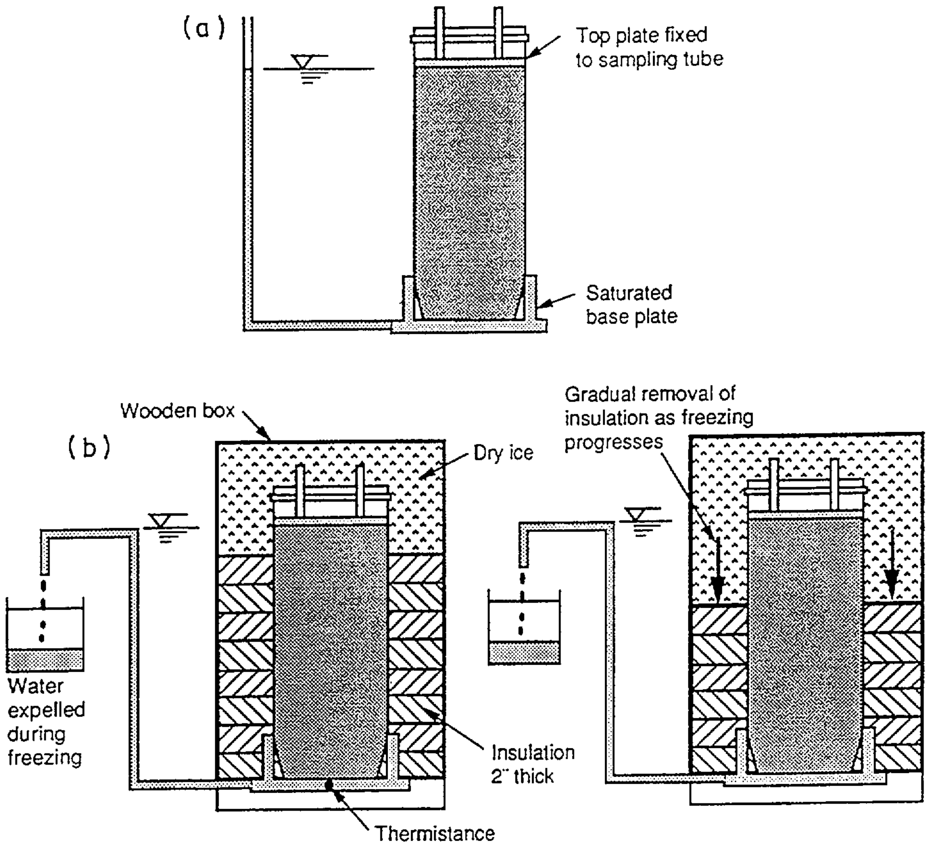


Fig. 4. Evolution of temperature profile in a sample during freezing.

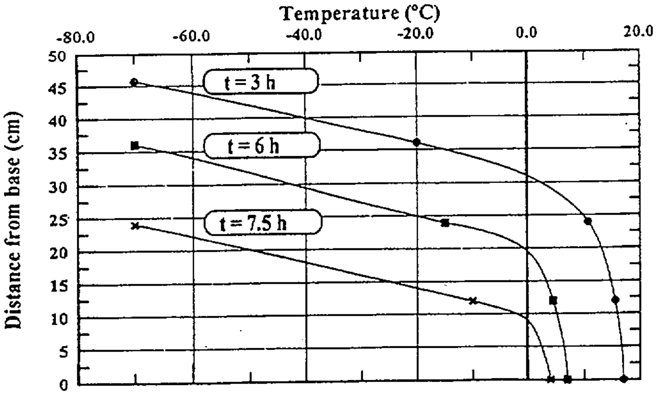


Figure 14. Procedure for freezing of samples (Konrad, St. Laurent, Gilbert, and Leroueil, 1995; reprinted with permission).

7.1.3 Sample Removal

7.1.3.1 Cohesionless Soils. In situ densities can be measured in the field from undisturbed cohesionless samples by removing soil from the tube in increments with a cup cleanout auger and measuring the volume and weight of the soil for each increment. Any material that adheres to the tube sides should be removed with a sampling tube wall scraper and included with the respective soil increment. The portion of soil remaining in a sampling tube may be extruded as discussed below for cohesive soils; freezing will ensure that disturbance of structure is very slight.

7.1.3.2 Cohesive Soils. Undisturbed samples of cohesive soils to be removed from the sampling tube should be extruded as soon as possible to minimize adhesion and friction development between the sample and the sampling tube. The sample should be pushed out of the sampler in the same direction that it was sampled, in a single, smooth, uniform stroke. This may be accomplished using a hydraulic extruding piston at the bottom of the sample that has been previously trimmed. Hydraulic jacks are the preferred method for extruding soil samples; since pneumatic jacks are not acceptable and mechanical jacks are used only if hydraulic pressure is not available.

The sample should be extruded into half-section receiving tubes of the same diameter as the sampling tube, to permit the sample to be examined for classification and stratification and preserved to be transported to the laboratory. This process has to be done as quickly as possible to reduce moisture loss and keep disturbances to a minimum. To preserve the sample, it should be placed in a wax-coated cardboard tube of about 25 mm (1 inch) larger than the sample and coated with a wax mixture (1:1 mixture of paraffin and microcrystalline wax) at a temperature of less than 10 deg C (18 deg F) so it will not penetrate the soil and thus limiting the usefulness of the sample. The sample should be centered in the tube and the space between the tube and the sample filled with wax, including the top. The sample should not be wrapped with foil or plastic. If it is too fragile for handling, cheese cloth can be used to reinforce the sample for handling.

7.1.4 Sample Transportation

Undisturbed sample tubes should be packed in the appropriate orientation (as sampled originally) in prefabricated shipping containers or in moist sawdust or similar packing materials to reduce the disturbance due to handling and shipping. Soil samples should be protected from temperature extremes and exposure to moisture (EM 1110-1-1906, DA, 1996). Guidance for handling, transportation, and preservation of samples can be obtained from ASTM D 4220-95.

7.2 Disturbed Samples

Disturbed samples can be taken for moisture content and determination of plasticity characteristics in the laboratory, or for determination of in situ density. The samples can be sealed into watertight containers and transported by any convenient means and should be clearly labeled inside and outside of the box with: site identification, boring number, sample interval, length of sample lost or not recovered in each sample interval, and top and bottom depth of sample interval.

Rock cores should be stored and transported in durable boxes with dividers to avoid shifting of cores in any direction. Special containers may be needed for samples for fluid content determinations or to prevent changes in mechanical properties in shales that would be caused by changes in moisture content.

7.3 Care and Storage

Exploratory or other cores, regardless of age, should be retained until the detailed logs, photographs, and test data have been made a matter of permanent record. Precautions should be taken to insure against the disposal, destruction, or loss of cores that may have a bearing on any unsettled claim. Such cores should be retained until final settlement of all obligations and claims.

7.4 Disposal

Soil samples may be discarded once the testing program for which they were taken is complete and as the responsible party decides. 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.

All exploratory and other cores not used for test purposes should be properly preserved, boxed, and stored in a protected storage facility until disposal. The following procedures govern the ultimate disposition of the cores in accordance with ER 1110-1-1803 and are presented as an example protocol:

> "Cores over 6 inches in diameter may be discarded after they have served their purpose. 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 and previous use permits it."

7.5 Low-Level Radioactive Waste and Mixed Waste

Regulatory and management responsibilities for low-level radioactive waste and mixed waste, are defined/enforced by federal agencies such as the Nuclear Regulatory Commission (NRC), the Department of Energy (DOE), Environmental Protection Agency (EPA), the Department of Transportation (DOT), and the Occupational Safety And Health Administration (OSHA). Background guidance for regulatory and management responsibilities for Army users, for low-level radioactive waste and mixed waste are specified in EM 1110-35-1, as an example. Specific technical and engineering guidance to personnel for the collection, handling, treatment, and disposal of these wastes for Army users, is available in EM 1110-1-4002.

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APPENDIX A
GEOLOGIC MAPPING PROCEDURES
OPEN EXCAVATIONS

A-1. Purpose of Excavation Mapping.

a. The primary purpose of the excavation map and/or foundation geologic map is to provide a permanent record of conditions during excavation. This permanent record will assist in making the most equitable contract adjustments, provide otherwise unattainable information for use in diagnosing post-construction problems and in planning remedial action, and allow for a better interpretation of post-construction foundation instrumentation data. An important prelude to performing geologic mapping of the final foundation and/or excavation is monitoring conditions during excavation. Monitoring conditions during excavation provides the basis for discovering, at the earliest possible moment, those adverse conditions (differing from original predictions) that may cause expensive design modifications and construction delays. Foundation and other excavation surfaces should not be covered until mapping is complete and approved by project geologist of geotechnical engineer.

b. A plan for monitoring also provides a basis for installing appropriate instrumentation and interpreting foundation instrumentation data as the excavation proceeds. The Instrumentation Data Package (Woodward-Clyde, in press), a PC-based program that can store, retrieve, and graphically present instrumentation data related to construction monitoring, will soon be available through the U.S. Army Engineer Research and Development Center World Wide Web site.

A-2. Possible Adverse Conditions.

a. Adverse conditions can affect the stability of excavated slopes during construction, stability of permanent slopes, foundation settlement, foundation bearing capacity, sliding stability of structures, and planned water control measures such as grouting and drainage requirements. Such conditions can occur in both soil and rock. Features of engineering significance in both soil and rock frequently occur in geometrically predictable patterns. The prediction of geometry is enhanced by knowledge of the local geologic history.

b. Adverse conditions that occur in soils include soft compressible zones of clay or organic materials; lateral compositional changes related to variations in depositional environment; changes in the relative density of granular materials; fill containing trash or other undesirable materials; swelling or slaking in hard, fissured clays; and changes in permeability.

c. Adverse conditions that can occur in rocks include weathering, soft interbeds in sedimentary and volcanic rocks, lateral changes, presence of materials susceptible to volume change (e.g., swelling clay shales, sulfide-rich shales, gypsum, and anhydrite), adversely oriented fractures (e.g., joints, bedding planes, schistosity planes, and shear planes), highly fractured zones, and fault, joints, shear planes filled with soft materials, or exceptionally hard layers that inhibit excavation method or grout/drain hole drilling.

d. Adverse conditions related to groundwater include unexpectedly high cleft or pore pressures which reduce effective stress, swelling materials, slaking, piping, sand runs, and uplift pressures on partially completed structures. It should be noted that most water-induced problems stem from unanticipated changes in the groundwater regime.

A-3. Monitoring and Mapping Procedures.

a. The difference between excavation monitoring and record mapping is small; both involve the observation and reporting of natural conditions. The need for monitoring the condition of slopes during construction include safety during construction and the prediction of conditions at grade. Potential problem areas can be detected and avoided, or corrective treatment can be started before a problem becomes severe.

b. Depending on the speed of excavation, monitoring should be performed on a daily, biweekly, or weekly basis. Because of increasing steepness, rock slopes become less and less accessible as excavation progresses. Table A-1 is an excavation monitoring checklist, which, if followed should insure adequate coverage. Geologic sections can be constructed to assist in predicting the locations of features at grade. While many geologic features are arcuate or sinuous, many are planar. The location of a planar feature at grade can be found by graphic projection or by calculation as shown on Figure A-1.

c. Excavation and foundation mapping are generally performed on an intermittent and noninterference basis. If advantage is not taken of every mapping opportunity, the rock surface may be covered before another opportunity occurs or the contractor may be subjected to undue delay. Furthermore, systematic mapping makes for better monitoring. Thoroughness of mapping, type of mapping procedure, and sequence in which it is accomplished are functions of the purpose for which the mapping is required and of the construction schedule.

d. A number of items should be done to prepare for mapping before the excavation is started.

(1) The geologist with mapping responsibility should make an interpretation, or confirm the existing interpretation of the geologic conditions (a geologic model). He should decide on a mapping strategy and prepare field base map sheets. The map scale is partially dependent on the amount of detail to be mapped. If the excavation will be in hard, fractured rock, a field scale of 1 in. = 5 ft. and a final compiled map scale of 1 in. = 10 ft. would be suitable. If the excavated material is a soil, a soft lightly fractured sedimentary rock, or a glacial till, field and compiled scales of 1 in. = 10 ft. and 1 in. = 50 ft. could be suitable. The field base maps should have reference lines for location purposes. In structure foundation areas, the structure outline will be enclosed by concrete forms that are easily locatable, making handy reference lines. The location of features inside reference lines can be facilitated by use of cloth tape grids, or with differential GPS.

(2) Decide at what intervals to map as the excavation progresses. Mapping intervals will be affected by a number of factors including the rate of excavation, lift thicknesses, and the need for temporary slope protection. In most cases, the mapping should be done in the plane of slopes; projection to other planes can be made after the mapping is completed. An exception occurs if mapping is done with a plane table. In this case, a horizontal reference plane is required. Camera positions should be selected for sequential photographs during excavation. Reference lines for mapping can be provided by stretching tapes from the top to bottom of the slope at 10- to 20-ft intervals. Final excavation topographic maps should be made that can be used as a base for the geologic map.

(3) Determine whether the side slopes will be too steep for unassisted access. Temporary soil slopes usually range from lV on 3H to 1V on lH. Temporary rock slopes usually range from 1V on 1H to vertical. It is not possible to walk slopes steeper than 1V on 1-1/2H unless they are very irregular. Thus, safety lines will be needed on most rock slopes and on some soil slopes.

Table A-1
Suggested Geologic Excavation Monitoring Checklist

Project ______

Excavation For ______ Structure

Period: ______ To ______

1. Excavation Progress:
   a. Type Excavation: (common or rock) ______
   b. Location: Sta ______ To ______ Offset ______
2. Rock or soil type: ______
3. Rock or soil conditions: (hardness, stiffness, weathering, fracturing, sloughing, etc.) ______
4. Water inflow, locations and gpm: ______
5. Significant features or defects: (those which may cause problems (and/or may extend to grade) ______
6. Slope protection: (protection or reinforcement, location, type thickness, etc.) ______
7. Blasting conditions: (presplitting locations and successes, production blasting, powder factor, hole spacing, delay patterns, deviations from approved rounds, fragment size, overbreak, etc.) ______
8. Ripping conditions: (single or multitooth, drawbar horsepower, easy or hard, disturbance below grade or slopes) ______
9. Additional remarks: (unusual incidents, accidents, explorations, etc.) ______
10. Mapping progress:
    a. Location ______
    b. Adequacy of coverage (rock surface clean?, percent obscured by slope protection?, etc.) ______
    c. Photos taken: ______ (where, or what) ______
11. Instrumentation installed:
    a. Location ______
    b. Type and amount ______
12. Instrumentation read:
    a. Location ______
    b. Type ______

$$B = \frac{D}{\tan \beta \sin \alpha} - d \text{ and}$$

$$H = \frac{D - d (\tan \beta \sin \alpha)}{1 - \cot \lambda \tan \beta \sin \lambda} \text{ or}$$

$$B = H\left(\frac{1}{\tan \beta \sin \alpha} - \frac{1}{\tan \alpha}\right) \text{ where:}$$

α = acute angle between strike of planar feature and section
β = true dip of planar feature
λ = acute angle of excavation slope
B = distance, in section, from toe of excavation slope to outcrop at excavation grade
d = distance, at section, from toe of excavation slope to surface outcrop of feature
D = depth of excavation
H = height in section of feature outcrop above grade

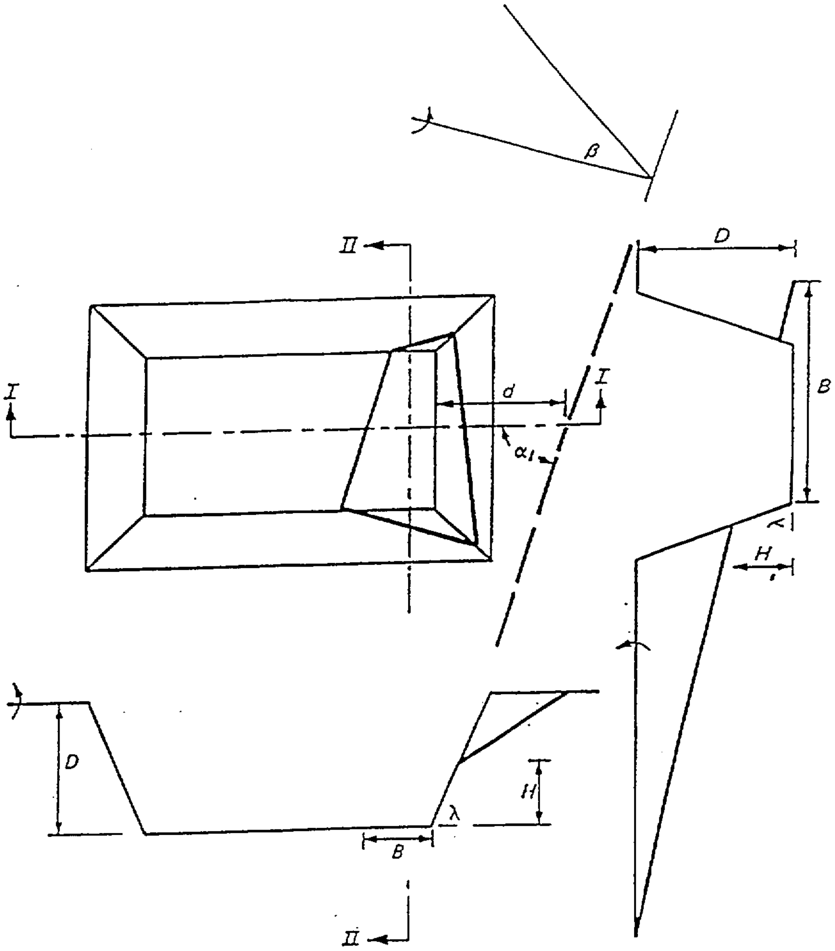


Figure A-1. Excavation plan and sections showing intersecting planar feature.

(4) Where the slopes are nearly vertical, consideration can be given to mapping on large-scale photographs. However, there must be time to produce the photographs for use as a map base.

e. It is desirable to have a contract provision for interim rock surface cleanup. The final foundation cleanup item will suffice for mapping purposes at final grade. However, the need may arise for detailed examination of particular areas during excavation. Excavation slopes may need sealing or other interim protection against weathering.

f. During mapping, complete descriptions of all geologic features should be made (e.g., rock types, bedding, fracturing, joints, shear zones, etc.). All features, geologic and otherwise (including ledges and breaks in slope) should be located and drawn on the base map. Table A-2 provides descriptive criteria for use during mapping. Insofar as possible, the information on Table A-2 is consistent with information provided in EM 1110-1-1806.

g. To the maximum extent possible, USGS map symbols or variations of these symbols should be used. In most cases, the geologist should represent geologic features by showing the trace of the feature on the map. The trace will allow a reasonably accurate location of each significant feature.

h. Frequently, foundation maps and sections are prepared with rock type symbols covering the entire area of the particular rock type. However, if all the recognizable distinct geologic features are also located on the drawing, it will be cluttered and difficult to read. Thus, the primary purpose of the foundation record will be obscured. Each mark on the foundation map should have physical significance. Figure A-2 depicts the foundation for the cutoff trench in sedimentary rocks beneath an earth dam.

i. Records should be made of foundation treatment, such as grout hole locations, dental work, pneumatic concrete, rock-bolt locations, and wire mesh. Portrayal of such treatment can be included on the geologic map. However, if the resulting map is too cluttered, either the treatment should be portrayed in a series of transparent overlays, or the scale of the mapping should be enlarged. A GIS is ideal for subdividing the geotechnical (and other construction) information into a series of data layers which can be digitally overlaid. In a GIS, map scales can be easily altered, and adjustments of the geotechnical information can be readily accomplished for presentation in a comprehensible format.

j. The importance of adequate photography and videos of the excavation process and the final slope and foundation conditions cannot be overemphasized. Complete video and photographic coverage is as important as the foundation maps. All are required for an accurate and complete record of encountered conditions. Photographs and videos can be readily incorporated into a GIS.

k. The photographic coverage should include unobstructed, medium-scale photographs of the entire foundation and closeup views of significant geologic features; a photograph through a mat of reinforcing steel is useless. All photographs should be annotated by the geologist and clearly sited on a photograph location map.

Table A-2
Descriptive Criteria, Excavation Mapping

1. Rock Type.

   a. Rock Name (Generic).

   b. Hardness.

      (1) Very soft: can be deformed by hand.

      (2) Soft: can be scratched with a fingernail.

      (3) Moderately hard: can be scratched easily with a knife.

      (4) Hard: can be scratched with difficulty with a knife.

      (5) Very hard: cannot be scratched with a knife.

   c. Degree of Weathering.

      (1) Unweathered: no evidence of any mechanical or chemical alteration.

      (2) Slightly weathered: slight discoloration on surface, slight alteration along discontinuities, less than 10 percent of the rock volume altered, and strength substantially unaffected.

      (3) Moderately weathered: discoloring evident, surface pitted and altered with alteration penetrating well below rock surfaces, weathering "halos" evident; 10 to 50 percent of the rock altered, and strength noticeably less than fresh rock.

      (4) Highly weathered: entire mass discolored, alteration pervading nearly all of the rock with some pockets of slightly weathered rock noticeable, some minerals leached away, and only a fraction of original strength retained (with wet strength usually lower than dry strength).

      (5) Decomposed: rock reduced to a soil with relict rock texture (saprolite), and generally molded and crumbed by hand.

   d. Lithology, Macro Description of Mineral Components. Use standard adjectives such as shaly, sandy, silty, and calcareous. Note inclusions, concretions, nodules, etc.

   e. Texture and Grain Size.

      (1) Sedimentary rocks:

Texture	Grain Diameter	Particle Name	Rock Name
*	80 mm	Cobble	Conglomerate
*	5 - 80 mm	Gravel	
Coarse grained	2 - 5 mm		
Medium grained	0.4 - 2 mm	Sand	Sandstone
Fine grained	0.1 - 0.4 mm		
Very fine grained	0.1 mm	Clay, silt	Shale, claystone, siltstone

[*] Use clay-sand texture to describe conglomerate matrix.

      (2) Igneous and metamorphic rocks:

Texture	Grain Diameter
Coarse grained	5 mm
Medium grained	1 - 5 mm
Fine grained	0.1 - 1 mm
Aphanite	0.1 mm

(Continued)

Table A-2 (Continued)

(3) Textural adjectives: Use simple standard textural adjectives such as porphyritic, vesicular, pegmatitic, granular, and grains well developed, but not sophisticated terms such as holohyaline, hipidiomorphic granular, crystalloblastic, and cataclastic.

2. Rock Structure.

  a. Bedding.

    (1) Massive: 3 ft thick.

    (2) Thick bedded: beds from 1-3 ft thick.

    (3) Medium bedded: beds from 0.3 ft - 1 ft thick.

    (4) Thin bedded: beds less than 0.3 ft thick.

  b. Degree of Fracturing (jointing).

    (1) Unfractured: fracture spacing 6 ft.

    (2) Slightly fractured: fracture spacing 3-6 ft.

    (3) Moderately fractured: fracture spacing 1-3 ft.

    (4) Highly fractured: fracture spacing 0.3-1 ft.

    (5) Intensely fractured: fracture spacing 0.3 ft.

  c. Shape of Rock Blocks.

    (1) Blocky: nearly equidimensional.

    (2) Elongated: rodlike.

    (3) Tabular: flat or bladed.

3. Discontinuities.

  a. Joints.

    (1) Type: bedding, cleavage, foliation, schistosity, and extension.

    (2) Separations: open or closed; how far open.

    (3) Character of surface: smooth or rough; if rough, how much relief; average asperity angle.

    (4) Weathering or clay products between surfaces.

  b. Faults and Shear Zones.

    (1) Single plane or zone: how thick?

    (2) Character of sheared materials in zone.

    (3) Direction of movement, and slickensides.

    (4) Clay fillings.

  c. Solution Cavities and Voids.

    (1) Size.

(Continued)

Table A-2 (Concluded)

(2) Shape: planar, irregular, etc.

(3) Orientation: (if applicable) developed along joints, bedding planes, at intersections of joints and bedding planes, etc.

(4) Filling: percentage of void volume and type and of filling material (e.g., sand, silt, clay, etc.).

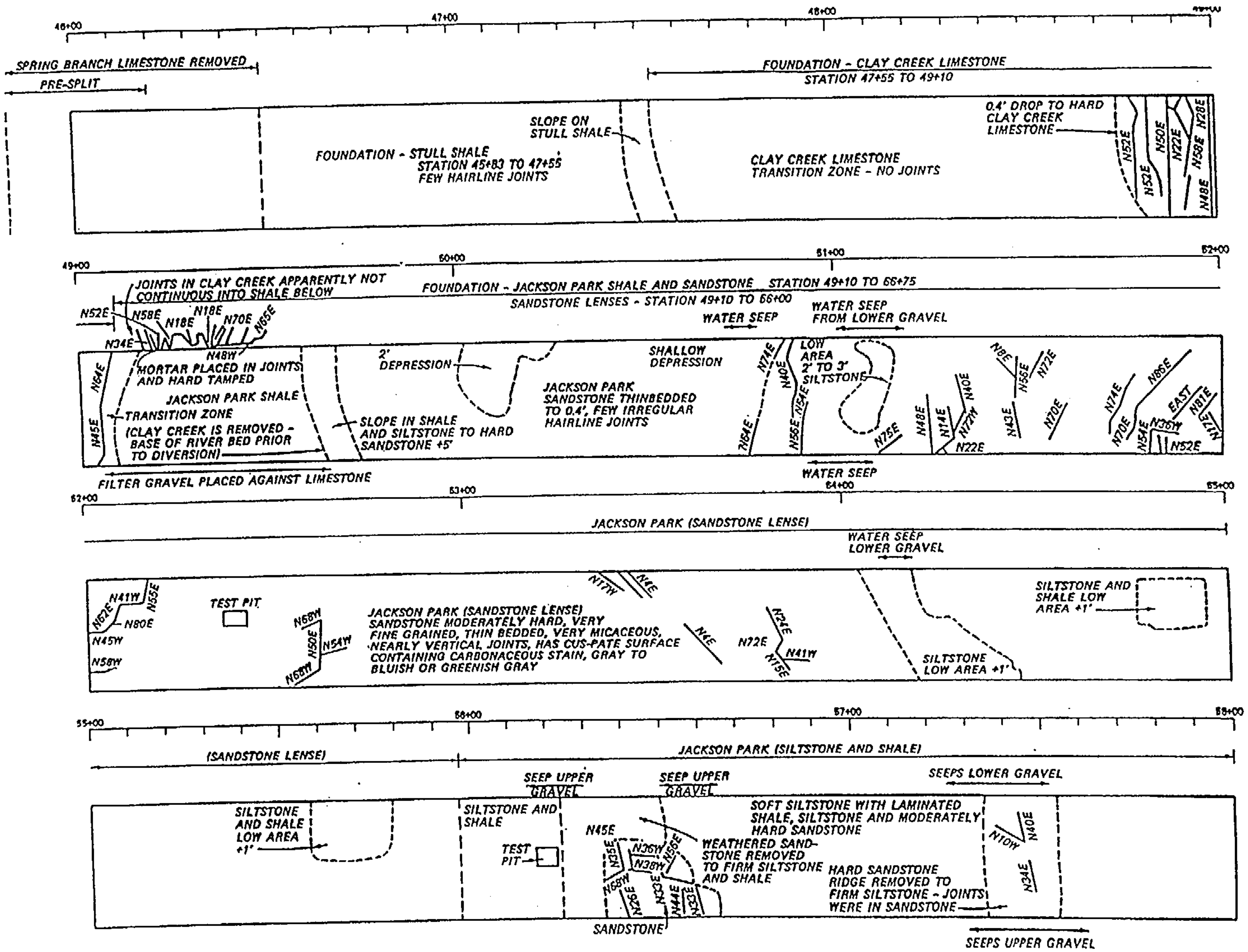


Figure A-2. Foundation map for earth dam impervious core on sedimentary rocks.

APPENDIX B
GEOLOGIC MAPPING OF TUNNELS AND SHAFTS

B-1. Background.

A method to log all geologic features exposed by underground excavations has been developed by U.S. Army Engineer District, Omaha, geologists where all necessary data of a specific geologic discontinuity can be recorded at a single point; thus the system may be used in tunnels of almost any configuration and inclination. This method is called peripheral geologic mapping. It allows logging of all geologic defects regardless of their position on the tunnel walls. Furthermore, this method usually will keep pace with modern continuous mining techniques and will immediately provide useful data without projecting to plan or profile. Prior to development of this method, the accepted method was to project geologic features to a plan placed tangent to a point on the tunnel circumference. Ordinarily such tangent points were at springline, wasteline level, or crown. In many instances, geologic features not passing through these points were not logged. Further, some systems were useful for logging planar discontinuities only, such as joints, faults, and bedding planes, as exposed in straight, nearly horizontal tunnels of circular cross section. Peripheral geologic mapping uses a developed plan by "unrolling the circumference" to form a plan of the entire wall surface. A log of the exposed geology is plotted on this plan as mining progresses. Mapping on a developed layout of a cylindrical surface is similar to the method used to log the interior of a calyx hole. Actually, a circular tunnel might be visualized as a large horizontal or nearly horizontal drill hole.

B-2. Applications.

Peripheral geologic mapping may be used to log large-diameter power tunnels and surge tank risers (both straight and wye-shaped), vertical shafts, horseshoe-shaped drifts and chambers, and various odd-shaped openings on both civil and military projects. It can be used to map a wide range of geomaterials from stratified, soft, sedimentary rocks to hard igneous and metamorphic rock masses. The method has proved to be simple enough mechanically that technicians can be trained to perform round-the-clock mapping under the general supervision of a professional geologist--a necessity where several parallel tunnels are driven simultaneously.

This method is not applicable to TBM driven tunnels with precast liners where mapping may be impracticable or impossible.

B-3. Procedure.

a. Advance planning is of paramount importance. The developed layouts on which mapping will be done should be prepared well in advance. Usually this step in the procedure can be accomplished by using the contract plans. A thorough surface and subsurface study of the geology of the immediate area is recommended. This study enables the mapper to recognize which geologic features are important and readily identify them on the excavation walls.

b. The map is typically laid out to a scale of 1 in. equals 10 ft. In some instances where closely-spaced geologic discontinuities are anticipated, a scale of 1 in. equals 5 ft. should be considered. To prepare a mapping plan, draw the crown center line of the tunnel in the center of the plan. Place the center line of the invert at both the right- and left-hand edges of the developed layout. The right and left springlines of a circular tunnel will be midway between the center line and edges of the plans (Figure B-1). Distances down tunnel may be laid out on tunnel stationing. Separate developed plan tracings are

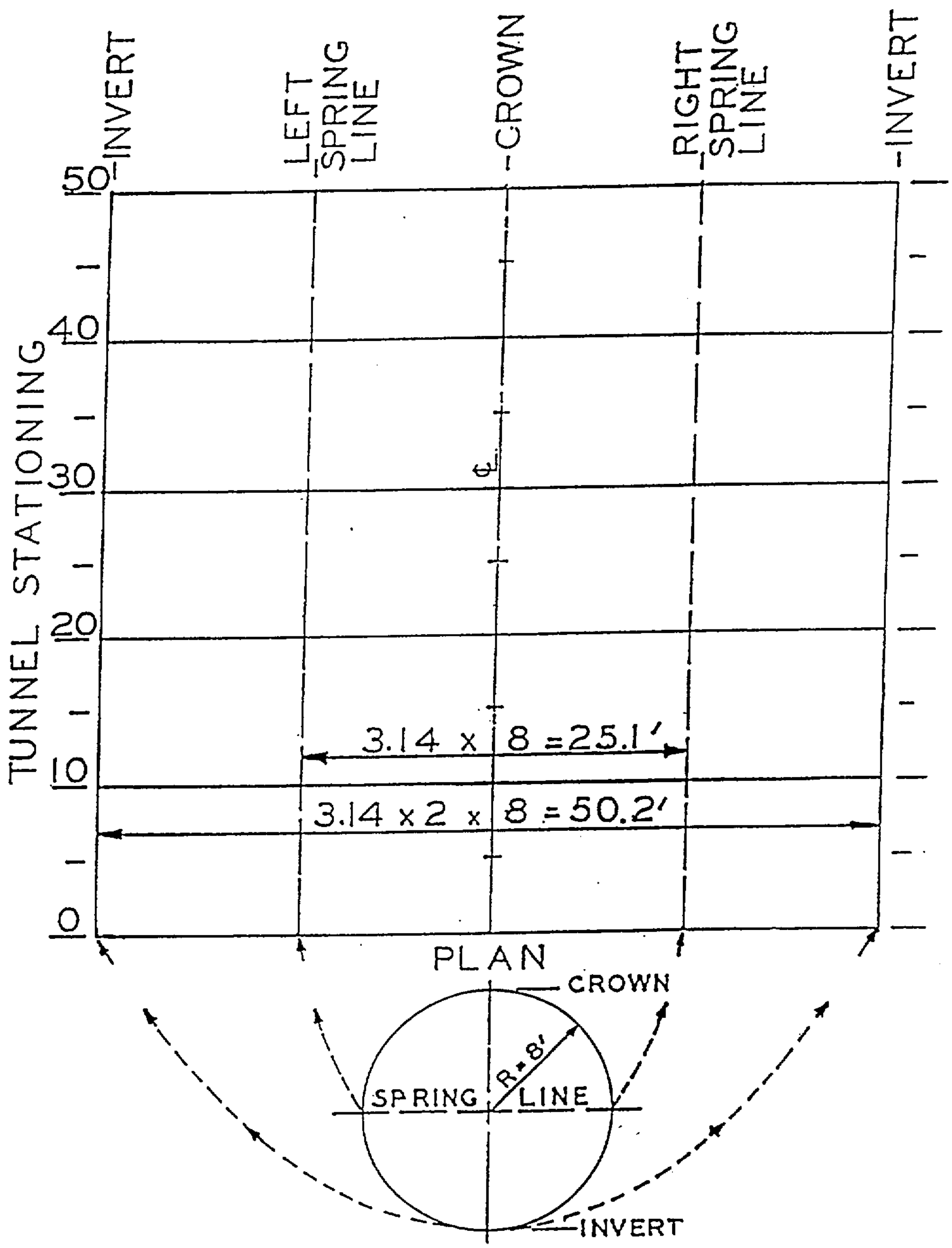


Figure B-1. Preparation of developed plan from a cylindrical cross section.

typically made for 100-ft lengths of tunnel. For long reaches of equal diameter tunnel, a master tracing may be repeatedly printed on a continuous length of paper to cover the entire tunnel length. For continuous uninterrupted printing use three master tracings. This long sheet of paper may be rolled up and carried in the field in the form of a scroll.

c. Intermediate control points should be added wherever possible to more precisely locate points along geologic discontinuities. On Figure B-2, which is a mapping sheet used at Fort Randall Dam, South Dakota, the horizontal distances from the center line and vertical distances from springline were computed and drawn on the developed plan to form a grid. When plotting a point, the mapper measures these two distances (horizontal distance from center line and vertical distance above or below springline) and plots the point at the proper tunnel stationing. To eliminate long measurements in large-diameter tunnels, distances were actually measured from fixed known points on the tunnel support ring beams (splices, bolt holes, and spreader bars). At Oahe Dam, South Dakota, horizontal and vertical distances of fixed features on the ring beam supports were drawn on the mapping sheets as lines so that points on geologic features could be plotted from the nearest ring beam reference point. On Figure B-3, which is a mapped portion of Oahe Dam power tunnel No. 2, a developed ring beam is shown at the top of the page, vertical distances from springline of identifiable fixed points on the ring beam are shown along the top of the mapping section, and horizontal distances from center line of these same points are shown along the bottom of the mapping section. The ring beam number and its tunnel station is shown along the right-hand edge. Excellent mapping control was thus provided on this project. In excavations not requiring close checks on alinement, control points may be almost nonexistent. In such cases, the mapper must establish his own control points. He may have to stretch a tape along the tunnel from the nearest spad, then mark stationing at 5- or 10-ft intervals along the walls, and use an assumed elevation at his reference point. Obviously, the resultant geologic log will not be as accurate, but the relative position of discontinuities should remain constant from tunnel wall to geologic log.

d. The conventional method of measuring the strike, or orientation of a joint, shear, or fault, by magnetic needle (Brunton) compass is not reliable in most underground work because of the proximity to electrical circuits, reinforcing steel, or support steel. Also in some areas, the rock mass itself may be magnetic. To overcome this problem, an adjustable protractor can be devised. Essentially, it is an instrument for measuring the angle between the trend of a planar geologic defect, as measured in the horizontal plane, and the bearing of the tunnel center line. The protractor is fitted with a revolving pointer, which rotates around the center point of the protractor. The baseline of the protractor is held parallel to the tunnel center line, the pointer is sighted along the strike of the discontinuity, and the angle is read on the protractor at the point where a line scribed on the pointer coincides with the degree lines on the protractor arc (Figure B-4). The strike of the geologic defect is then computed from the observed angle and the bearing of the tunnel. In small-diameter drifts, tunnels, adits, etc., a small, light, fixed-base protractor will be adequate for fairly accurate readings. In large-diameter openings, a special protractor may be constructed that has an adjustable baseline. The baseline of the instrument is then revolved to the known tunnel bearing so that direct readings of strike may be taken (Figure B-4). A circular spirit level bubble may be mounted on the instrument to assure that readings are in the horizontal plane. Dip readings are observed by using the inclinometer on a Brunton compass or pocket transit.

B-4. Helpful Suggestions.

a. The geologic features that have the greatest effect on the physical and engineering properties of the rock mass should be logged first. Geologic logging should be performed close to the heading as fresh rock is exposed, before the exposed walls become dust covered or smeared over, and before the geologic features are partially or completely covered by tunnel supports, lagging, pneumatically placed mortar,

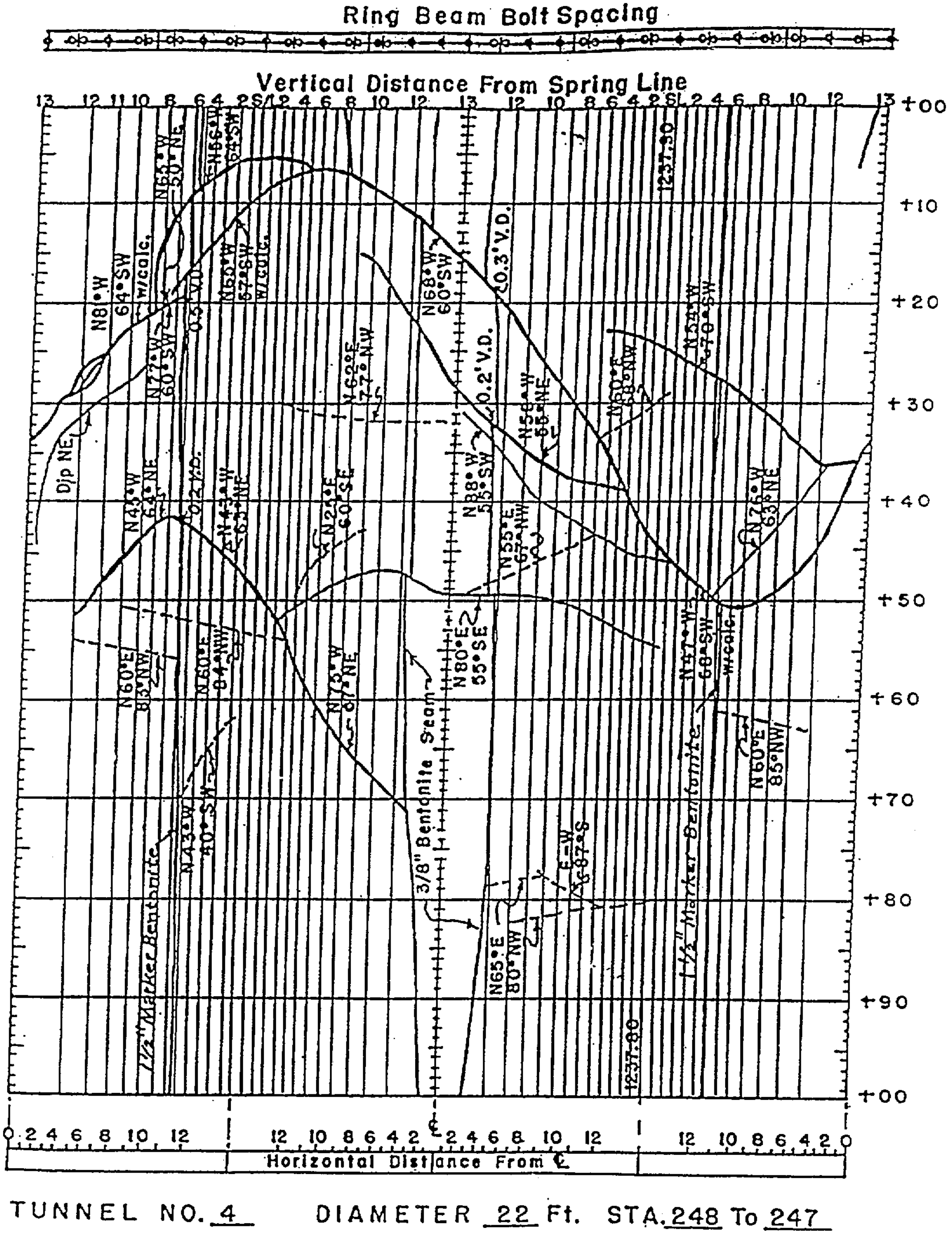


Figure B-2. Developed plan of cylindrical tunnel section, Fort Randall Dam, South Dakota

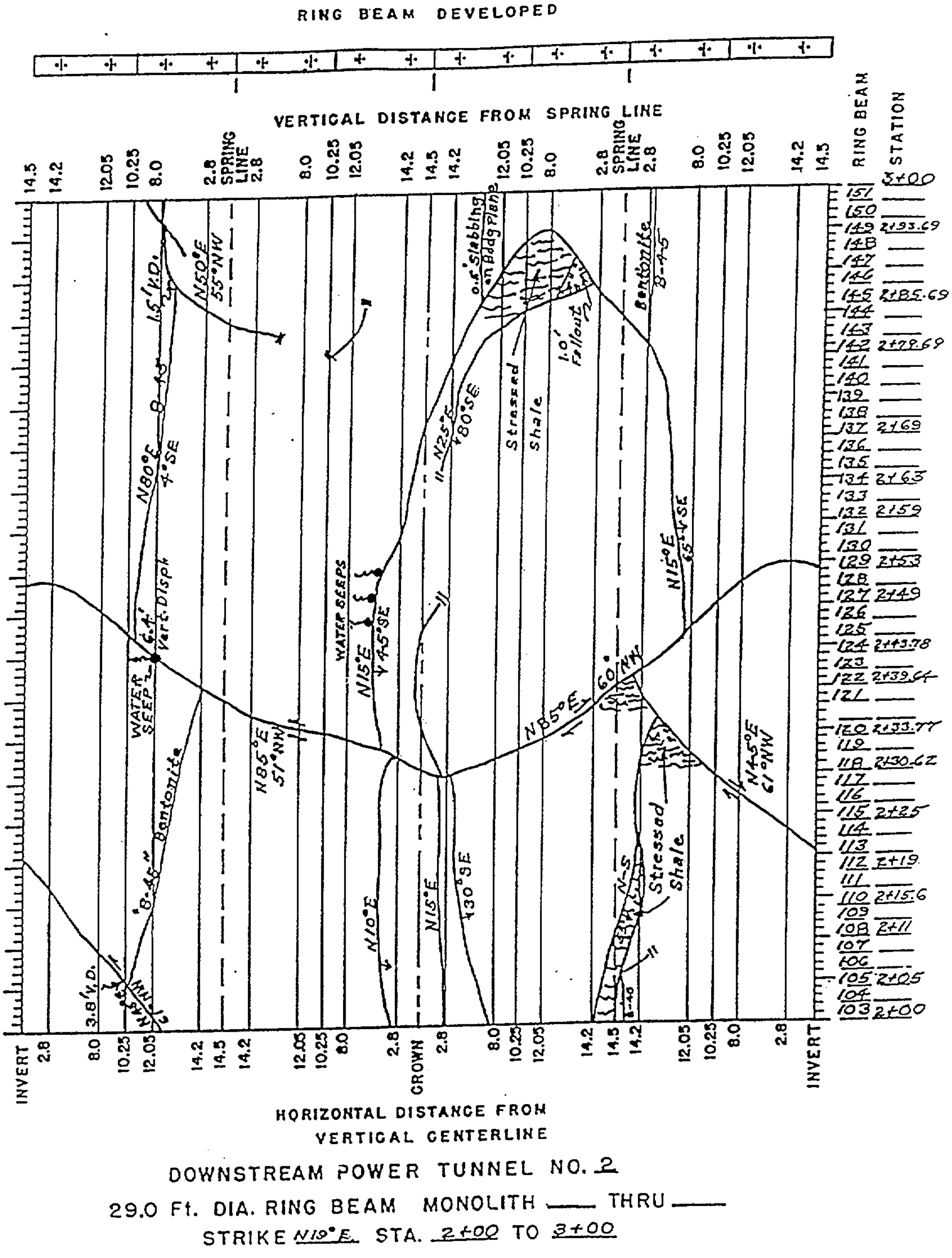
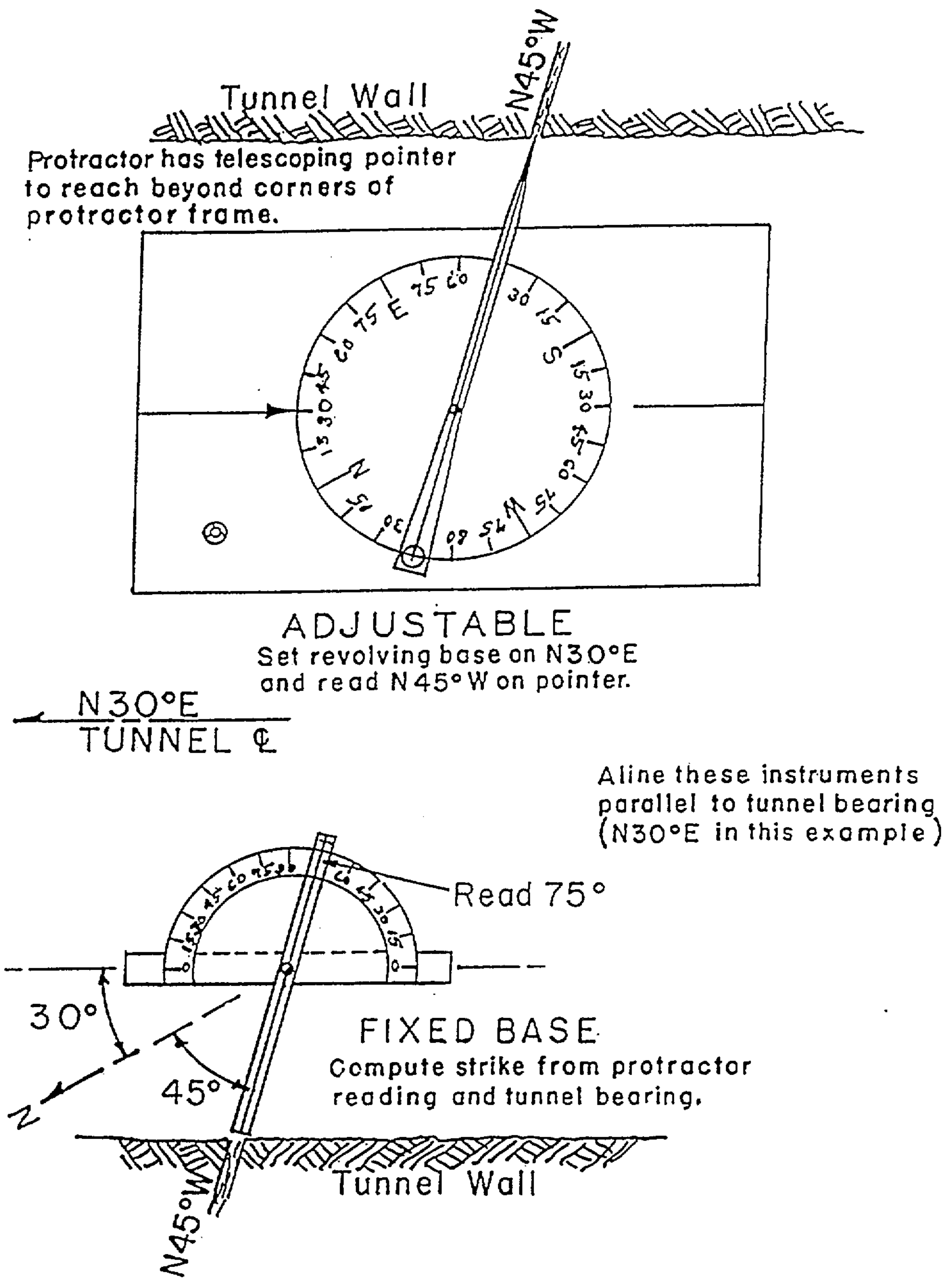


Figure B-3. Developed plan of cylindrical tunnel section, Oahe Dam, South Dakota.



Protractor has telescoping pointer to reach beyond corners of protractor frame.

Set revolving base on N30°E and read N45°W on pointer.

Aline these instruments parallel to tunnel bearing (N30°E in this example)

Compute strike from protractor reading and tunnel bearing.

Figure B-4. Sketch of typical protractors used in peripheral geologic mapping.

etc. The mapper should ensure that adequate lighting is available. Mapping should be from the back of the mining machine or on the drill jumbo to help the mapper reach the higher sidewalls and crown in large-diameter tunnels. The main geologic features, such as faults, joints, shear zones, bedding planes, and clay seams, should be carefully plotted first. Then as time permits, other less important features may be filled in between the previously plotted features on the geologic log. Additional features to be logged include fractures, stressed zones, fallouts, water seeps, etc. These features make up only a partial list because additional important geologic features will be encountered at each specific project.

b. In large- and medium-diameter tunnels consecutive mapping sections may be printed on a long sheet of paper to form a scroll. This continuous length of paper can be carried on a mapping board so designed that only the section being mapped is exposed while the remainder of the roll is inclosed in boxes on each side of the mapping board. Cranks and rollers may be added to assist in moving the proper section onto the mapping board. The board may be faced with a piece of sheet metal to provide a smooth writing surface. In small or odd-shaped tunnels or drifts the mapping sheets are usually carried in individual, conveniently sized sections. A covered clipboard makes a good mapping board. The cover is to protect the mapping sheets from the ever present dust, moisture, etc., associated with underground excavations.

If necessary, the mapper can extend his own control from known points, a steel tape is stretched along the tunnel and 5- or 10-ft. station intervals may be marked on the wall or supports with an aerosol can of spray paint. Photographs of important or unusual geologic features are a valuable addition to the mapping. It is also suggested that a small portable tape recorder for noting the location and attitude of secondary features will help the mapper, especially in adding secondary features to the mapping when time in the tunnels is limited.

c. The completed geologic log of a horizontal or nearly horizontal tunnel will wrap around a mold of proper dimensions to form a model with the mapped features and recorded information in their proper position; however, the geologic log of peripheral mapping in a vertical shaft or end face will not be in its proper position unless the information is traced through the paper to reverse the image. The reversal of the image presents no particular problem because in most instances the field maps and data are transcribed to finished drawings in the office. Also in odd-shaped raises or in vertical shafts, it is difficult for the mapper to remain properly oriented unless vertical reference points around the periphery have been surveyed-in prior to the start of geologic mapping.

B-5. Analysis of Data.

a. Although peripheral geologic logging, or mapping, provides a permanent record of all geologic defects exposed on the walls of an underground excavation, maximum benefits cannot be gained unless the data are properly studied and analyzed. One study method is by cutting and trimming the drawings and forming them into the proper shape for three-dimensional viewing, which causes the relationship of discontinuities to the tunnel geometry to become much more apparent.

b. Projection of the trace of geologic discontinuities to two-dimensional plans or profiles may be made, but not directly because the mapping has been done on a developed plan. One method of transferring data to plan is by plotting to corresponding stationing. Data may be transferred to profile, or cross section, by plotting the points where the discontinuities intersect measured stationing at crown, invert, and/or springline. Where only one point can be plotted, the trace of the discontinuity may be extended along a line drawn on the recorded strike or dip of a discontinuity (the use of apparent dip may also be necessary). Figure B-5 illustrates a method of projecting geologic data to sections drawn through a circular tunnel.

c. Statistical studies may be made from the accumulated data. By counting all discontinuities per unit of length and circumference, an average piece size or block size may be determined. Plotting the trends of joints, faults, and shears on equal-area nets will help determine the major and minor joint sets and the preferred orientation of faults and shear zones. Another method of statistical analysis might be by making rosette plots of the joints and shears.

B-6. Uses for Geologic Data.

The value of peripheral geologic mapping has been proven many times. Below are listed some of the uses for this type of geological logging.

a. Predicting geologic conditions in intermediate tunnels where driving a series of parallel tunnels.

b. Projecting geology from the pilot drift to the full bore of a tunnel before enlarging is started.

c. Planning tunnel support systems and selecting the best location and inclination of supplemental rock bolts.

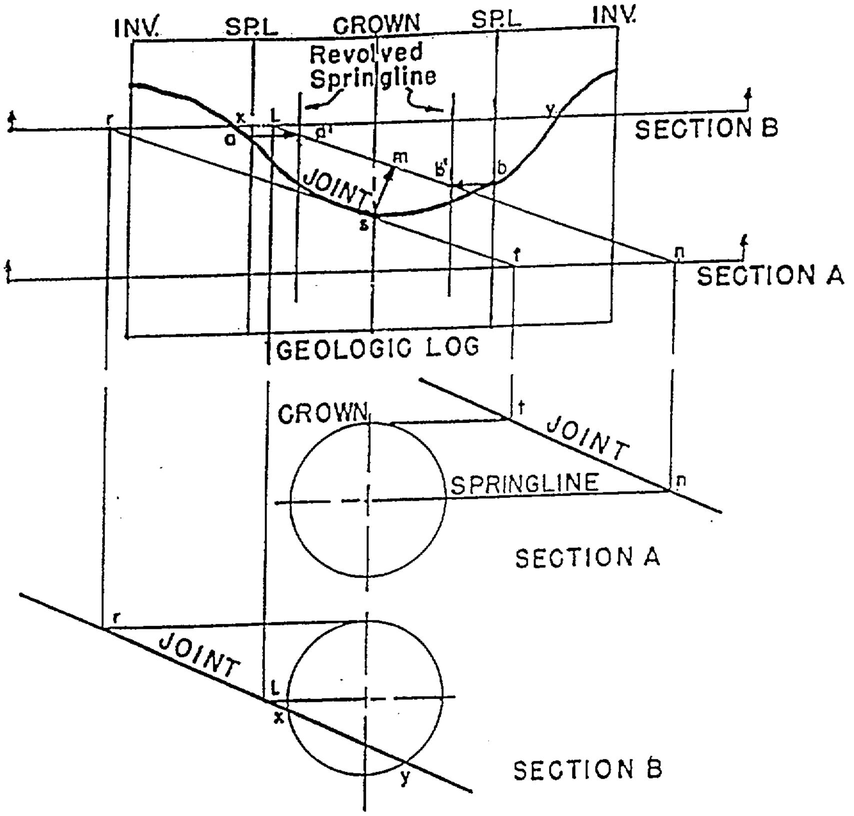
d. Maintaining a record of difficult mining areas, overbreak and fallout, and mining progress by daily notation of the heading station. This type of record is valuable in changed condition claims.

e. Comparing cracking of concrete tunnel liners with weaknesses logged in tunnel walls.

f. Analyzing stress conditions around tunnel openings using methods that evaluate the spacing and orientation of geologic discontinuities.

g. Choosing strategic locations for various types of instrumentation to study tunnel behavior.

h. Selecting the best locations for pore pressure-type piezometer tubes where it is desirable to position them to intercept particular types of discontinuities at specific elevations near previously driven tunnels.



1. The strike in relation to the tunnel can be found by three different methods:
   a. Revolve springline to tunnel diameter and measure strike a'b'.
   b. Measure tangent to curve at the crown;
   c. Use measured strike.
2. Project the intersection of the strike at the crown (s) to sections desired. For example, joint is at crown level at t and r respectively in sections A & B.
3. Project the strike of the joint at springline level to the section desired; in the example, the joint is at springline elevation at points n and L respectively in sections A and B.
4. Line tn is the trace of the plane on section A. Line rl is the trace of the plane on section B.

   Note: Points x and y show where the joint plane intersects the tunnel boundary in section B. It does not intersect the boundary in section A.
5. Find dip (δ) from distance sm. $\delta = \tan^{-1}\left(\frac{\text{tunnel radius}}{sm}\right)$, or use dip measured in the field.

Figure B-5. Method of projecting geologic data to cross sections from geologic log as developed by R. E. Goodman, PhD, Univ. of California.

APPENDIX C
EXAMPLES OF DRILLING LOGS

C-l. General.

This appendix contains seven examples of drilling logs, five for overburden drilling and two for rock coring. These logs conform to the guidance presented in EM 1110-1-1806, "Presenting Subsurface Information in Contract Plans and Specifications." The examples are not meant to cover all possible subsurface conditions which may be encountered during field investigation, but are presented to give direction to the minimum acceptable input to completing drilling logs for the most common drilling activities.

C-2. Preparation of Drilling Logs.

Drilling logs should be made of each boring. A similar log will be prepared for each excavation which is constructed for the purpose of characterizing subsurface materials and geologic conditions. The drilling log form approved for Corps of Engineers borings and presented for example here is ENG Form 1836 (March 1971). This form may be used as a continuation sheet or, at the option of the user, ENG Form 1836-A (June 1967) may be used.

a. Scale. A scale of 1 in. = 2 ft. or larger should be used. A smaller scale may be used where, for example, the boring is advanced without sampling or logging, the upper portion of the log would represent water, or the boring was made to identify some geologic horizon such as top of rock. Other similar exceptions would be allowable.

b. Heading. All logs will have the pertinent division, installation, location, hole number, project identification, elevation, and page number entered on all log sheets. Items 1 through 19 on ENG Form 1836 should be completed to the fullest extent possible as indicated in the seven examples. Boring numbers will be consecutive for each project. The boring numbers will be proceeded by letter symbols which will identify the method of drilling. These letters are as follows:

A - Auger (Hand or Power)
C - Core
D - Drive
P - Probe
U - Undisturbed (Hydraulic or Rotary)

Additional letters and numbers for boring identification may be used at the user's discretion. Inclusion of the graphic soil symbol in column c is optional.

c. Examples. The drilling log examples of ENG FORM 1836, Figures C-l through C-7, are described as follows:

Figure C-l: Overburden, disturbed, standard penetration test and auger.

Figure C-2: Overburden, disturbed, drive.

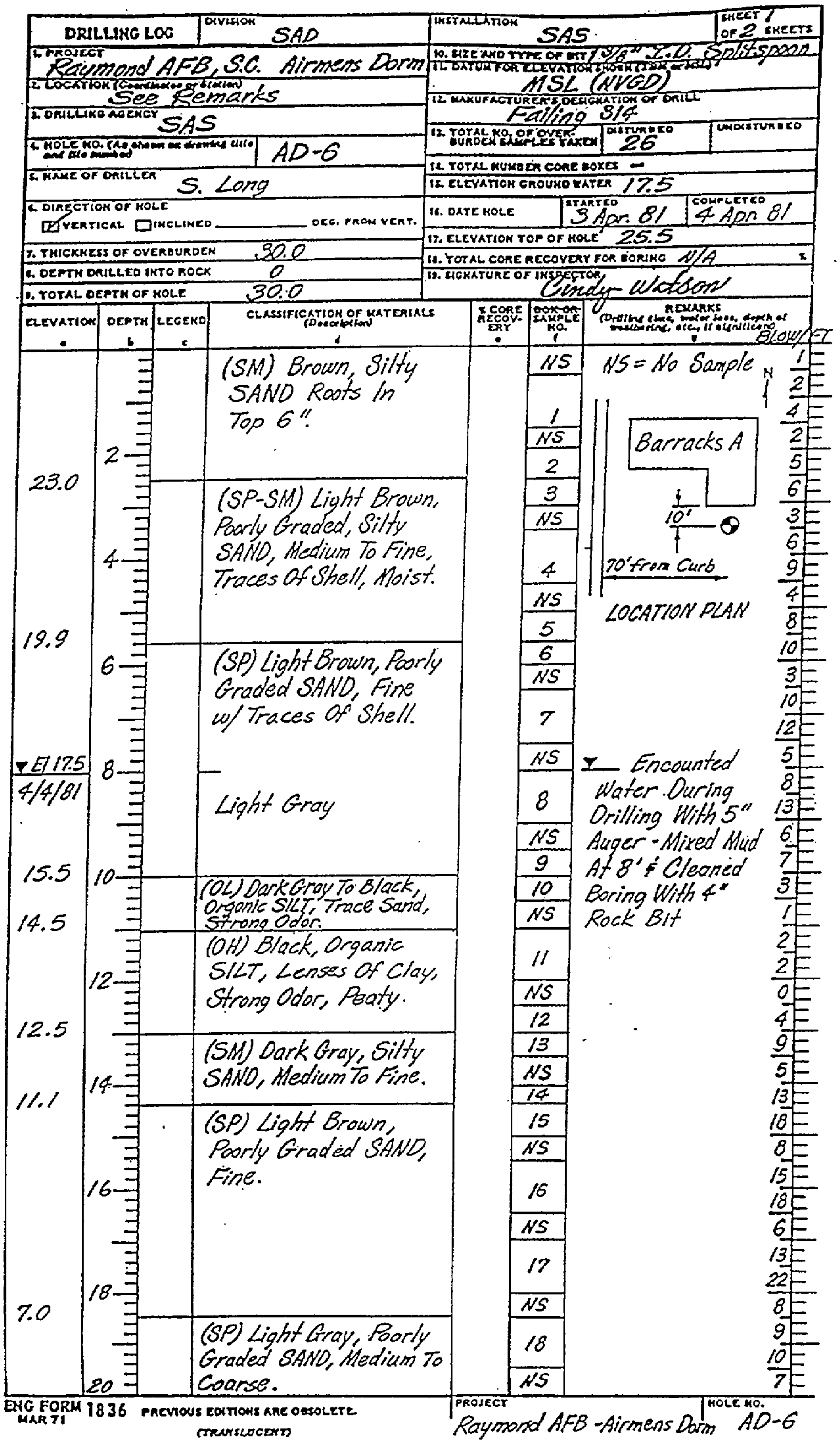
Figure C-3: Overburden, disturbed, auger.

Figure C-4: Overburden, undisturbed, Denison.

Figure C-5: Overburden, undisturbed, Shelby and auger.

Figure C-6: Bedrock, disturbed, SPT and core.

Figure C-7: Bedrock, core.



DRILLING LOG
DIVISION SAD
INSTALLATION SAS
SHEET 1 OF 2 SHEETS

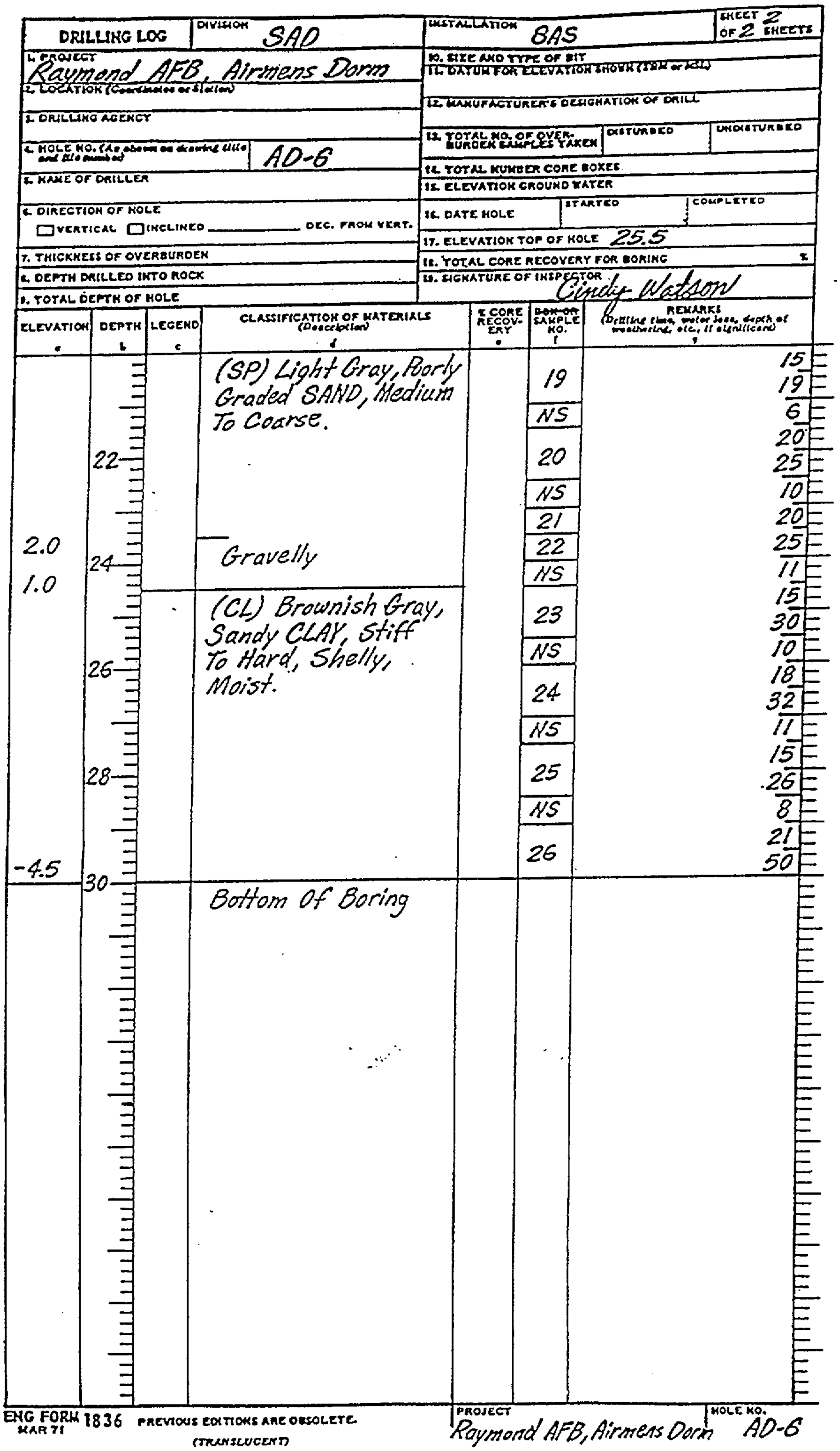
1. PROJECT Raymond AFB, S.C. Airmens Dorm
2. LOCATION (Coordinates or Station) See Remarks
3. DRILLING AGENCY SAS
4. HOLE NO. (As shown on drawing title and file number) AD-6
5. NAME OF DRILLER S. Long
6. DIRECTION OF HOLE ☑ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN 30.0
8. DEPTH DRILLED INTO ROCK 0
9. TOTAL DEPTH OF HOLE 30.0

10. SIZE AND TYPE OF BIT 1 3/8" I.D. Splitspoon
11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL (NVGD)
12. MANUFACTURER'S DESIGNATION OF DRILL Falling 314
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN DISTURBED 26 UNDISTURBED
14. TOTAL NUMBER CORE BOXES –
15. ELEVATION GROUND WATER 17.5
16. DATE HOLE STARTED 3 Apr. 81 COMPLETED 4 Apr 81
17. ELEVATION TOP OF HOLE 25.5
18. TOTAL CORE RECOVERY FOR BORING N/A %
19. SIGNATURE OF INSPECTOR Cindy Watson

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	BLOW/FT
	2		(SM) Brown, Silty SAND Roots In Top 6".		NS 1 NS 2	NS = No Sample N Barracks A 10' 70' from Curb LOCATION PLAN	1 2 4 2 5
23.0	4		(SP-SM) Light Brown, Poorly Graded, Silty SAND, Medium To Fine, Traces Of Shell, Moist.		3 NS 4 NS 5		6 3 6 9 4 8
19.9	6		(SP) Light Brown, Poorly Graded SAND, Fine w/ Traces Of Shell.		6 NS 7 NS		10 3 10 12 5
▼El 17.5 4/4/81	8		Light Gray		8 NS 9	▼ Encounted Water During Drilling With 5" Auger - Mixed Mud At 8' & Cleaned Boring With 4" Rock Bit	8 13 6 7
15.5	10		(OL) Dark Gray To Black, Organic SILT, Trace Sand, Strong Odor.		10 NS		3 1
14.5	12		(OH) Black, Organic SILT, Lenses Of Clay, Strong Odor, Peaty.		11 NS 12		2 2 0 4
12.5	14		(SM) Dark Gray, Silty SAND, Medium To Fine.		13 NS 14		9 5 13
11.1	16 18		(SP) Light Brown, Poorly Graded SAND, Fine.		15 NS 16 NS 17 NS		18 8 15 18 6 13 22 8
7.0	20		(SP) Light Gray, Poorly Graded SAND, Medium To Coarse.		18 NS		9 10 7

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)
PROJECT Raymond AFB - Airmens Dorm
HOLE NO. AD-6

Figure C-l. Overburden, disturbed, standard penetration test and auger.



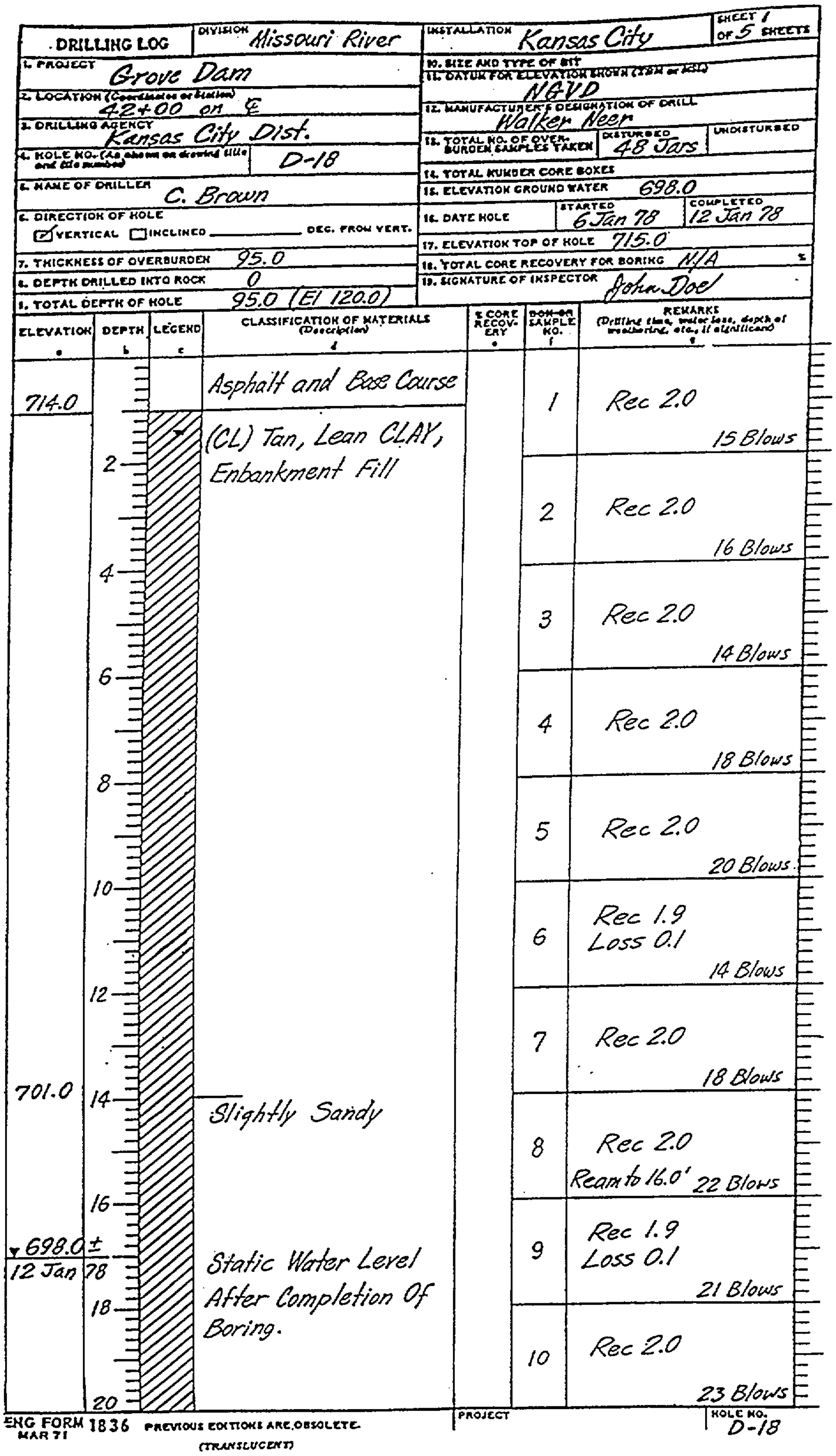
DRILLING LOG | DIVISION SAD | INSTALLATION BAS | SHEET 2 OF 2 SHEETS

1. PROJECT Raymond AFB, Airmens Dorm
2. LOCATION (Coordinates or Station)
3. DRILLING AGENCY
4. HOLE NO. (As shown on drawing title and file number) AD-6
5. NAME OF DRILLER
6. DIRECTION OF HOLE ☐ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN
8. DEPTH DRILLED INTO ROCK
9. TOTAL DEPTH OF HOLE
10. SIZE AND TYPE OF BIT
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)
12. MANUFACTURER'S DESIGNATION OF DRILL
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN | DISTURBED | UNDISTURBED
14. TOTAL NUMBER CORE BOXES
15. ELEVATION GROUND WATER
16. DATE HOLE | STARTED | COMPLETED
17. ELEVATION TOP OF HOLE 25.5
18. TOTAL CORE RECOVERY FOR BORING %
19. SIGNATURE OF INSPECTOR Cindy Watson

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
2.0 1.0 -4.5	22 24 26 28 30		(SP) Light Gray, Poorly Graded SAND, Medium To Coarse. Gravelly (CL) Brownish Gray, Sandy CLAY, Stiff To Hard, Shelly, Moist.		19 NS 20 NS 21 22 NS 23 NS 24 NS 25 NS 26	15 19 6 20 25 10 20 25 11 15 30 10 18 32 11 15 26 8 21 50
			Bottom Of Boring			

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) | PROJECT Raymond AFB, Airmens Dorm | HOLE NO. AD-6

Figure C-1 (concluded). Overburden, disturbed, standard penetration test and auger.



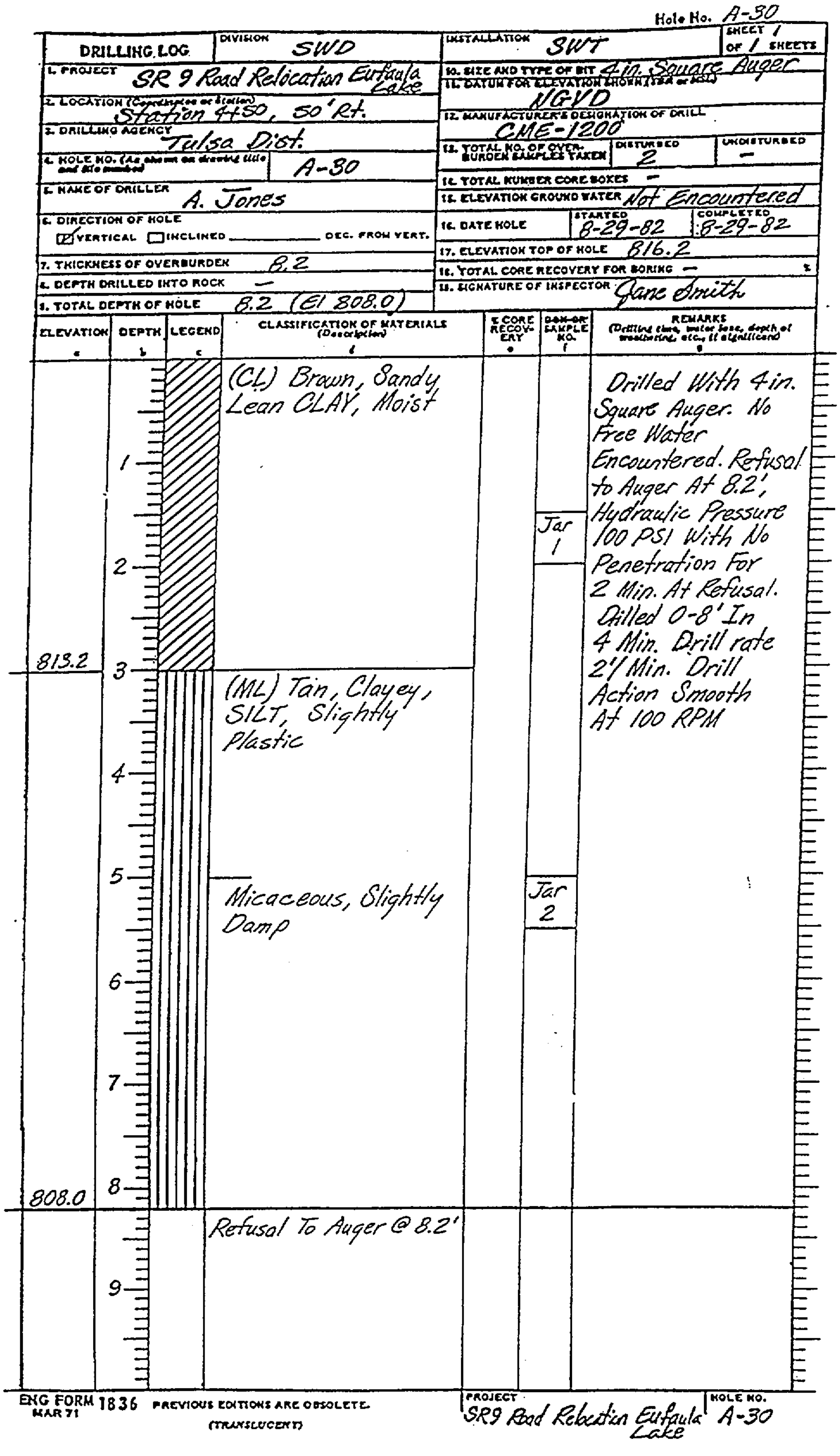
DRILLING LOG
DIVISION Missouri River
INSTALLATION Kansas City
SHEET 1 OF 5 SHEETS

1. PROJECT Grove Dam
2. LOCATION (Coordinates or Station) 42+00 on ℄
3. DRILLING AGENCY Kansas City Dist.
4. HOLE NO. (As shown on drawing title and file number) D-18
5. NAME OF DRILLER C. Brown
6. DIRECTION OF HOLE ☑ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN 95.0
8. DEPTH DRILLED INTO ROCK 0
9. TOTAL DEPTH OF HOLE 95.0 (El 120.0)
10. SIZE AND TYPE OF BIT
11. DATUM FOR ELEVATION SHOWN (TBM or MSL) NGVD
12. MANUFACTURER'S DESIGNATION OF DRILL Walker Neer
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN DISTURBED 48 Jars UNDISTURBED
14. TOTAL NUMBER CORE BOXES
15. ELEVATION GROUND WATER 698.0
16. DATE HOLE STARTED 6 Jan 78 COMPLETED 12 Jan 78
17. ELEVATION TOP OF HOLE 715.0
18. TOTAL CORE RECOVERY FOR BORING N/A %
19. SIGNATURE OF INSPECTOR John Doe

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
714.0		▨	Asphalt and Base Course (CL) Tan, Lean CLAY, Enbankment Fill		1	Rec 2.0 15 Blows
	2	▨			2	Rec 2.0 16 Blows
	4	▨			3	Rec 2.0 14 Blows
	6	▨			4	Rec 2.0 18 Blows
	8	▨			5	Rec 2.0 20 Blows
	10	▨			6	Rec 1.9 Loss 0.1 14 Blows
	12	▨			7	Rec 2.0 18 Blows
701.0	14	▨	Slightly Sandy		8	Rec 2.0 Ream to 16.0' 22 Blows
▼ 698.0± 12 Jan 78	16	▨	Static Water Level After Completion Of Boring.		9	Rec 1.9 Loss 0.1 21 Blows
	18 20	▨			10	Rec 2.0 23 Blows

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)
PROJECT
HOLE NO. D-18

Figure C-2. Overburden, disturbed, drive.



Hole No. A-30

DRILLING LOG	DIVISION SWD	INSTALLATION SWT
		SHEET 1 OF 1 SHEETS

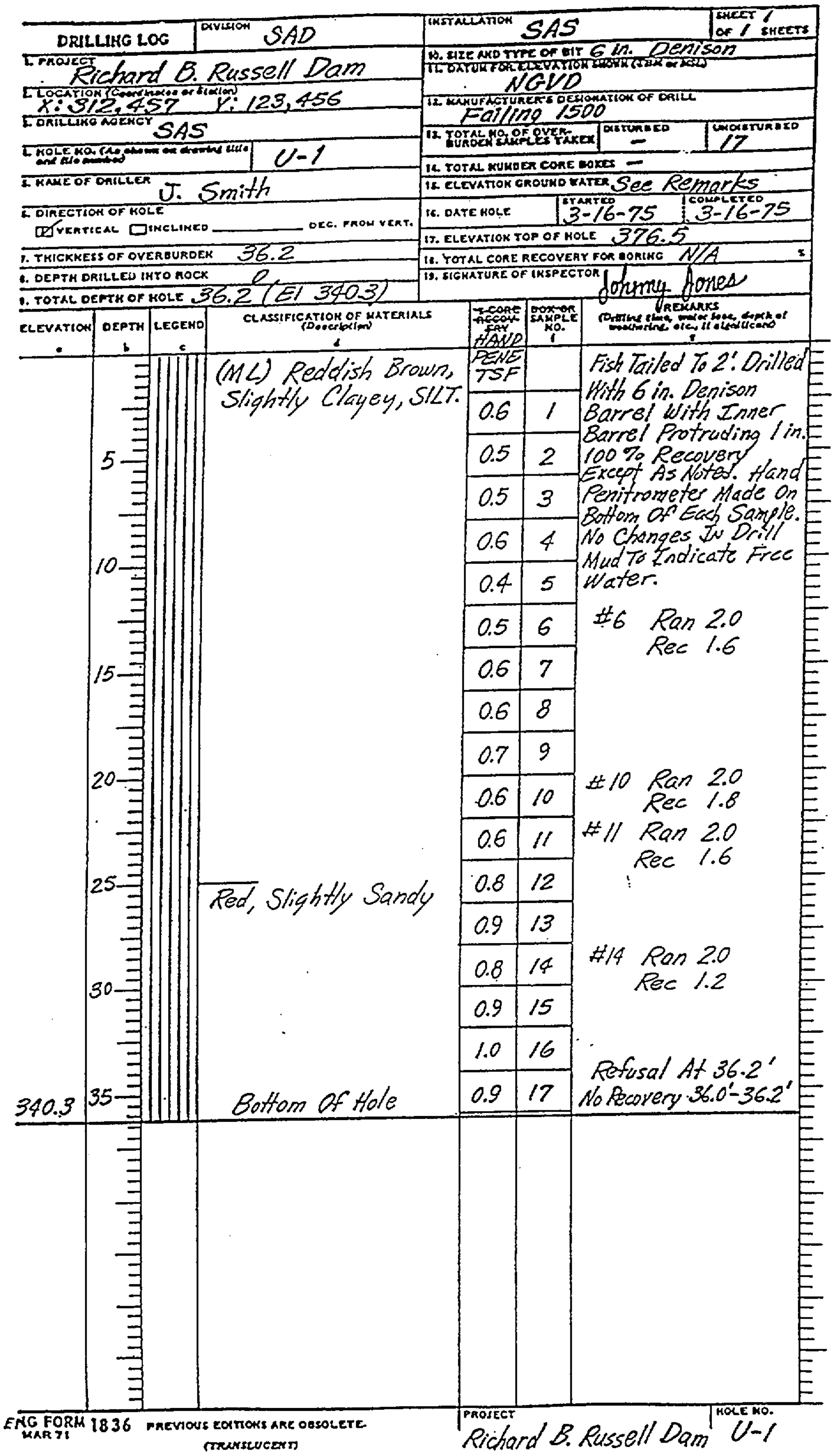
1. PROJECT SR 9 Road Relocation Eufaula Lake
2. LOCATION (Coordinates or Station) Station 4450, 50' Rt.
3. DRILLING AGENCY Tulsa Dist.
4. HOLE NO. (As shown on drawing title and file number) A-30
5. NAME OF DRILLER A. Jones
6. DIRECTION OF HOLE ☑ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN 8.2
8. DEPTH DRILLED INTO ROCK —
9. TOTAL DEPTH OF HOLE 8.2 (El 808.0)
10. SIZE AND TYPE OF BIT 4 in. Square Auger
11. DATUM FOR ELEVATION SHOWN (TBM or MSL) NGVD
12. MANUFACTURER'S DESIGNATION OF DRILL CME-1200
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN DISTURBED 2 UNDISTURBED —
14. TOTAL NUMBER CORE BOXES —
15. ELEVATION GROUND WATER Not Encountered
16. DATE HOLE STARTED 8-29-82 COMPLETED 8-29-82
17. ELEVATION TOP OF HOLE 816.2
18. TOTAL CORE RECOVERY FOR BORING — %
19. SIGNATURE OF INSPECTOR Jane Smith

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		▨	(CL) Brown, Sandy Lean CLAY, Moist			Drilled With 4in. Square Auger. No Free Water Encountered. Refusal to Auger At 8.2', Hydraulic Pressure 100 PSI With No Penetration For 2 Min. At Refusal. Drilled 0-8' In 4 Min. Drill rate 2'/Min. Drill Action Smooth At 100 RPM
					Jar 1	
813.2	3	▥	(ML) Tan, Clayey, SILT, Slightly Plastic			
	5		Micaceous, Slightly Damp		Jar 2	
808.0			Refusal To Auger @ 8.2'			

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

PROJECT SR9 Road Relocation Eufaula Lake HOLE NO. A-30

Figure C-3. Overburden, disturbed, auger.



DRILLING LOG | DIVISION SAD | INSTALLATION SAS | SHEET 1 OF 1 SHEETS

1. PROJECT Richard B. Russell Dam
2. LOCATION (Coordinates or Station) X: 312,457 Y: 123,456
3. DRILLING AGENCY SAS
4. HOLE NO. (As shown on drawing title and file number) U-1
5. NAME OF DRILLER J. Smith
6. DIRECTION OF HOLE ☑ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN 36.2
8. DEPTH DRILLED INTO ROCK 0
9. TOTAL DEPTH OF HOLE 36.2 (El 340.3)
10. SIZE AND TYPE OF BIT 6 in. Denison
11. DATUM FOR ELEVATION SHOWN (TBM or MSL) NGVD
12. MANUFACTURER'S DESIGNATION OF DRILL Failing 1500
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN DISTURBED — UNDISTURBED 17
14. TOTAL NUMBER CORE BOXES —
15. ELEVATION GROUND WATER See Remarks
16. DATE HOLE STARTED 3-16-75 COMPLETED 3-16-75
17. ELEVATION TOP OF HOLE 376.5
18. TOTAL CORE RECOVERY FOR BORING N/A %
19. SIGNATURE OF INSPECTOR Johnny Jones

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	~~% CORE RECOVERY~~ HAND PENE TSF e	~~BOX OR~~ SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			(ML) Reddish Brown, Slightly Clayey, SILT.	0.6	1	Fish Tailed To 2'. Drilled With 6 in. Denison Barrel With Inner Barrel Protruding 1 in. 100 % Recovery Except As Noted. Hand Penitrometer Made On Bottom Of Each Sample. No Changes In Drill Mud To Indicate Free Water.
	5			0.5	2	
				0.5	3	
				0.6	4	
	10			0.4	5	
				0.5	6	#6 Ran 2.0 Rec 1.6
	15			0.6	7	
				0.6	8	
				0.7	9	
	20			0.6	10	#10 Ran 2.0 Rec 1.8
				0.6	11	#11 Ran 2.0 Rec 1.6
	25		Red, Slightly Sandy	0.8	12	
				0.9	13	
				0.8	14	#14 Ran 2.0 Rec 1.2
	30			0.9	15	
				1.0	16	Refusal At 36.2'
340.3	35		Bottom Of Hole	0.9	17	No Recovery 36.0'-36.2'

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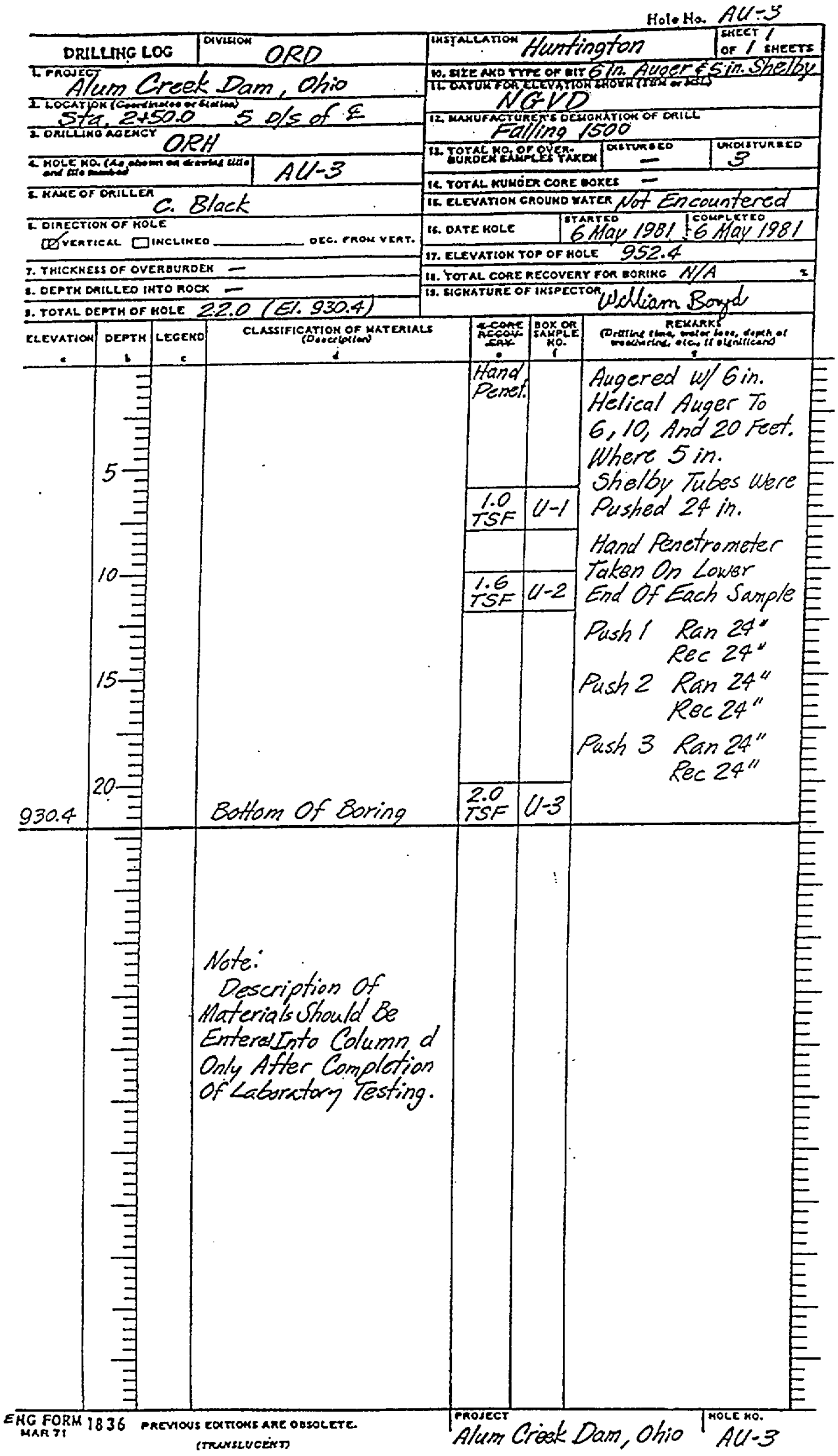
PROJECT Richard B. Russell Dam | HOLE NO. U-1

Figure C-4. Overburden, undisturbed, Denison.



Hole No. AU-3

DRILLING LOG	DIVISION ORD	INSTALLATION Huntington	SHEET 1 OF 1 SHEETS
1. PROJECT Alum Creek Dam, Ohio		10. SIZE AND TYPE OF BIT 6 in. Auger & 5 in. Shelby	
2. LOCATION (Coordinates or Station) Sta. 2+50.0 5 d/s of ℄		11. DATUM FOR ELEVATION SHOWN (TBM or MSL) NGVD	
3. DRILLING AGENCY ORH		12. MANUFACTURER'S DESIGNATION OF DRILL Falling 1500	
4. HOLE NO. (As shown on drawing title and file number)	AU-3	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN	DISTURBED — UNDISTURBED 3
5. NAME OF DRILLER C. Black		14. TOTAL NUMBER CORE BOXES —	
6. DIRECTION OF HOLE ☑ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.		15. ELEVATION GROUND WATER Not Encountered	
		16. DATE HOLE	STARTED 6 May 1981 COMPLETED 6 May 1981
7. THICKNESS OF OVERBURDEN —		17. ELEVATION TOP OF HOLE 952.4	
8. DEPTH DRILLED INTO ROCK —		18. TOTAL CORE RECOVERY FOR BORING N/A %	
9. TOTAL DEPTH OF HOLE 22.0 (El. 930.4)		19. SIGNATURE OF INSPECTOR William Boyd	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
				Hand Penet.		Augered w/ 6 in. Helical Auger To 6, 10, And 20 Feet. Where 5 in. Shelby Tubes Were Pushed 24 in.
	5			1.0 TSF	U-1	Hand Penetrometer Taken On Lower End Of Each Sample
	10			1.6 TSF	U-2	Push 1 Ran 24" Rec 24"
	15					Push 2 Ran 24" Rec 24"
						Push 3 Ran 24" Rec 24"
930.4	20		Bottom Of Boring	2.0 TSF	U-3	

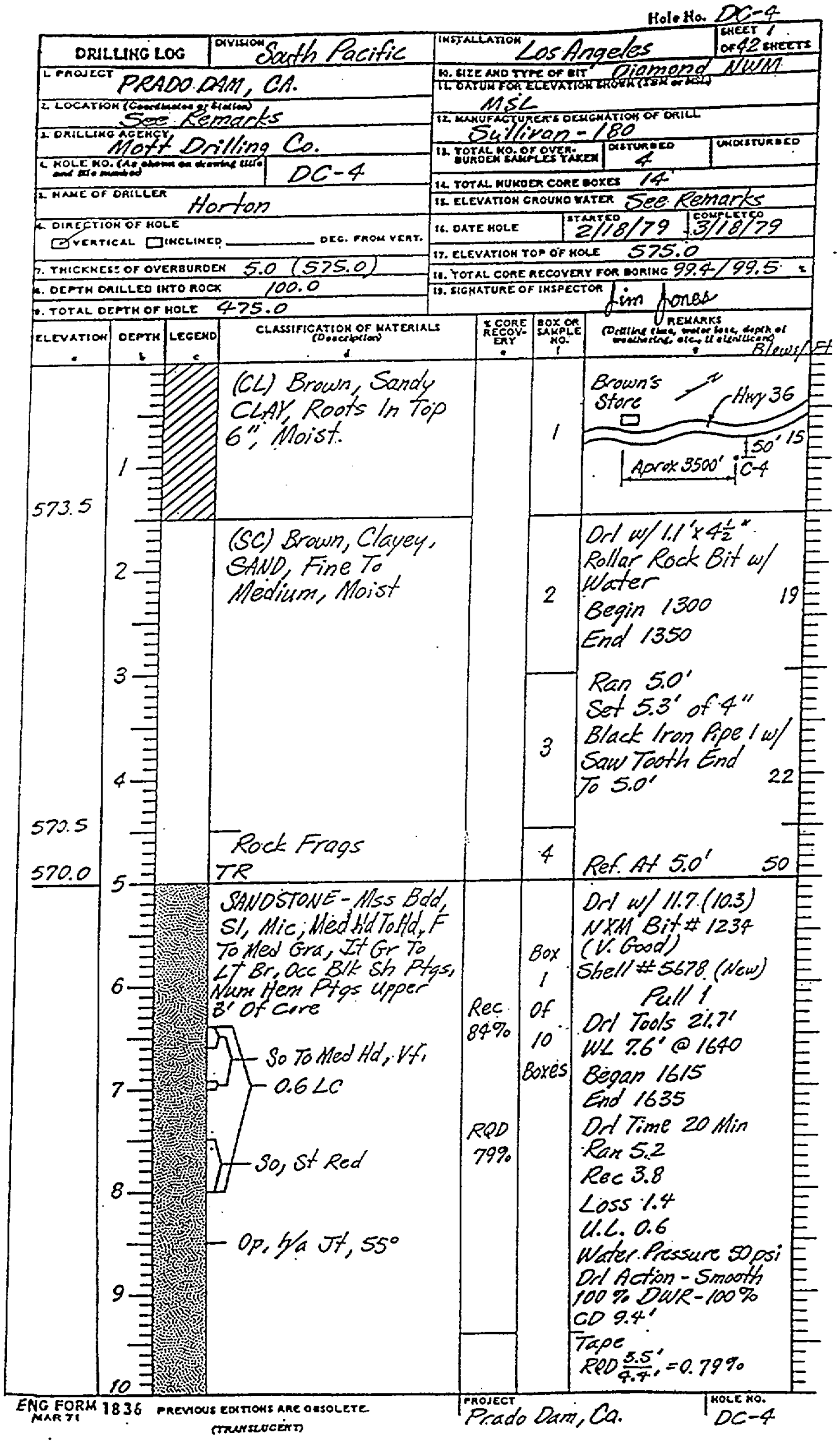
Note:
Description Of Materials Should Be Entered Into Column d Only After Completion Of Laboratory Testing.

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

PROJECT Alum Creek Dam, Ohio

HOLE NO. AU-3

Figure C-5. Overburden, undisturbed, Shelby and auger.



Hole No. DC-4

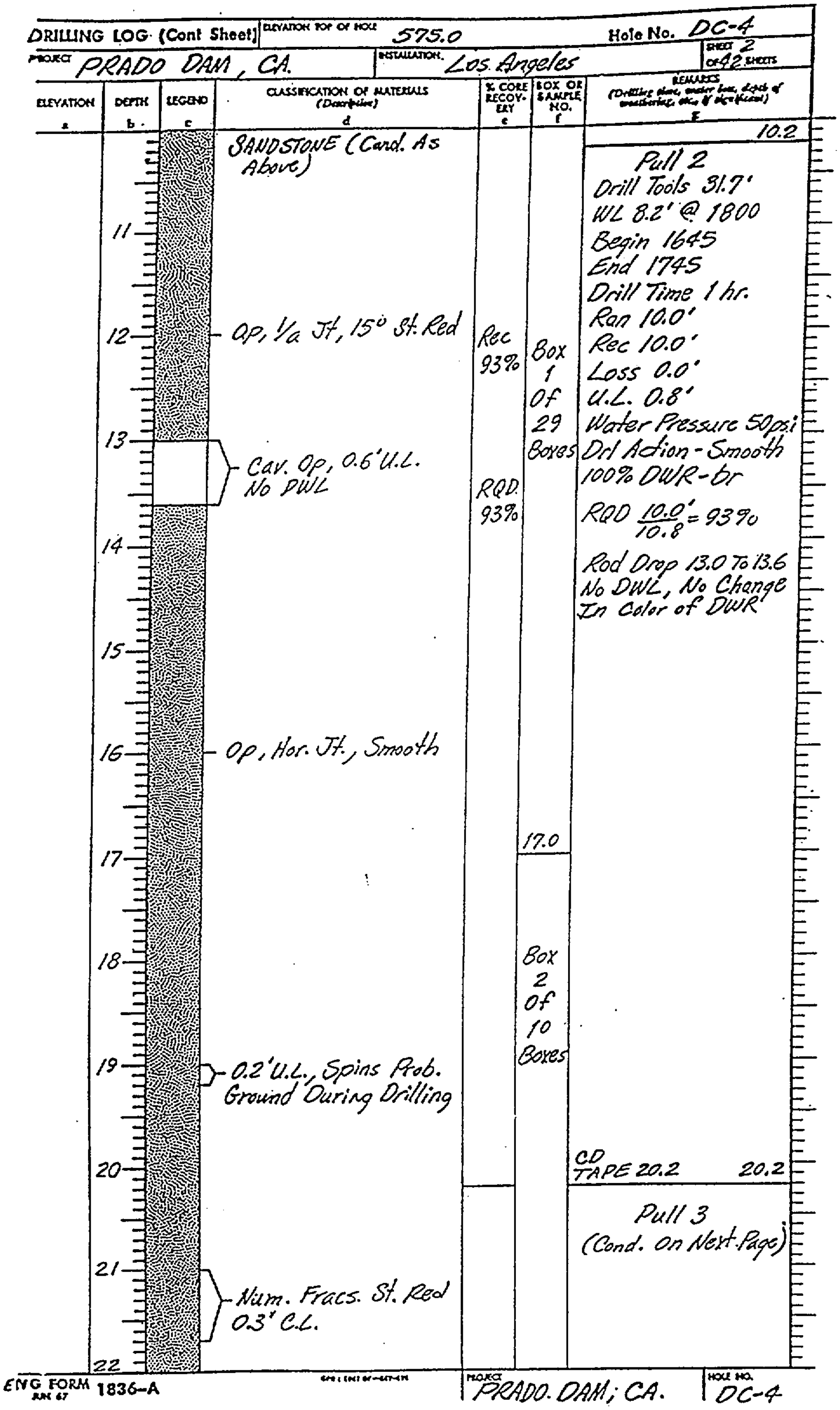
DRILLING LOG
DIVISION South Pacific
INSTALLATION Los Angeles
SHEET 1 OF 42 SHEETS

1. PROJECT PRADO DAM, CA.
2. LOCATION (Coordinates or Station) See Remarks
3. DRILLING AGENCY Mott Drilling Co.
4. HOLE NO. (As shown on drawing title and file number) DC-4
5. NAME OF DRILLER Horton
6. DIRECTION OF HOLE ☑ VERTICAL ☐ INCLINED ________ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN 5.0 (575.0)
8. DEPTH DRILLED INTO ROCK 100.0
9. TOTAL DEPTH OF HOLE 475.0
10. SIZE AND TYPE OF BIT Diamond NWM
11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL
12. MANUFACTURER'S DESIGNATION OF DRILL Sullivan-180
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN DISTURBED 4 UNDISTURBED
14. TOTAL NUMBER CORE BOXES 14
15. ELEVATION GROUND WATER See Remarks
16. DATE HOLE STARTED 2/18/79 COMPLETED 3/18/79
17. ELEVATION TOP OF HOLE 575.0
18. TOTAL CORE RECOVERY FOR BORING 99.4/99.5 %
19. SIGNATURE OF INSPECTOR Jim Jones

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	Blows/Ft
573.5	1	▨	(CL) Brown, Sandy CLAY, Roots In Top 6", Moist.		1	Brown's Store Hwy 36 50' Aprox 3500' C-4	15
	2		(SC) Brown, Clayey, SAND, Fine To Medium, Moist		2	Drl w/ 1.1' x 4½" Rollar Rock Bit w/ Water Begin 1300 End 1350	19
570.5	3 4				3	Ran 5.0' Set 5.3' of 4" Black Iron Pipe I w/ Saw Tooth End To 5.0'	22
570.0			Rock Frags TR		4	Ref. At 5.0'	50
	5 6 7 8 9 10	░	SANDSTONE - Mss Bdd, Sl, Mic; Med Hd To Hd, F To Med Gra, Lt Gr To Lt Br, Occ Blk Sh Ptgs, Num Hem Ptgs Upper 3' Of Core So To Med Hd, Vf, 0.6 LC So, St Red Op, b/a Jt, 55°	Rec 84% RQD 79%	Box 1 Of 10 Boxes	Drl w/ 11.7 (10.3) NXM Bit # 1234 (V. Good) Shell # 5678 (New) Pull 1 Drl Tools 21.7' WL 7.6' @ 1640 Began 1615 End 1635 Drl Time 20 Min Ran 5.2 Rec 3.8 Loss 1.4 U.L. 0.6 Water Pressure 50 psi Drl Action - Smooth 100% DWR - 100% CD 9.4' Tape RQD $\frac{5.5'}{9.4'}$ = 0.79%	

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)
PROJECT Prado Dam, Ca.
HOLE NO. DC-4

Figure C-6. Bedrock, disturbed, SPT and core.



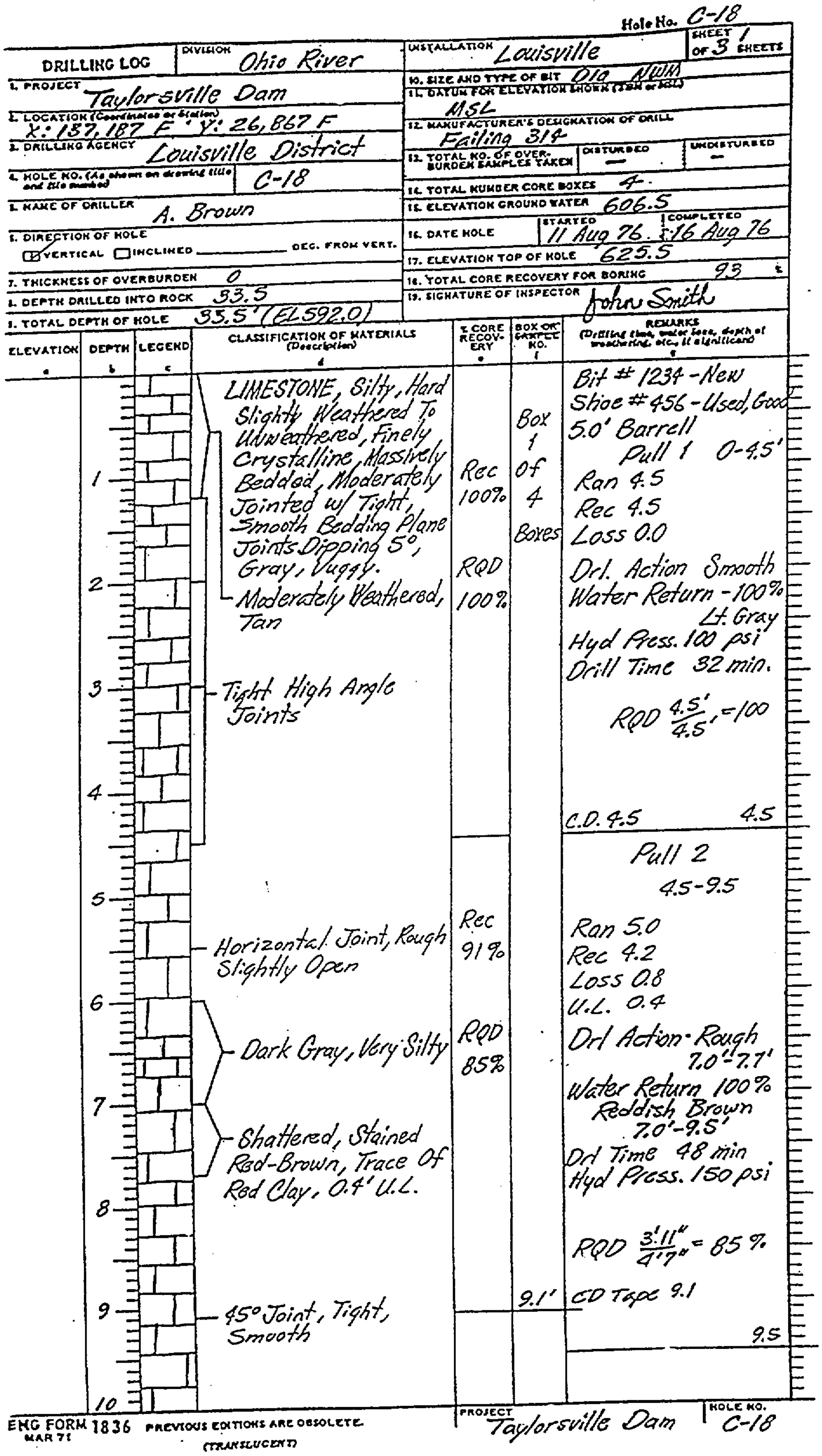
DRILLING LOG (Cont Sheet) ELEVATION TOP OF HOLE 575.0 Hole No. DC-4

PROJECT PRADO DAM, CA. INSTALLATION Los Angeles SHEET 2 OF 42 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			SANDSTONE (Cond. As Above)			10.2
						Pull 2 Drill Tools 31.7' WL 8.2' @ 1800 Begin 1645 End 1745 Drill Time 1 hr. Ran 10.0' Rec 10.0' Loss 0.0' U.L. 0.8' Water Pressure 50psi Drl Action - Smooth 100% DWR-br
	11					
	12		Op, 1/4 Jt, 15° St. Red	Rec 93%	Box 1 Of 29 Boxes	
	13		Cav. Op, 0.6' U.L. No DWL	RQD 93%		RQD $\frac{10.0'}{10.8}$ = 93%
	14					Rod Drop 13.0 To 13.6 No DWL, No Change In Color of DWR
	15					
	16		Op, Hor. Jt., Smooth			
	17				17.0	
	18				Box 2 Of 10 Boxes	
	19		0.2' U.L., Spins Prob. Ground During Drilling			
	20					CD TAPE 20.2 20.2
						Pull 3 (Cond. on Next Page)
	21		Num. Fracs. St. Red 0.3' C.L.			
	22					

ENG FORM 1836-A JUN 67 PROJECT PRADO DAM; CA. HOLE NO. DC-4

Figure C-6 (concluded). Bedrock, disturbed, SPT and core.



Hole No. C-18

DRILLING LOG DIVISION Ohio River INSTALLATION Louisville SHEET 1 OF 3 SHEETS

1. PROJECT Taylorsville Dam
2. LOCATION (Coordinates or Station) X: 157,187 F ' Y: 26,867 F
3. DRILLING AGENCY Louisville District
4. HOLE NO. (As shown on drawing title and file number) C-18
5. NAME OF DRILLER A. Brown
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN 0
8. DEPTH DRILLED INTO ROCK 33.5
9. TOTAL DEPTH OF HOLE 33.5' (EL592.0)
10. SIZE AND TYPE OF BIT DIA NWM
11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL
12. MANUFACTURER'S DESIGNATION OF DRILL Failing 314
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN DISTURBED — UNDISTURBED —
14. TOTAL NUMBER CORE BOXES 4
15. ELEVATION GROUND WATER 606.5
16. DATE HOLE STARTED 11 Aug 76 COMPLETED 16 Aug 76
17. ELEVATION TOP OF HOLE 625.5
18. TOTAL CORE RECOVERY FOR BORING 93 %
19. SIGNATURE OF INSPECTOR John Smith

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			LIMESTONE, Silty, Hard Slightly Weathered To Unweathered, Finely Crystalline, Massively Bedded, Moderately Jointed w/ Tight, Smooth Bedding Plane Joints Dipping 5°, Gray, Vuggy.		Box 1 of 4 Boxes	Bit # 1234 - New Shoe # 456 - Used, Good 5.0' Barrell Pull 1 0-4.5'
	1			Rec 100%		Ran 4.5 Rec 4.5 Loss 0.0
	2		Moderately Weathered, Tan	RQD 100%		Drl. Action Smooth Water Return - 100% Lt. Gray Hyd Press. 100 psi Drill Time 32 min.
	3		Tight High Angle Joints			RQD $\frac{4.5'}{4.5'}$ = 100
	4					C.D. 4.5 4.5
						Pull 2 4.5-9.5
	5		Horizontal Joint, Rough Slightly Open	Rec 91%		Ran 5.0 Rec 4.2 Loss 0.8 U.L. 0.4
	6		Dark Gray, Very Silty	RQD 85%		Drl Action - Rough 7.0'-7.7' Water Return 100% Reddish Brown 7.0'-9.5'
	7		Shattered, Stained Red-Brown, Trace Of Red Clay, 0.4' U.L.			Drl Time 48 min Hyd Press. 150 psi
	8					RQD $\frac{3'11''}{4'7''}$ = 85 %
	9		45° Joint, Tight, Smooth		9.1'	CD Tape 9.1 9.5
	10					

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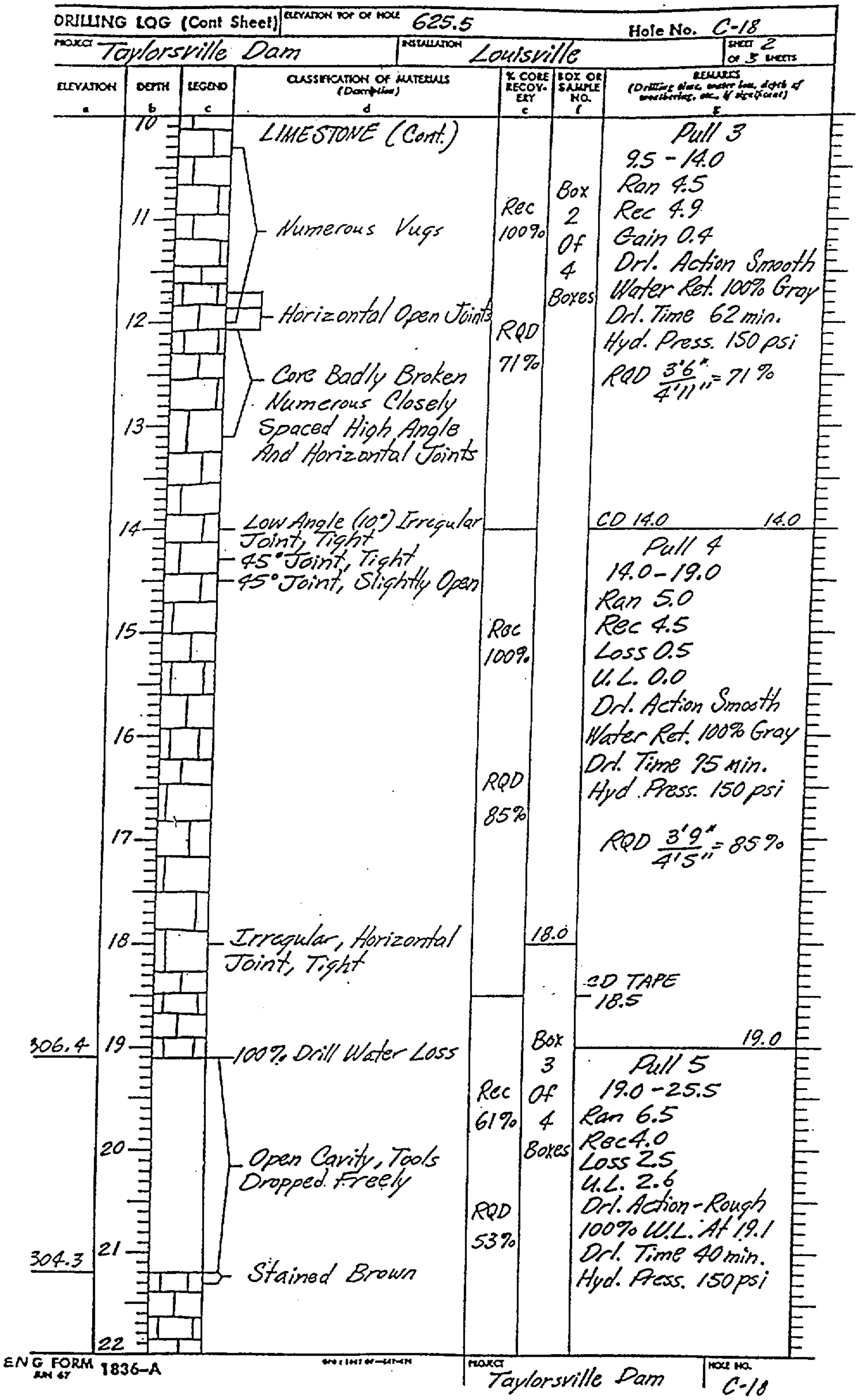
PROJECT Taylorsville Dam HOLE NO. C-18

Figure C-7. Bedrock, core.

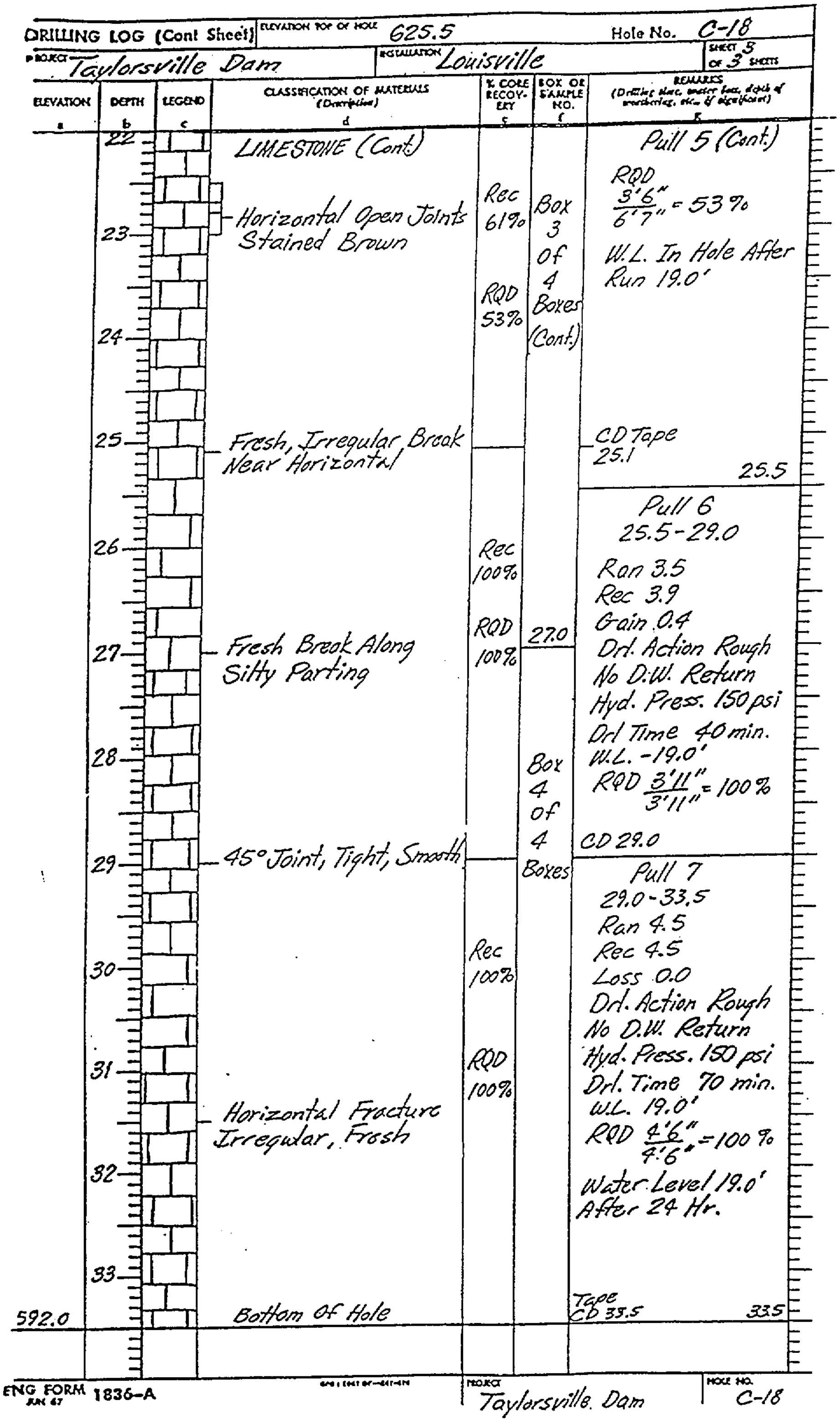


DRILLING LOG (Cont Sheet)
ELEVATION TOP OF HOLE 625.5
Hole No. C-18
PROJECT Taylorsville Dam
INSTALLATION Louisville
SHEET 2 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	10		LIMESTONE (Cont.)			Pull 3
						9.5 - 14.0
						Ran 4.5
	11		Numerous Vugs	Rec 100%	Box 2 Of 4 Boxes	Rec 4.9 Gain 0.4 Drl. Action Smooth Water Ret. 100% Gray
			Horizontal Open Joints			Drl. Time 62 min.
	12		Core Badly Broken Numerous Closely Spaced High Angle And Horizontal Joints	RQD 71%		Hyd. Press. 150 psi RQD $\frac{3'6''}{4'11''}$ = 71 %
	13					
	14		Low Angle (10°) Irregular Joint, Tight			CD 14.0 14.0
			45° Joint, Tight			Pull 4
			45° Joint, Slightly Open			14.0 - 19.0
	15			Rec 100%		Ran 5.0 Rec 4.5 Loss 0.5 U.L. 0.0 Drl. Action Smooth
	16			RQD 85%		Water Ret. 100% Gray Drl. Time 95 min. Hyd. Press. 150 psi
	17					RQD $\frac{3'9''}{4'5''}$ = 85 %
	18		Irregular, Horizontal Joint, Tight		18.0	CD TAPE 18.5
306.4	19		100% Drill Water Loss		Box 3 Of 4 Boxes	19.0 Pull 5 19.0 - 25.5
	20		Open Cavity, Tools Dropped Freely	Rec 61%		Ran 6.5 Rec 4.0 Loss 2.5 U.L. 2.6
304.3	21		Stained Brown	RQD 53%		Drl. Action - Rough 100% W.L. At 19.1 Drl. Time 40 min. Hyd. Press. 150 psi
	22					

ENG FORM 1836-A JUN 67
PROJECT Taylorsville Dam
HOLE NO. C-18

Figure C-7 (continued). Bedrock, core.



DRILLING LOG (Cont Sheet) ELEVATION TOP OF HOLE 625.5 Hole No. C-18

PROJECT Taylorsville Dam INSTALLATION Louisville SHEET 3 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	22		LIMESTONE (Cont.)			Pull 5 (Cont.)
	23		Horizontal Open Joints Stained Brown	Rec 61% RQD 53%	Box 3 of 4 Boxes (Cont.)	RQD $\frac{3'6''}{6'7''}$ = 53% W.L. In Hole After Run 19.0'
	24					
	25		Fresh, Irregular Break Near Horizontal			CD Tape 25.1 25.5
	26			Rec 100%		Pull 6 25.5-29.0 Ran 3.5 Rec 3.9 Gain 0.4
	27		Fresh Break Along Silty Parting	RQD 100%	27.0	Drl. Action Rough No D.W. Return Hyd. Press. 150 psi Drl Time 40 min.
	28				Box 4 of 4 Boxes	W.L. -19.0' RQD $\frac{3'11''}{3'11''}$ = 100%
	29		45° Joint, Tight, Smooth			CD 29.0
				Rec 100%		Pull 7 29.0-33.5 Ran 4.5 Rec 4.5 Loss 0.0
	30					Drl. Action Rough No D.W. Return Hyd. Press. 150 psi
	31		Horizontal Fracture Irregular, Fresh	RQD 100%		Drl. Time 70 min. W.L. 19.0' RQD $\frac{4'6''}{4'6''}$ = 100%
	32					Water Level 19.0' After 24 Hr.
	33					
592.0			Bottom Of Hole			Tape CD 33.5 33.5

ENG FORM 1836-A JUN 67 PROJECT Taylorsville Dam HOLE NO. C-18

Figure C-7 (concluded). Bedrock, core.

APPENDIX D
METHODS OF SUBSURFACE EXPLORATION[1]

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[1] Department of the Army, 1984.

APPENDIX D
METHODS OF SUBSURFACE EXPLORATION[1]

METHOD	PROCEDURE	APPLICABILITY	LIMITATIONS
1. Methods of Access for Sampling, Test, or Observation			
Pits, Trenches, Shafts, Tunnels	Excavation made by hand, large auger, or digging machinery.	Visual observation, photography, disturbed and undisturbed sampling, in situ testing of soil and rock.	Depth of unprotected excavations is limited by groundwater or safety considerations. May need dewatering.
Auger Boring	Boring advanced by hand auger or power auger.	Recovery of remolded samples and determining groundwater levels. Access for undisturbed sampling of cohesive soils.	Will not penetrated boulders or most rock.
Hollow Stem Auger Boring	Boring advanced by means of Continuous-flight helix auger with hollow center stem.	Access of undisturbed or representative sampling through hollow stem with thin-wall tube sampler, core barrel, or spilt-barrel sampler.	Should not be used with plug in coarse-grained soils. Not suitable for undisturbed sampling in loose sand or silt.
Wash Boring	Boring advanced by chopping with light bit and by jetting with upward-deflected jet.	Cleaning out and advancing hole in soil between sample intervals.	Suitable for use with sampling operations in soil only if done with low water velocities and with up-ward-deflected jet.
Rotary Drilling	Boring advanced by rotating drilling bit; cuttings removed by circulating drilling fluid.	Cleaning out and advancing hole in soil or rock between sample intervals.	Drilling mud should be used in coarse-grained soils. Bottom discharge bits are not suitable for use with undisturbed sampling in soils unless combined with protruding core barrel, as in Denison sampler, or with upward-deflected jets.
Percussion Drilling	Boring advanced by air-operated Impact hammer.	Detection of voids and zones of weakness in rock by changes in drill rate or resistance. Access for in situ testing or logging.	Not suitable for use in soils.
Cable Drilling	Boring advanced by repeated dropping of heavy bit; removal of cuttings by bailing.	Advancing hole in soil or rock. Access for sampling, in situ testing, or logging in rock. Penetration of hard layers, gravel, or boulders in auger borings.	Causes severe disturbance in soils; not suitable for use with undisturbed sampling methods.
Continuous Sampling or Displacement Boring	Boring advanced by repeated pushing of sampler or closed sampler is pushed to desired depth and sample is taken.	Recovery of representative samples of cohesive soils and undisturbed samples in some cohesive soils.	Effects of advance and withdrawal of sampler result in disturbed sections at top and bottom of sample. In some soils, entire sample may be disturbed. Best suited for use in cohesive soils. Continuous sampling in cohesionless soils may be made by successive reaming and cleaning of hole between sampling.

(Continued)

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[1] Department of the Army, 1984.

APPENDIX D (Continued)
METHODS OF SUBSURFACE EXPLORATION

METHOD	PROCEDURE	APPLICABILITY	LIMITATIONS
2. Methods of Sampling Soil and Rock			
Hand-Cut Block or Cylindrical Sample	Sample is cut by hand from soil exposed in excavation.	Highest quality undisturbed samples in all soils and in soft rock.	Requires accessible excavation and dewatering if below water table. Extreme care is required in sampling cohesionless soils.
Fixed-Piston Sampler	Thin-walled tube is pushed into soil, with fixed piston in contact with top of sample during push.	Undisturbed samples in cohesive soils, silts, and sands above or below the water table.	Some types do not have a positive means to prevent piston movement.
Hydraulic Piston Sampler (Osterberg Sampler)	Thin-walled tube is pushed into soil by hydraulic pressure. Fixed piston in contact with top of sample during push.	Undisturbed samples in cohesive soils, silts, and sands above or below the water table.	Not possible to determine amount of sampler penetration during push. Does not have vacuum- breaker in piston.
Free-Piston Sampler	Thin-walled tube is pushed into soil, with. Piston rests on top of soil sample during push.	Undisturbed samples in stiff cohesive soils. Representative samples in soft-to-medium cohesive soils and silts.	May not be suitable for sampling in cohesionless soils. Free piston provides no control of specific recovery ratio.
Open Drive Sampler	Thin-walled, open tube is pushed into soil.	Undisturbed samples in stiff cohesive soils. Representative samples in soft-to-medium cohesive soils and silts.	Small diameter of tubes may not be suitable for sampling in cohesionless soils or for undisturbed sampling in uncased boreholes. No control of specific recovery ratio.
Swedish Foil Sampler	Sample tube is pushed into soil while stainless steel strips unrolling from spools envelop sample. Piston, fixed by chain from surface, maintains contact with top of sample.	Continuous undisturbed samples up to 66 feet (20 m) long in very soft to soft clays.	Not suitable for use in soils containing gravel, sand layers, or shells, which may rupture foils and damage samples. Difficulty may be encountered in alternating hard and soft layers with squeezing of soft layers and reduction in thickness. Requires experienced operator.
Pitcher Sampler	Thin-walled tube is pushed into by spring above sampler while outer core bit reams hole. Cuttings removed by circulating drilling fluid.	Undisturbed samples in stiff, hard, brittle, cohesive soils and sands with cementation and in soft rock. Effective in sampling alternating hard and soft layers. Representative samples in soft-to-medium cohesive soils and silts. Disturbed samples may be obtained in cohesionless materials with variable success.	Frequently ineffective in cohesion less soils.
Spilt-Barrel or Split Spoon Sampler	Spilt-barrel tube is driven into soil by blows of falling ram. Sampling is carried out in conjunction with Standard Penetration Test.	Representative samples in soils other than coarse-grained soils.	Samples are disturbed and not suitable for tests of physical properties.

(Continued)

APPENDIX D (Continued)
METHODS OF SUBSURFACE EXPLORATION

METHOD	PROCEDURE	APPLICABILITY	LIMITATIONS
2. Methods of Sampling Soil and Rock (Continued)			
Auger Sampling	Auger drill used to advance hole is withdrawn at intervals for recovery of soil samples from auger flights.	Determine boundaries of soil layers and obtain samples of soil classification.	Samples not suitable for physical properties or density tests. Large errors in locating strata boundaries may occur without close attention to details of procedure. In some soils, particle breakdown by auger or sorting effects may result in errors in determining gradation.
Rotary Core Barrel	Hole is advanced by core bit while core sample is retained within core barrel or within stationary inner tube. Cuttings removed by drilling fluid.	Core samples in competent rock and hard soils with single-tube core barrel. Core samples in poor or broken rock may be obtainable with double-tube core barrel with bottom discharge bit.	Because recovery is poorest in zones of weakness, samples generally fail to yield positive information on soft seams, joints, or other defects in rocks.
Denison Sampler	Hole is advanced and reamed by core drill while sample is retained in nonrotating inner core barrel with corecatcher. Cuttings removed by circulating drilling fluid.	Undisturbed samples in stiff-to-hard cohesive soil soil, sand with cementation, and soft rocks. Disturbed sample may be obtained in cohesionless materials with variable success.	Not suitable for undisturbed sampling in loose cohesionless soils or soft cohesive soils. Difficulties may be experienced in sampling alternating hard and soft layers.
Shot Core Boring (Calyx)	Boring advanced by rotating single core barrel, which cuts by grinding with chilled steel shot fed with circulating wash water. Used shot and coarser cuttings are deposited in an annular cup, or calyx, above the core barrel.	Large-diameter cores and accessible boreholes in rock.	Cannot be used in drilling at large angles to the vertical. Often ineffective in securing small-diameter cores.
Oriented Integral Sampling	Reinforcing rod is grouted into small-diameter hole, then overcored to obtain an annular core sample.	Core samples in rock with preservation of joints and other zones of weakness.	Samples are not well suited to tests of physical properties.
Wash Sampling or Cuttings Sampling	Cuttings are recovered from wash water or drilling fluid.	Samples useful in conjunction with other data for identification of major strata.	Sample quality is not adequate for site investigations for nuclear facilities.
Submersible Vibratory (Vibracore) Sampler	Core tube is driven into soil by vibrator.	Continuous representative samples in unconsolidated marine sediments.	Because of high area ratio and effects of vibration, samples may be disturbed.
Underwater Piston Corer	Core tube attached to drop weight is driven into soil by gravity after a free fall of controlled ____________________.	Representative samples in unconsolidated marine sediments.	Samples may be seriously disturbed. height. Cable-supported piston remains in contact with soil surface during drive.
Gravity Corer	Open core tube attached to drop weight is driven into soil by gravity after free fall.	Representative samples at shallow depth in unconsolidated marine sediments.	No control of specific recovery ratio. Samples are disturbed.

(Continued)

APPENDIX D (Continued)
METHODS OF SUBSURFACE EXPLORATION

METHOD	PROCEDURE	APPLICABILITY	LIMITATIONS
3. Methods of In Situ Testing of Soil and Rock[2]			
Standard Penetration Test	Split-barrel sampler is driven into soil by blows of free falling weight. Blow count for each 6 in. (15 cm) of penetration is recorded.	Blow count may be used as an index of consistency or density of soil. May be used for detection of changes in consistency or density in clay or sands. May be used with empirical relationships to estimate relative density of clean sand.	Extremely unreliable in silts, silty sands, or soils containing gravel. In sands below water table, positive head must be maintained in borehole. Determination of relative density in sands requires site-specific correlation or highly conservative use of published correlations. Results are sensitive to details of apparatus and procedure.
Cone Penetrometer Test	Steel cone is pushed into soil and followed by subsequent advance of friction sleeve. Resistance is measured during both phases of advance.	Detection of changes in consistency or relative density in clays or sands. Used to estimate static undrained shear strength of clay. Used with empirical relationships to obtain estimate of static compressibility of sand.	Strength estimates require onsite verification by other methods of testing.
Field Vane Shear Test	Four-bladed vane is pushed into undisturbed soil, then rotated to cause shear failure on cylindrical surface. Torsional resistance versus angular deflection is recorded.	Used to estimate in situ undrained shear strength and sensitivity of clays.	Not suitable for use in silt, sand, or soils containing appreciable amounts of gravel or shells. May yield unconservative estimates of shear strength in fissured clay soils or where strength is strain-rate dependent.
Drive-Point Penetrometer	Expendable steel cone is driven into soil by blows of falling weight. Blow count versus penetration is recorded.	Detection of gross changes in consistency or relative density. May be used in some coarse-grained soils.	Provides no quantitative information on soil properties.
Plate Bearing Test (Soil)	Steel loading plate is placed on horizontal surface and is statically loaded, usually by hydraulic jack. Settlement versus time is recorded for each load increment.	Estimation of strength and moduli of soil. May be used at ground surface, in excavations, or in boreholes.	Results can be extrapolated to loaded areas larger than bearing plate only if properties of soil are uniform laterally and with depth.
Plate Bearing Test or Plate Jacking Test (Rock)	Bearing pad on rock surface is statically loaded by hydraulic jack. Deflection versus load is recorded.	Estimation of elastic moduli of rocks masses. May be used at ground surface, in excavations, in tunnels, or in boreholes.	Results can be extrapolated to loaded areas larger than bearing pad only if rock properties are uniform over volume of interest and if diameter of bearing pad is larger than average spacing of joints or other discontinuities.

(Continued)

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[2]

APPENDIX D (Continued)
METHODS OF SUBSURFACE EXPLORATION

METHOD	PROCEDURE	APPLICABILITY	LIMITATIONS
3. Methods of In Situ Testing of Soil and Rock (Continued)			
Pressure Meter Test (Dilatometer Test)	Uniform radial pressure is applied hydraulically over a length of borehole several times its diameter. Change in diameter versus pressure is recorded.	Estimation of elastic moduli of rocks and estimation of shear strengths and compressibility of soils by empirical relationships.	Test results represent properties only of materials in near vicinity of borehole. Results may be misleading in testing materials whose properties may be anisotropic.
Field Pumping Test	Water is pumped from or into aquifer at constant rate through penetrating well. Change in piezometric level is measured at well and at one or more observation wells. Pumping pressures and flow rates are recorded. Packers may be used for pump-in pressure tests.	Estimation of in situ permeability of soils and rock mass.	Apparent permeability may be greatly influenced by local features. Effective permeability of rock is dependent primarily on frequency and distribution of joints. Test result in rock is representative only to extent that the borehole intersects a sufficient number of joints to be representative of the joint system of the rock mass.
Borehole Field Permeability Test	Water is added to an open-ended pipe casing sunk to desired depth. With constant head tests, constant rate of gravity flow into hole and size of casing of pipe are measured. Variations include applied pressure tests and falling head tests.	Rough approximation of in situ permeability of soils and rock mass.	Pipe casing must be carefully cleaned out just to the bottom of the casing. Clear water must be used or tests may be grossly misleading. Measurement of local permeability only.
Direct Shear Test	Block of in situ rock is isolated to permit shearing along a preselected surface. Normal and shearing loads are applied by jacking. Loads and displacements are recorded.	Measurement of shearing resistance of rock mass in situ.	Tests are costly. Usually variability of rock mass requires a sufficient number of tests to provide statistical control.
Pressure Tunnel Test	Hydraulic pressure is applied to sealed-off length of circular tunnel, and diametral deformations are measured.	Determination of elastic constants of the rock mass in situ.	Volume of rock tested is dependent on tunnel diameter. Cracking due to tensile hoop stresses may affect apparent stiffness of rock.
Radial Jacking Test	Radial pressure is applied to a length of circular tunnel by flat jacks. Diametral deformations are measured.	Same as pressure tunnel test.	Same as pressure tunnel test.
Borehole Jack Test	Load is applied to wall of borehole by two diametrically opposed jacks. Deformations and pressures are recorded.	Determination of elastic modulus of rock in situ. Capable of applying greater pressure than dilatometers.	Apparent stiffness may be affected by development of tension cracks.

(Continued)

APPENDIX D (Continued)
METHODS OF SUBSURFACE EXPLORATION

METHOD	PROCEDURE	APPLICABILITY	LIMITATIONS
3. Methods of In Situ Testing of Soil and Rock (Continued)			
Borehole Deformation Meter	Device for measurement of diameters (deformation meter) is placed in borehole, and hole is overcored to relieve stresses on annular rock core containing deformation meter. Diameters (usually 3) are measured before and after overcoring. Modulus of rock is measured by laboratory tests on core; stresses are computed by elastic theory.	Measurement of absolute stresses in situ.	Stress field is affected by borehole. Analysis subject to limitations of elastic theory. Two boreholes at different orientations are required for determination of complete stress field. Questionable results in rocks with strongly time- dependent properties.
Inclusion Stressmeter	Rigid stress indicating device (stressmeter) is placed in borehole, and hole is overcored to relieve stresses on annular core containing stressmeter. In situ stresses are computed by elastic theory.	Measurement of absolute stresses in situ. Does not require accurate knowledge of rock modulus.	Same as above.
Borehole Strain Gauge	Strain gauge is cemented to bottom (end) of borehole, and gauge is overcored to relieve stresses on core-containing strain gauge. Stresses are computed from resulting strains and from modulus obtained by laboratory tests on core.	Measurement of absolute stresses in situ. Requires only one core drill size.	Same as above.
Flat Jack Test	Slot is drilled in rock surface producing stress relief in adjacent rock. Flat jack is grouted into slot and hydraulically pressurized. Pressure required to reverse deformations produced by stress relief is observed.	Measurement of one component of normal stress in situ. Does not require knowledge of rock modulus.	Stress field is affected by excavation or tunnel. Interpretation of test results subject as assumption that loading and unloading moduli are equal. Questionable results in rock with strongly time-dependent properties.
Hydraulic Fracturing Test	Fluid is pumped into sealed-off portion of borehole with pressure increasing until fracture occurs.	Estimation of minor principal stress.	Affected by anisotropy of tensile strength of rock.
Crosshole Seismic Test	Seismic signal is transmitted from source in one borehole to receiver(s) in other borehole(s), and transit time is recorded.	In situ measurement of compression wave velocity and shear wave velocity in soils and rocks.	Requires deviation survey of boreholes to eliminate errors due to deviation of holes from vertical. Refraction of signal through adjacent high-velocity beds must be considered in interpretation.
Uphole/Downhole Seismic Test	Seismic signal is transmitted between borehole and ground surface, and transit time is recorded.	In situ measurement of compression wave velocity and shear wave velocity in soils and rocks.	Apparent velocity obtained is time-average for all strata between source and receiver.

(Continued)

APPENDIX D (Concluded)
METHODS OF SUBSURFACE EXPLORATION

METHOD	PROCEDURE	APPLICABILITY	LIMITATIONS
3. Methods of In Situ Testing of Soil and Rock (Continued)			
Acoustic Velocity Log	Logging tool contains transmitting transducer and two receiving transducers separated by fixed gage length. Signal is transmitted through rock adjacent to borehole, and transit time over the gage length is recorded as difference in arrival times at the receivers.	Measurement of compression wave velocity. Used primarily in rocks to obtain estimate of porosity.	Results represent only the material immediately adjacent to the borehole. Can be obtained only in uncased, fluid-filled borehole. Use is limited to materials with P-wave velocity greater than that of borehole fluid.
3-D Velocity Log	Logging tool contains transmitting transducer and receiving transducer separated by fixed gage length. Signal is transmitted through rock adjacent to borehole, and wave train at receiver is recorded.	Measurement of compression wave and shear wave velocity ties in rock. Detection of void spaces, open fractures, and zones of weakness.	Results represent only the material immediately adjacent to the borehole. Can be obtained only in uncased, fluid-filled borehole. Correction required for variation in hole size. Use is limited to materials with P-wave velocity greater than that of borehole fluid.
Electrical Resistivity Log	Apparent electrical resistivity of soil or rock in neighborhood of borehole is measured by in-hole logging tool containing one of a wide variety of electrode configurations.	Appropriate combinations of resistivity logs can be used to estimate porosity can degree of water saturation in rocks. In soils, may be used as qualitative indication of changes in void ratio or water content, for correlation of strata between boreholes, and for location of strata boundaries.	Can be obtained only in uncased boreholes. Hole must be fluid filled, or electrodes must be pressed against wall of hole. Apparent resistivity values are strongly affected by changes in hole diameter, strata thickness, resistivity contrast between adjacent strata, resistivity of drilling fluid, etc.
Neutron Log	Neutrons are emitted into rock or soil around borehole by a neutron source in the logging tool, and a detector isolated from the source responds to either slow neutrons or secondary gamma rays. Response of detector is recorded.	Correlation of strata between boreholes and location of strata boundaries. Provides an approximation to water content and can be run in cased or uncased, fluid-filled or empty boreholes.	Because of very strong borehole effects, results are generally not of sufficient accuracy for quantitative engineering uses.
Gamma-Gamma Log ("Density Log")	Gamma rays are emitted into rock around the borehole by a source in the logging tool, and a detector isolated from the source responds to back-scattered gamma rays. Response of detector is recorded.	Estimation of bulk density in rocks, qualitative indication of changes in density of soils. May be run in empty or fluid-filled holes.	Effects of borehole size and density of drilling fluid must be accounted for. Presently not suitable for qualitative estimate of density in soils other than those of "rock-like" character. Cannot be used in cased boreholes.
Borehole Cameras	Film-type or television camera in a suitable protective container is used for observation of walls of borehole.	Detection and mapping of joints, seams, cavities, or other visually observable features in rock. Can be used in empty, uncased holes or in holes filled with clear water.	Results are affected by an condition that affects visibility.

APPENDIX E
SPACING AND DEPTH OF SUBSURFACE EXPLORATIONS FOR SAFETY-RELATED[1] FOUNDATIONS

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[1] Department of the Army, 1984.

APPENDIX E
SPACING AND DEPTH OF SUBSURFACE EXPLORATIONS FOR SAFETY-RELATED[1] FOUNDATIONS

TYPE OF STRUCTURE	SPACING OF BORINGS[2] OR SOUNDINGS	MINIMUM DEPTH OF PENETRATION
General	For favorable, uniform geologic conditions, where continuity of subsurface strata is found, the recommended spacing is as indicated for the type of structure. At least one boring should be at the location of every safety-related structure. Where variable conditions are found, spacing should be smaller, as needed, to obtain a clear picture of soil or rock properties and their variability. Where cavities or other discontinuities of engineering significance may occur, the normal exploratory work should be supplemented by borings or soundings at a spacing small enough to detect such features.	The depth of borings should be determined on the basis of the type of structure and geologic conditions. All borings should be extended to a depth sufficient to define the site geology and to sample all materials that may swell during excavation, may consolidate subsequent to construction, may be unstable under earthquake loading, or whose physical properties would affect foundation behavior or stability. Where soils are very thick, the maximum required depth for engineering purposes, denoted d_{max}, may be taken as the depth at which the change in the vertical stress during or after construction for the combined foundation loading is less than 10 percent of the in situ effective overburden stress. It may be necessary to include in the investigation program several borings to establish the soil model for soil-structure interaction studies. These borings may be required to penetrate depths greater than those depths required for general engineering purposes. Borings should be deep enough to define and evaluate the potential for deep stability problems at the site. Generally, all borings should extend at least 30 feet (9 meters) below the lowest part of the foundation. If competent rock is encountered at lesser depths than those given, borings should penetrate to the greatest depth where discontinuities or zones of weakness or alteration can affect foundations and should penetrate at least 20 feet (6 meters) into sound rock. For weathered shale or soft rock, depths should be as for soils.
Structures including buildings, retaining walls, concrete dams	Principal borings: at least one boring beneath every safety-related structure. For larger, heavier structures, such as the containment and auxiliary buildings, at least one boring per 10,000 ft^2 (900 m^2) (approximately 100-foot (300-meter) spacing). In addition, a number of borings along the periphery, at corners, and other selected locations. One boring per 100 linear feet (30 linear meters) for essentially linear structures.[3]	At least one-fourth of the principal borings and a minimum of one boring per structure to penetrate into sound rock or to a depth equal to d_{max}. Others to a depth below foundation elevation equal to the width of structure or to a depth equal to the foundation depth below the original ground surface, whichever is greater.[3]
Earth dams, dikes, levees, and embankments	Principal borings: one per 100 linear feet (30 linear meters) along axis of structure and at critical locations perpendicular to the axis to establish geological sections with groundwater conditions for analysis.[3]	Principal borings: one per 200 linear feet (60 linear meters) to d_{max}. Others should penetrate all strata whose properties would affect the performance of the foundation. For water-impounding structures, to sufficient depth to define all aquifers and zones of underseepage that could affect the performance of structures.[3]

(Continued)

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[1] As determined by the final locations of safety-related structures and facilities.

[2] Includes shafts or other accessible excavations that meet depth requirements.

[3] Also supplement borings or soundings that are design dependent or necessary to define anomalies, critical conditions, etc.

APPENDIX E (Concluded)
SPACING AND DEPTH OF SUBSURFACE EXPLORATIONS FOR SAFETY-RELATED FOUNDATIONS

TYPE OF STRUCTURE	SPACING OF BORINGS OR SOUNDINGS	MINIMUM DEPTH OF PENETRATION
Deep cuts,[4] canals	Principal borings: one per 200 linear feet (60 linear meters) along the alignment and at critical locations perpendicular to the alignment to establish geologic sections with groundwater conditions for analysis.[3]	Principal borings: one per 200 linear feet (60 linear meters) to penetrate into sound rock or to d_{max}. Others to a depth below the bottom elevation of excavation equal to the depth of cut or to below the lowest potential failure zone of the slope.[3] Borings should penetrate previous strata below which groundwater may influence stability.[2]
Pipelines	Principal borings: This may vary depending on how well site conditions are understood from other plant site borings. For variable conditions, one per 100 linear feet (30 linear meters) for buried pipelines; at least one boring for each footing for pipelines above ground.[5]	Principal borings: For buried pipelines, one of every three to penetrate into sound rock or to d_{max}. Others to 5 times the pipe diameters below the invert elevation. For pipelines above ground, depths as for foundation structures. [3,5]
Tunnels	Principal borings: one per 100 linear feet (30 linear meter),[3] may vary for rock tunnels, depending on rock type and characteristics, and planned exploratory shafts or adits.	Principal borings: one per 200 linear feet (60 linear meters) to penetrate into sound rock or to d_{max}. Others to 5 times the tunnel diameter below the invert elevation.[4,5]
Reservoirs, impoundments	Principal borings: In addition to borings at the locations of dams or dikes, a number of borings should be used to investigate geologic conditions of the reservoir basin. The number and spacing of borings should vary with the largest concentration being near control structures and the coverage decreasing with distance upstream.	Principal borings: at least one-fourth to penetrate that portion of the saturation zone that may influence seepage conditions or stability. Others to a depth of 25 feet (7.6 meters) below reservoir bottom elevation.[5]

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[4] Includes temporary cuts that would affect ultimate site safety.

[5] Supplementary borings or soundings as necessary to define anomalies.

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11. ABSTRACT *(200 words or less)*

This document provides a technical basis for revision of the U.S. Nuclear Regulatory Commission Regulatory Guide 1.132, Site Investigations for Foundations of Nuclear Power Facilities, reflecting current and state-of-the-art techniques related to field site investigations. The report summarizes the processes of acquiring geological, geophysical, geotechnical, and other kinds of relevant information that may affect the construction or performance of a building or other engineered structure at selected sites. Guidance is presented for in situ studies during the various stages of site characterization. Topics range from initial information gathering, literature review, and site reconnaissance investigations, to on-site testing and the collection and management of samples for laboratory testing. Specific laboratory tests and techniques for the engineering analysis of soils and specific requirements for liquefaction analysis are not addressed in this document but are covered in companion technical basis documents.

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