MANUAL ON SUBSURFACE INVESTIGATIONS

1988

Published by the American Association
of State Highway and Transportation Officials, Inc.
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PREFACE

The American Association of State Highway Transportation Officials (AASHTO) through its Standing Committee on Highways and its Subcommittees on Materials, and Bridges and Structures have recognized the need for a comprehensive manual that documents and explains the increasingly complex and diverse techniques for conducting subsurface investigations for transportation facilities. Although the AASHTO Subcommittee on Bridges and Structures has previously developed the "Manual on Foundation Investigations," that manual is specifically focused on the acquisition and use of subsurface investigation information in the design of foundations for bridges and other structures. The subject matter of this publication, "Manual on Subsurface Investigations" is very broad and covers in great detail the many aspects of conducting subsurface investigations for transportation facilities. However, it should be noted that subsurface conditions are often highly varied and complex. Neither this Manual or any manual can cover every condition likely to be encountered when conducting a subsurface investigation. Consequently although the Manual is comprehensive and detailed, it is but a guide to be supplemented and continually improved by exercising engineering judgment and experience.

The "Manual on Subsurface Investigations" was initiated by AASHTO and accomplished through the National Cooperative Highway Research Program (NCHRP) which is funded through AASHTO's Member Departments. The preparation and editing of the Manual was administered by the Transportation Research Board following NCHRP procedures established by AASHTO.

1.0 INTRODUCTION

There is always a need for subsurface information and geotechnical data during the planning and development stages of construction projects. An understanding of the site geology is necessary for any project that has major components supported on or in the earth and underlying rock. The geotechnical features that will affect design and construction of the transportation facility must be investigated and evaluated.

1.1 PURPOSE

The purpose of this manual is to describe the various procedures for subsurface investigation applicable to the transportation field. An outline of a sequence of operations for conducting an investigation is presented. Data obtained at each operational step should be interpreted and the findings applied to optimize each successive work step. These geotechnical data should be considered as influential or even critical in all planning, design and construction stages of the project.

The manual discusses the increasing demand for detailed geotechnical information which has initiated extensive and costly subsurface explorations. The level of investigation appropriate to a particular project must be given careful consideration. Though the additional information will generally decrease possible unknowns and construction risks, a balance must be maintained between the costs of the exploration program and the level of information which will be produced.

Throughout the manual, mention is often made of the fact that no standard approach for subsurface investigation has been adopted. Widely diverse geologic environments, local equipment, personal preferences and time and budget constraints have all contributed to the development of different approaches. It has been found that subsurface exploration procedures cannot be reduced to a few guidelines that fit all conditions. The effects of specific geologic conditions on the type of proposed facility must be evaluated for each project.

The viewpoint taken in this manual is that the selection of individuals to direct the investigation, interpret the information and present the conclusions in a concise and usable form to those responsible for design and construction is of primary importance in any subsurface exploration program.

An area mentioned only briefly, but which will probably become more significant, is the importance of subsurface investigation and geotechnical participation in maintenance and rehabilitation projects. Subsurface exploration should not only be seen as important in the planning and designing of new projects, but in the maintenance and rehabilitation of existing transportation facilities as well.

1.2 DEVELOPMENT OF THE MANUAL

This manual was developed as a result of research initiated by AASHTO and performed under the NCHRP project 24-1, "Manual on Subsurface Investigations."

Previously, a discussion of subsurface investigation was included in the "Manual on Foundation Investigations," developed by the AASHTO Highway Subcommittee on Bridges and Structures. Acquisition and use of subsurface investigation data in the design of foundations for bridges and other structures were the focus of that report.

This is the first manual devoted exclusively to a discussion of subsurface explorations for all purposes and reflects the growing importance of this topic.

1.3 SUMMARY

A summary of the individual sections follows:

Section 2.0 Discusses data requirements; (1)

for most projects, (2) related to other geotechnical project concerns and (3) for major components of transportation-related projects.

Section 3.0	Lists a general sequence for conducting subsurface explorations and sources of existing data one may draw upon in the process of these investigations.
Section 4.0	Discusses field subsurface mapping and the field reconnaissance report.
Section 5.0	Covers geologic constraints and how subsurface investigations should identify potential geologic impacts early in the field reconnaissance and define their key aspects so the proper engineering response can be provided.
Section 6.0	Outlines the geophysical techniques that apply to geotechnical investigations.
Section 7.0	Outlines various planning and contractural procedures and describes drilling equipment, sampling, and logging methods.
Section 8.0	Discusses the relationship between transportation structures and subsurface water and presents some methods whereby hydrologic information can be acquired, analyzed, and put to use to prevent, alleviate, or correct undesirable conflicts between transportation structures and subsurface water.
Section 9.0	Discusses the purpose and classification of laboratory testing of soil and rock, requirements of the laboratory personnel, quality assurance, the primary tests and their approximate cost, sample handling, laboratory aspects of soild classification, shear strength determination, consolidation tests and permeability tests.
Section 10.0	Outlines the formal presentation and use of geotechnical information consisting of both factual and interpreted data.
Appendix A	Summarizes the various drilling sampling and instrumentation installations procedures required to obtain the necessary subsurface information.
Appendix B	Describes *in situ* borehole tests which determine various properties of soil or rock formations. The advantages, costs, limitations, and types of borehole testing are discussed.
Appendix C	A selected summary of field testing procedures required to determine various soil and rock properties and the forms used to record the data.
Appendix D	A summary of the test procedures discussed in Section 9.0.
Appendix E	Outlines soil and rock classification. Discusses the various classification systems, and in particular the Unified Soil Classification System (USCS). Suggests procedures and guidelines for preparing a complete description of a soil sample.
Appendix F	Discusses rock excavation methods.
Appendix G	Describes instrumentation of engineering structures as a way of detecting present or potential structural damage before the magnitude of deformation becomes uncorrectable.
Appendix H	Describes the effects of earthquakes on transportation systems and discusses subsurface investigation as an aid in earthquake resistant design.
Appendix I	Discusses the contribution of subsurface investigation to environmental impact analysis.

2.0 SUBSURFACE DATA REQUIREMENTS

2.1 GENERAL

Subsurface explorations for a transportation-related project typically have the objectives of providing: (1) general information on subsurface soil, rock and water conditions on the site or route, and (2) specific information on the subsurface conditions or soil or rock properties that are important to the various stages of project planning. An understanding of basic site geology is necessary throughout the planning process for any project that has major components supported on, or in the earth and underlying rock. In many cases, general geologic information, and in some cases specific information on subsurface conditions in the project area, will be available from technical references and reports, and previous subsurface explorations on and near the site or route.

Whatever the extent of available information on a particular project or site, there may become a need at some stage in the planning process for additional subsurface investigation. This investigation will usually have to be accomplished within budgetary and time constraints that will limit the level of effort that can be applied. It is therefore important that subsurface investigations be carefully planned, and coordinated between those who will obtain and those who will use the information.

The geotechnical data that are necessary for planning a particular type of project will vary from project to project. In the early stages, it may be sufficient to obtain only preliminary geotechnical information for alternative sites or routes to enable planners to evaluate project feasibility and identify major constraints and premium costs. However, these early data must be extensive enough and have sufficient accuracy to be appropriate for these objectives, so that correct planning decisions can be made before intensive design effort is initiated.

During project design, subsurface exploration and testing programs will be required to provide geotechnical data specific to the needs of the design team. The explorations and testing will serve the obvious needs of civil and structural design, but must also provide information pertinent to other related considerations, such as corrosion and environmental protection. The design-phase data must have sufficient accuracy, coverage and applicability to support design analyses and decisions. It should also permit reasonably accurate estimates of material quantities and construction costs.

In many cases relating to roadways, standard practice for the agency will apply unless unforeseen conditions arise that require special attention. For many states, this means logged borings at 100–175 m-spacing, with variations providing concentrated data at cut sections, borrow areas, or where geologically-related problems are expected. Structure foundations commonly have individually-planned explorations.

When a project is under construction there is not normally further subsurface investigation, except to resolve questions or problems that have arisen during construction. Design-phase explorations would have provided adequate subsurface information for design and, in most cases, for contractor bidding for construction. However, in some instances there may be a need for limited or local explorations to confirm design evaluations, particularly when there have been design changes subsequent to the main exploration program. There may also be a need for explorations and geotechnical data in connection with construction-phase instrumentation and monitoring.

As previously noted, the geotechnical data that are required for a project can be broadly categorized as general or specific. The first category encompasses identification and delineation of various soil and rock strata and ground water levels. The second category will provide both qualitative and quantitative information on the character and engineering properties of all or part of one or more of the various strata. Data for the first category will normally be derived from one or more of the various methods of subsurface explorations, while data for the second category will quite often require field or laboratory testing.

It is not possible to establish strict criteria for the data that should be obtained for a particular type of project. However, the typical or usual geotechnical considerations are: (1) data requirements common to most projects, (2) data requirements related to other geotechnical project concerns, and (3) usual data requirements for major components of transportation-related projects. It must be emphasized that the determination of data requirements is part of the planning process, and requires individual and continued attention on each project.

2.2 DATA REQUIREMENTS COMMON TO MOST PROJECTS

2.2.1 Definition of Stratum Boundaries

This requires identification and determination of vertical and horizontal locations of the various subsurface materials on a site or route. The data can range from visual observations or remote sensing output to detailed logs and physical samples of soil and rock from test borings or test pits. Relatively limited data are typically obtained for large areas during early project stages, while later stages will require increasingly detailed information, often for progressively smaller areas as project alternatives are narrowed down or final structure locations selected. Each addition of data should improve stratum boundary definition. The type of exploration that is selected for each stage should be appropriate for the data requirements.

In some cases field or laboratory testing may be necessary to define boundaries that are not otherwise evident. As an example, Standard Penetration Test AASHTO (T-206) blow counts may acceptably differentiate between dense or stiff and loose or soft strata, but natural water content determinations, shear strength testing or laboratory consolidation tests may be necessary to define limits of sensitive or overconsolidated clay.

2.2.2 Groundwater Level

This is not a static condition, being a function of season and precipitation. In addition, the water level in a test boring can be affected by the introduction of water for the drilling process. The ground-water level should be determined by readings over an extended period and by correlation with weather data. Water level data can range from observations in test borings or test pits to periodic observation well or piezometer readings, usually with corresponding improvement of data precision and reliability.

It should be noted that a low permeability stratum can cause either an overlying "perched" water table or an underlying artesian condition. In this situation there may be a need to seal a piezometer or observation well within each stratum of interest in order to yield a complete picture of groundwater behavior at the site.

2.2.3 Foundation Support

The planning and design of structures requires a determination of the strength of proposed foundation material. For light to moderate design loads and relatively competent bearing materials, such as rock, dense granular soil or stiff clay, data derived under the preceding two items may be sufficient to establish presumptive allowable bearing pressures for shallow foundations. Where there are clearly unsuitable near-surface soils, such as peat, the same data may also be sufficient for the design of deep foundations, such as piles. For most projects stratum definition and groundwater data will at least be adequate for early project planning. The peformance or problems of existing foundations in the area should certainly be considered, and there must also be a determination that underlying geologic features, such as solution cavities, or weak, collapsing or compressible soils do not control the bearing capacity.

In the case of shallow foundations, shear strength data for theoretical calculation of granular soil bearing capacity will usually be empirically derived from Standard Penetration Test blow-count determinations and laboratory gradation analyses. The shear strength of cohesive soils can be determined by field vane tests or laboratory shear tests on undisturbed samples. Where there are major foundation loads, or where further refinement of strength or bearing properties is necessary there can be more sophisticated field tests or laboratory triaxial testing of undisturbed samples of granular or cohesive soil.

In the case of deep foundations the need for additional data depends on the types of foundations being considered. For bearing piles there is a need to predict penetration into various strata. This is usually estimated on the basis of soil classification and density, or rock type and quality, as determined by test borings. Friction piles, unless designed on the basis of presumptive code values, require data or assumptions as to soil friction and adhesion characteristics, and caissons similarly require shear strength information. Such strength data for deep foundations can be developed by design-phase explorations and testing, but are normally substantiated by full-scale load tests of pile units and penetrometer tests of caisson bearing surfaces during construction.

2.2.4 Settlement or Heave Potential

This consideration can be pertinent whenever a new or increased structure or embankment loading is applied to a compressible soil. Major excavations can also result in heave of the foundation bottom and adjacent areas. Certain soils, such as soft clays, loose sands or organic deposits, are known to be compressible without demonstration by laboratory testing, and early planning can be based on this general knowledge. Knowledge of existing settlement problems in the project area can also be used for planning. However, actual data are necessary to predict rates and amounts of settlement. Other soils require data and analysis to determine settlement or heave potential under particular loading conditions. In either case, stratum definition and groundwater information are necessary parts of the data.

Settlement due to compression of granular soils can occur as the load is applied. Data for estimating settlement can be obtained from empirical Standard Penetration Test relationships, from field plate bearing tests and, in the case of elastic compression, from the results of laboratory triaxial testing of undisturbed samples.

Estimates of the rate and amount of long-term settlement due to volume-change compression of cohesive soils, such as clays or organic soil deposits, are commonly based on data derived from laboratory consolidation testing of undisturbed samples. Elastic compression of cohesive soils can be calculated on the basis of modulus data from laboratory triaxial testing on undisturbed samples. In some areas the consolidation or compression properties of a major soil stratum are sufficiently well known for preliminary or general evaluations. The presence and identification of the stratum may be confirmed by classification testing of disturbed samples from borings or test pits.

At some locations there can also be potential for settlement due to subsidence caused by conditions in underlying strata, such as solution cavities, mines, groundwater lowering or soil erosion.

2.2.5 Slope or Bottom Stability

This consideration is applicable to temporary or permanent earth or rock slopes that exist or are constructed as part of a project. It can also apply to the bottoms of major excavations. Instability can range from ravelling of a granular surface to a deep base failure of an entire embankment or the heave of an excavation bottom. Early stages of planning can utilize general geologic and groundwater information, supplemented by the physical evidence of existing stable or unstable slopes. However, design phase evaluations of major slopes or excavations must be based on defined strata and groundwater information, and on shear strength properties of soil and rock.

Soil data requirements include groundwater seepage patterns, the friction angle of granular soils, and the shear strength of cohesive soils. Laboratory triaxial testing of undisturbed samples of cohesive soils may be necessary to determine either drained or undrained properties, depending on the type of analysis required. It may also be necessary to monitor observation wells over a period of time to determine changes in groundwater levels.

Rock data requirements consist in part of determination of the strength of intact specimens from cores, however, the properties of the rock mass are of primary importance. Weathering, jointing, and other discontinuities will control the stability of a steep rock face. Some information can be obtained from ordinary core borings, but where jointing is critical or unfavorable, and rock falls cannot be tolerated, there must be supplemental data. These can be obtained from sophisticated coring techniques, geologic mapping of available rock exposures, or mapping of rock in test holes or adits. Used in combination, these techniques can provide a reasonable representation of the system of joints and other discontinuities, permitting valid stability analyses.

2.2.6 Lateral Earth Pressure and Excavation Support

Most projects will include some form of wall that is subject to earth pressures, either a retaining or foundation wall, or temporary excavation support. Data on soil strata and properties, groundwater levels, and the structural characteristics of the wall, will be necessary during the design phase for permanent walls, and during the design or construction phase for temporary excavation support, depending on the provisions of the construction contract. In either case the soil data would normally be obtained during design-phase subsurface explorations.

Gradation test results and Standard Penetration Test data from drive sample test borings are usually sufficient to derive reasonable properties for granular soils, but field vane tests or laboratory shear testing may be necessary to determine drained or undrained properties of cohesive soils. It is important to also consider the effects of fill, backfill, and construction procedures on the properties that are selected for analysis. In the case of temporary excavation support it may be necessary to analyze several stages of the excavation, with appropriate soil properties for each.

2.2.7 Dewatering

Whenever a project involves excavation there is a potential need for dewatering. It is particularly important that groundwater levels and possible ranges of groundwater levels be carefully determined to minimize the occurrence of unexpected dewatering problems during construction (Figure 2-1). When there can be water within an excavation depth, or there can be artesian water pressures below an excavation, it is necessary to have data on stratum boundaries and soil and/or rock permeability for design and construction-phase evaluations.

For routine work, adequate permeability data for estimating inflow and planning dewatering may often be developed from stratum definition and soil or rock classification. However, where there can be major water inflow or excavation bottom instability, or there is a need to maintain groundwater level outside the excavation, it will be necessary to obtain information on the vertical and horizontal permeability of various strata. If re-charging is to be attempted the probability of clogging should be evaluated, necessitating information on water quality.

The permeability of relatively uniform isotropic

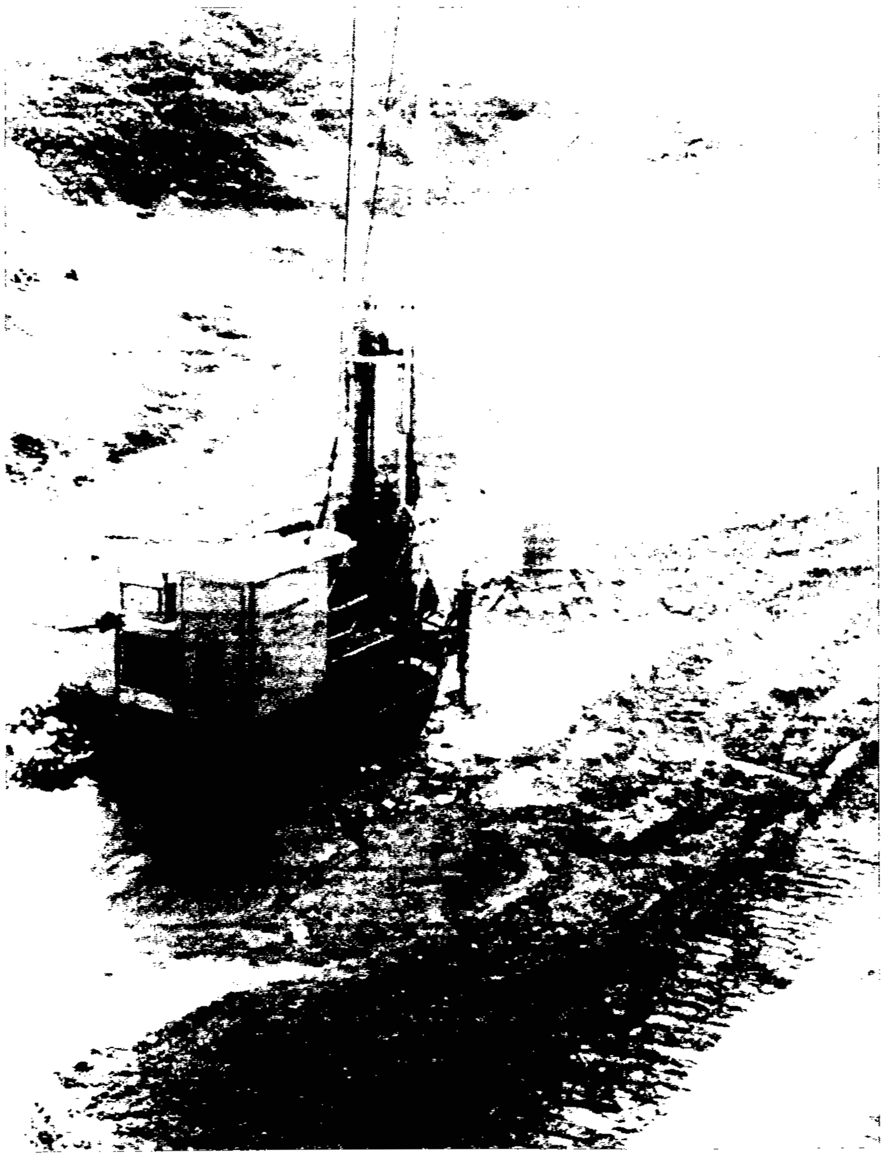

Figure 2-1. Without adequate dewatering, site preparation and grading becomes waterlogged and schedules slip unnecessarily. (A.W. Hatheway)

granular soil can usually be satisfactorily estimated from gradation and Standard Penetration Test information, but most broadly-graded, cohesive, or anisotropic soils require field or laboratory permeability testing. Simple field permeability tests can usually be acceptably performed below cased boreholes, or in observation wells or piezometers, particularly if the test is performed below the water table. Laboratory permeability tests are preferably performed on undisturbed samples of soil. However, in the case of fine to medium granular soils, reconstituted samples are generally used.

Representative rock mass permeability data are more difficult to obtain because of the effects of joint systems and other discontinuities. Effective or equivalent permeability data can be obtained from pressure or pumping tests performed in rock in boreholes with the aid of packers for test isolation. Multiple tests should be performed because the presence or absence of discontinuities within the limits of an individual test will dramatically affect the test results.

Large-scale pumping tests from drilled wells, using patterns of piezometers or observation wells to define stabilized drawdown levels, can provide good specific information on dewatering requirements for a particular site or structure. These also permit the evaluation of stratum permeability, or transmissibility. It should be noted that large scale pumping tests have limited value beyond the actual test location when pervious strata are irregular or discontinuous.

2.2.8 Use of Excavated Material

Whenever significant volumes of material are excavated for a project the use or disposal of the material becomes a cost consideration. Large volumes of material can influence design, either because the material can be effectively used in the particular project or in other projects, or because disposal cost outweighs the benefits of excavation. Thus, the determination of quantities and properties of excavated material becomes important.

Early planning can usually be based on stratum and groundwater definition, but positive commitment to use of material requires investigation commensurate with the quality requirements for the proposed use. Simple disposal of non-natural materials can require investigation and testing to determine if hazardous materials are present, while the use of non-natural materials in embankments can be limited by corrosive properties or potential decomposition. Existing fills require particularly careful investigation before commitment to project use because of the potential for random inclusions of unsuitable materials.

Natural soils can usually be used for ordinary fill as long as there are not significant organic materials, such as topsoil or peat, and the soil can be satisfactorily placed and compacted. Laboratory testing of jar or bag samples can determine organic content and natural water content of soil, the latter for comparison with the laboratory determination of optimum water content for compaction. Gradation and Atterberg limit determinations can provide additional data with respect to frost susceptibility and expansion characteristics.

Excavated rock and clean granular soils can sometimes be economically utilized for riprap, aggregate, processed material, select borrow, or other specification items. The highest grade use would normally be the most desirable. If the use is to be a contractor option, only routine testing may be necessary during the design phase, with more extensive sampling and testing to be carried out at the time of proposed use. If the use is to be specified, a comprehensive design-phase sampling and testing program is necessary to establish the availability of adequate quality and quantity of material. Explorations should provide enough information to evaluate the cost of selectively excavating the material. Testing must address all of the specification requirements for the proposed material use, and should also consider other possible lower grade uses.

2.3 OTHER GEOTECHNICAL DATA REQUIREMENTS

2.3.1 Geologic Constraints

While site geology is always a geotechnical consideration for project planning, there are situations where geologic constraints will be a primary factor controlling planning and design. Geologic constraints could include faults, major glacial features such as buried valleys, landslides, volcanic formations, leached soils, or groundwater acquifers (Figure 2-2).

During early project planning, data for the evaluation of possible geologic constraints will normally be obtained from available references, aerial photograph interpretation, local geologic knowledge and/or site reconnaissance. Design-phase subsurface explorations, possibly including extensive test trenches, test pits, or adits for visual examination of geologic features, are likely to be necessary to confirm preliminary evaluations. These confirming explorations will permit assessment of the impact of each geologic constraint upon the project.

Figure 2-2. Geologic constraints can seriously impact transportation projects, from construction through operation and maintenance. This secondary road has suffered total damage from slope movements over a period of years and has been abandoned. (A.W. Hatheway)

2.3.2 Seismic Evaluations

When a proposed project is located in an area that has potential for earthquakes there must be an evaluation of seismic risk. Depending on the level of risk, there may or may not be a need to develop seismic design parameters.

The evaluation of seismic risk can range from simple acceptance of local codes to intensive geologic studies of the site or route and probabilistic evaluation of data on past seismic events, possibly with the aid of computer programs. A comprehensive risk evaluation will consider earthquake magnitude, return period, and epicentral distance to arrive at a design value of bedrock or ground acceleration, and possibly duration, for which a project must be designed.

Dynamic analyses for a project are generally concerned with foundation or embankment stability, and with earthquake forces to which a structure may be subjected. Soil data for these analyses will include cyclic shear strength and/or shear modulus values. Basic stratum definition and groundwater information are necessary. In some cases it will be sufficient to establish soil classification and density from the results of ordinary drive sample test borings and routine classification tests, and then evaluate earthquake performance on the basis of historical comparisons and published data for typical soils.

Comprehensive analyses for major projects may be based on shear strength and modulus properties determined by sophisticated laboratory dynamic testing of undisturbed samples of cohesionless or cohesive soils. Shear modulus properties may also be determined by field seismic testing. The approach to be taken should be selected on the basis of project requirements. It should be noted that dynamic laboratory testing is relatively costly, and may not accurately model a particular design condition.

2.3.3 Corrosion or Decay Potential

If a project involves in-ground steel, concrete or wood structural components, or buried utilities, there has to be consideration of the potential for corrosion or decay. The corrosion problem can be particularly acute if large amounts of electric current are used, conducted or generated in the vicinity. It is generally considered that steel requires protection from cinders and near surface organic soils, and wood from dampness without submergence. Various salts and alkaline or acid groundwater will attack concrete or metals.

Geotechnical investigation for corrosion evaluation will consist primarily of determination of appropriate properties for the strata and groundwater that have been defined by subsurface explorations. The tests for corrosion evaluation will usually include resistivity tests on disturbed soil samples in the laboratory or *in situ* in the field, along with pH determinations and chemical analyses of both soil and groundwater in the laboratory. The decay potential of untreated wood in the ground is primarily a function of groundwater conditions.

2.3.4 Frost Penetration and Freezing

Projects in areas that will have sub-freezing temperatures must consider frost, with the main concern being possible heave of foundations or pavements due to the formation of ice lenses. Frozen ground will also tend to lift embedded structures because of adhesion. Frozen slope surfaces will interfere with drainage, leading to spring sloughing, and the freezing of water in rock joint systems will reduce rock cut stability. Arctic areas will also have much broader foundation concerns associated with permafrost and extreme winter conditions.

The three necessary conditions for the occurrence of frost heaving are sub-freezing temperatures, available water, and frost-susceptible soil. Thus the necessary data will include soil strata and groundwater definition. In addition, the soil type and water content will determine the rate or depth of frost penetration, and soil gradation is the commonly used measure of frost susceptibility. Care must be taken with respect to gradation where there is natural layering that is not reflected in laboratory test results.

2.3.5 Soil Expansion or Swell

Certain soils, most commonly in relatively warm dry climates, are characterized by problems with volume change due to changes in water content. The avoidance of differential foundation, floor slab heave, and settlement depends upon the avoidance of either expansive soils in project areas or detrimental changes in soil water content. Soil modification with lime is sometimes proposed to mitigate expansion problems.

Project design in areas of potential expansive soil problems should first consider historical information from other projects in the area. Specific data acquisition for the project will consist mainly of stratum definition, groundwater information, and the determination of index properties by classification testing of disturbed samples. In some situations it may be desirable to make laboratory determinations of the swelling pressure of undisturbed or compacted soil samples.

2.3.6 Environmental Concerns

This covers a variety of considerations, primarily related to the effect of the construction and operation of the proposed project on its surroundings. There is a distinct geotechnical aspect to environmental effects because many features of project design and construction techniques are directly related to subsurface conditions. Poor soils can necessitate deep foundations, with resulting dewatering and groundwater drawdown or pile driving and accompanying noise. Embankment construction can obstruct or contaminate surface and subsurface water flow, and their construction may involve dust and noise. Grading will expose soils to erosion. Many other construction operations that are the necessary outcome of planning and design decisions, or the logical result of economic considerations, will affect the environment and should be evaluated.

Geotechnical data for environmental considerations can include almost all of the data that are necessary for project planning and design. It can also involve field or laboratory testing that might not otherwise be necessary for design, such as permeability determinations, rock quality evaluation with respect to blasting characteristics, and soil gradation and plasticity for the prediction of behavior during earthwork operations. Environmental consideration of effects outside of the site or route will also require some knowledge of subsurface conditions outside the project limits; in most cases this knowledge will be extrapolated or inferred from available information, rather than determined directly by off-site subsurface explorations.

2.3.7 Erosion Protection

This can be both a design and a construction consideration, with the latter relating primarily to environmental concerns. Erosion is commonly related to surface water flow, but can also be a condition related to subsurface seepage and drainage.

Data for the design of erosion protection will include both surface and subsurface water levels and velocities or gradients (Figure 2-3). Possible extreme levels and potential changes due to proposed construction must be considered. Where flow can be against or in natural soils, stratum definition is necessary.

Soil susceptibility to surface erosion is primarily a function of the water flow and the gradation and plasticity of the soil. Density and cementation will also affect the susceptibility. Most of the soil information will be provided by test boring data and laboratory classification testing of disturbed samples, but cementation may only be evident in undisturbed exposures. Cementation, if given consideration, must also be evaluated as to possible deterioration when exposed to water flow. Where erosion protection is determined to be necessary it must be designed to economically resist the water flow without loss of, or damage to the protected soil surface.

Erosion by subsurface flow can be a major threat to a project if it extends by piping as an open conduit under a water retaining structure or a foundation. The manner of occurrence is similar to that for surface flow, but there must also be an open path for the movement and loss of the eroded soil. Protection against subsurface erosion is commonly afforded by granular filter materials or filter fabrics which have particles or perforations sized to satisfactorily pass the water flow without permitting movement or loss of the soil particles. The basic data requirement for the design of filter protection is the gradation or range of gradations of the soils that are to be protected.

2.3.8 Permanent Groundwater Control

Design maximum and minimum water levels for below-grade portions of projects are commonly developed from groundwater information. Where water levels would otherwise extend up into pavement, railroad track base or sub-base layers, underdrain systems are designed to hold groundwater down at acceptable levels. In some cases the normal groundwater level will be similarly artificially lowered to avoid a need for waterproofing below-grade structures. Occasionally, recharging may be necessary to preserve existing groundwater levels outside of a project area.

The data required for the design of permanent groundwater control is substantially the same as is required for planning dewatering. From the point of view of system longevity there will be added concern for design of the collection system to meet filter criteria and minimize the potential for clogging or corrosion. More accurate permeability determinations and flow calculations may be warranted when piping and pumping costs will be a significant part of overall project cost.

2.3.9 Soil or Rock Modification

Some projects will involve one form or another of soil or rock modification for engineering or economic reasons. Until a particular type of modification is given detailed consideration for design, the subsurface data that are used for planning will usually consist of the basic stratum definition and groundwater information, along with such other data as may be provided by project subsurface explorations.

A particular proposed modification, such as grouting, sand or stone drains, or lime stabilization, will usually require data on specific properties, or more detailed information on the soil or rock that is to be modified. A determination of groundwater is likely to also be necessary.

Where grouting is planned, the type of grouting that is utilized will depend on the intent of the grouting and the character of the spaces to be filled. For soil this necessitates determination of gradation, and some evaluation of *in situ* density, void ratio or permeability. Soil gradation can be obtained by laboratory testing of disturbed boring or test pit samples, but actual density, void ratio or permeability determination will require field or laboratory testing of undisturbed material. Information on joint spacing, continuity and condition is similarly necessary for rock. The evaluation of rock for grouting is often attempted on the basis of records and recovered rock cores from

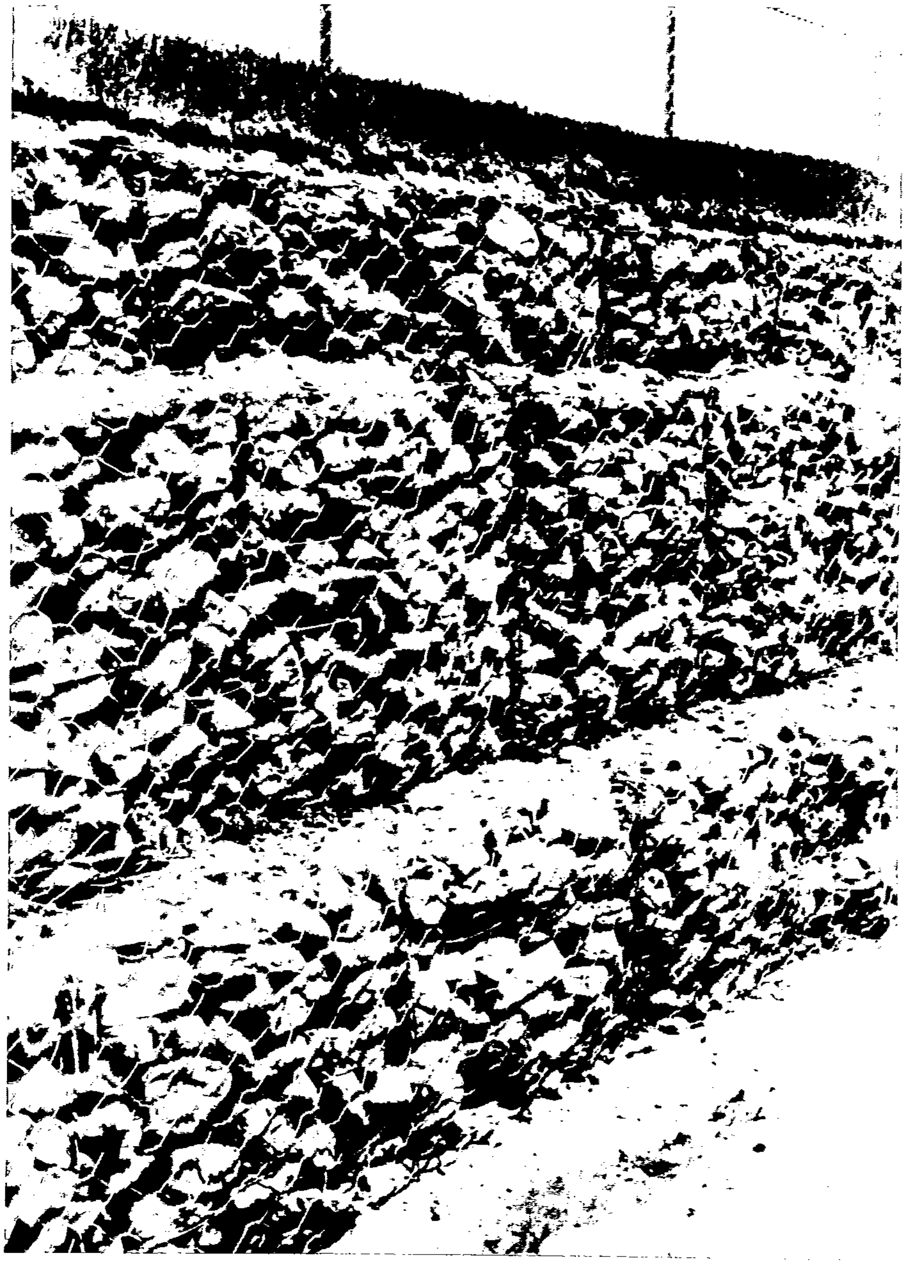

Figure 2-3. Wire-basket gabions offer on-site fabrication of heavy-duty erosion protection. (A.W. Hatheway)

test borings. Water pressure or pumping tests can provide rock permeability data, and useful information can sometimes be obtained from examination of exposed rock surfaces in cuts, adits, or shafts.

Vertical or horizontal drains have the objective of relieving pore pressure within a soil or rock mass, both when that water is a product of natural processes or results from soil consolidation. Soil permeability data from field or laboratory testing are sometimes appropriate for evaluating required drain capacities, out in other situations the capacity must be matched to an existing subsurface flow condition. Where drains are to be installed in cohesive soil to accelerate consolidation, as would be the case for a surcharging operation, the consolidation data that are necessary to set drain spacing can be obtained from laboratory testing of undisturbed samples. Granular drain fill material should be sized to carry the flow while meeting filter criteria with respect to the surrounding soil. Thus, gradation information is necessary.

Other modification techniques can require more specialized data. As an example, the effectiveness of lime stabilization in improving the performance of clay is partly a function of the reactivity of the clay.

Reactivity is related to the chemical properties of the clay and can be measured as the increase in compressive strength of compacted specimens prepared with the addition of lime.

2.3.10 Material Sources

There is not usually a preconstruction investigation of soil or rock material sources outside of the normal excavation limits for a project. Furthermore, the evaluation of off-project material sources is customarily left to the Contractor, subject to testing and approval by the Engineer during construction.

However, in some cases it will be desirable to carry out a design-phase investigation to locate sources of borrow materials for a project, at least to the extent of confirming that suitable materials are available. It may be sufficient to map surficial geology by aerial photography interpretation or other remote sensing, and/or ground reconnaissance, supplemented by review of available geologic references and plans. Where this approach does not provide enough certainty as to either quality or quantity there can be subsurface exploration by auger holes, test borings, or test pits to confirm stratum boundaries and groundwater levels, and obtain disturbed samples.

Borrow material will typically be ordinary fill or bankrun sand and gravel. There is not a need for in-depth determination of properties unless the material is to be processed for special use such as aggregate, or is itself the result of previous processing by man. Routine laboratory gradation and compaction testing of representative samples from test pits is usually adequate.

2.3.11 Underpinning

Excavation for structures or roadbeds in urban areas can reduce or endanger the support of existing structures, necessitating underpinning for temporary or permanent transfer of existing loads to lower level supporting strata. This is another aspect of project construction that is often considered to be a Contractor responsibility, subject to contract stipulations as to structure monitoring and tolerable movement. Alternatively, where there is an obvious need for complex or major underpinning, the necessary structure support may be included in the project design and detailed in the contract documents.

Whatever approach is chosen, there is a need for subsurface information for analysis of the load transfer and design of the underpinning support. In addition to the basic stratum definition and groundwater information, which may be incomplete because of access limitations, there may be a need for test pits to provide information on existing foundations for which records are lacking. Analyses of support capacity for underpinning require much the same data as those for new construction. Since movements during and after load transfer can be differential with respect to other parts of the underpinned structure, it is also important that short- and long-term settlement and heave be considered, and that appropriate data be obtained.

2.3.12 Post-construction Maintenance

Design decisions should consider maintenance cost, and a number of geotechnical factors can influence long-term maintenance requirements. For the most part, these factors are given consideration under the various design items, but local conditions may be neglected or the long-term effects may be slighted to serve short-term economy.

Differential settlement, frost heave, or expansive soils can greatly accelerate the need for pavement repair or reconstruction, or cause serious damage to buildings or buried utilities. Groundwater seepage or springs can cause slope problems or wet basements, certain soils are particularly susceptible to erosion by surface flow, and some soil or rock slopes have a high probability of gradual sloughing or ravelling.

Subsurface exploration programs should be carefully planned to locate potential maintenance problems. There is no substitute for on-site or along-route reconnaissance by experienced geologists or engineers to detect problem areas or conditions that have only limited extent. Initial mapping of surficial geology can delineate areas of soil types or groundwater conditions that should be field checked for evidence of problem conditions. Field checking for potential problems should extend through construction; experienced personnel should get out and look, and should involve both design and maintenance personnel in the resolution of potential problems.

2.4 USUAL DATA REQUIREMENTS FOR TRANSPORTATION-RELATED PROJECTS

2.4.1 Bridges and Viaducts

Most major transportation projects will include bridge or viaduct-type structures, and the design and construction of these structures will usually involve most of what has been categorized as "common" data requirements (Figure 2-4). The primary concerns will be foundation support and potential settlement, as these factors will frequently control bridge type and

Figure 2-4. Bridge-pier foundation construction of drilled shafts in weak rock. (A.W. Hatheway)

span lengths. Competent soil or rock will permit spread footing support of relatively economical short spans, using rolled steel or prestressed concrete beams, and conditions of minimal settlement will permit the use of rigid frames or continuous spans. Deep foundations, such as piles, are ordinarily likely to be more costly than footings with the result that poor foundation conditions will tend to favor longer spans. Conventional arch bridges require both vertical and horizontal support capability at the abutments.

Lateral earth pressure on abutments and temporary excavation support, along with dewatering, are likely to also be major concerns for bridges and viaducts. However, slope stability and use of excavated material may have little or no impact on design and construction. The various items in the "other data requirements" category may or may not apply to a particular bridge or viaduct project. Probably the most frequent concerns will be environmental and erosion protection, the latter becoming important when the particular project involves a water crossing and is subject to scour or wave action. Corrosion or decay can be important for the design of pile foundations.

2.4.2 Retaining Structures

These are also included in most transportation-related projects; they can range from simple bridge wingwalls to long walls retaining embankments in urban areas. Walls also involve most of the more common geotechnical data requirements, with the need for a retaining wall, or the type of retaining wall, being very much dependent on foundation support conditions and the potential for settlement. Lateral earth pressures will normally control the design of whatever type of wall is selected, and resistance to sliding must be considered. Competent foundation soil or rock, or a suitable bearing stratum at moderate depth, will favor conventional retaining walls, while poor or unusual foundation conditions can make unconventional walls more appropriate.

2.4.2.1 Conventional Retaining Walls. The design of conventional reinforced concrete walls requires very much the same geotechnical data input as bridges and viaducts, with lateral earth assuming more importance and excavation support possibly becoming more complex. Design earth pressures for cantilever or gravity walls will typically be the "active" case unless dynamic forces due to machine-induced vibrations or earthquakes cause a build-up in pressures. Slope stability during construction will be important if the wall is part of a cut into an existing slope. Permanent groundwater control may be necessary to minimize lateral pressures acting on the wall.

2.4.2.2 Crib and Reinforced-earth Walls. Some alternative types of retaining walls offer greater tolerance of settlement, along with resistance to lateral earth pressures that is derived from the earth mass behind the face of the wall. In this category are crib walls, gabions, and reinforced-earth walls. These walls are most commonly used in connection with embankments, or possibly side-hill cuts, rather than for the support of soil alongside of excavations.

Crib walls and gabions are formed by the containment of soil or gravel and cobble-sized rock in relatively flexible structural units. The crib walls use steel, concrete, or timber members interconnected to form a series of box-like cells, while gabions utilize filled and stacked wire mesh baskets. Neither would be expected to have the appearance or durability of a well-built reinforced concrete wall, but both can tolerate substantial settlement without distress. They function as gravity walls, and do require foundation soil or rock to provide adequate overall stability for the wall and retained earth.

A reinforced-earth wall incorporates a wide zone of soil backfill behind the wall into the mass of the wall by means of tension steel strips that are laid out onto backfill layers as the fill is placed. The strips tie back a relatively flexible wall face. Design is semi-empirical and involves consideration of the friction capacity and corrosion potential of the steel strips, along with the basic concern for the stability of the reinforced mass on its foundation.

Geotechnical data for both crib walls and reinforced-earth walls should therefore be similar to that for conventional walls, with added consideration of properties of proposed fill materials.

2.4.2.3 Diaphragm Walls. Diaphragm walls are usually used to support the earth alongside of excavations, and can provide both temporary excavation support and the finished wall in one operation. The term diaphragm is most commonly applied to a concrete wall cast in-place in a slurry-filled trench prior to the general excavation. It can also include other installation procedures that provide a wall consisting of laterally supported panels or units, typically with all or part of the wall construction accomplished prior to the general excavation. Bracing in the form of tiebacks or struts, or permanent decks or floors, is installed as the excavation between the walls progresses.

The evaluation and design of diaphragm walls requires consideration of the impact of the *in situ* material on the excavation process, i.e. will obstructions significantly hamper excavation or result in unacceptable wall quality? The *in situ* material must also provide vertical support for the weight of the wall and stability of the excavation, at least during the construction process, and there must be a practicable way to provide lateral support of the wall by tieback anchors or struts. When bentonite slurry is utilized in the excavation process there must be consideration of groundwater quality.

Geotechnical data for diaphragm walls should therefore also be similar to that for conventional retaining walls, with particular emphasis on the character of the material in which the walls will be constructed. When appropriate, there should be further data from the testing of groundwater samples and/or data on potential anchor zone materials and groundwater conditions for tiebacks.

2.4.3 Cuts and Embankments

Roads, railroads, and airport runways will usually require major cuts and fills to meet design grade limitations. To the extent possible, grades and alignments will be planned to balance cut and fill quantities on a given project, thereby minimizing borrow or waste. However, the effort at balancing quantities will be subject to a variety of limitations, ranging from embankment stability or settlement to non-geotechnical considerations, such as meeting existing alignments and grades or reducing environmental impact.

Most of the previously categorized "common" data requirements can apply to cuts and embankments, although foundation support, lateral earth pressure, excavation support, and dewatering may have limited applicability. The primary concerns will be embankment and slope stability and settlement potential, which will control cut and fill slopes, embankment heights, and possibly rate of construction. Emplacement of embankment fill should be continuously monitored by geotechnical personnel so as to achieve proper strength and settlement characteristics and to avoid later deformational damage. Weak or highly compressible soils may have to be removed, displaced, or bypassed, and any major limitations should be known during early project planning, so that premium costs can be evaluated before alignments are finalized. Detailed information along the selected alignment should then be obtained by means of design-phase subsurface explorations.

Data pertinent to the use of excavated material, and other probable considerations, such as expansive soil or frost penetration, environmental concerns, erosion protection or underdrains, or material sources, should also be obtained during the design-phase investigation.

2.4.4. Roadway and Airfield Pavements

Pavement projects require data for the structural design of pavement sections. Where the pavement will be on an embankment the pavement subgrade can be controlled as part of the embankment construction, but in cuts the *in situ* soil or rock conditions and properties must be determined.

Local consolidation settlement under short duration pavement loadings would not be expected to be a consideration, except possibly in areas of subgrade disturbance or trench backfilling during construction. Subgrade strength is a basic consideration, generally requiring the data described under foundation support; the California Bearing Ratio (CBR) test is also a direct measure of subgrade support capacity widely used in empirical pavement design procedures.

Weak subgrade soils can necessitate a thickened pavement section, removal and replacement of poor quality soil, or some form of soil stabilization or improvement. There are also other potential considerations, which may or may not apply to a particular project, that can require appropriate data for input to pavement section design. These include frost penetration, soil expansion, groundwater control, maintenance, and the availability of pavement materials.

2.4.5 Railroad and Transit Tracks

Data required relative to track support are similar to that required for pavement design. Dynamic effects of track loads are typically more extreme than the effects of wheel loads on pavements, and there is a greater concern for good drainage of exposed ballast. Subgrade strength and the other considerations enumerated here must also be considered in the design of a track system.

In addition, the potential for movement of relatively fine-grained subgrade soils into the voids of crushed stone ballast is a major concern. Both the use of vibration-type compaction and filter fabric on the subgrade is frequently specified for railbed construction. Filter protection data requirements are discussed under erosion protection.

2.4.6 Tunnels and Underground Structures

Design for underground construction is basically a geotechnical engineering effort, with project configuration being subject to the limitations imposed by soil and rock conditions and properties. There must be sufficient subsurface data input during early project planning to reasonably assess the feasibility and cost of various alternatives. Any geologic constraints must be known at an early stage.

Design-phase data for tunnels and underground construction will be primarily concerned with stability of materials being excavated, with particular emphasis on soil or rock surfaces exposed during construction, and on gradual or long-term adjustments that may affect unsupported walls or roofs after construction. Data must also relate to earth or rock pressure and temporary support, and sophisticated *in situ* pressure testing may be warranted. Dewatering will usually be a concern, and soil or rock modification, underpinning, maintenance, and use of excavated material can also be important considerations.

The engineering of tunnels and underground structures will extend into the construction phase, as excavation and exposure permit confirmation or require revision of the properties that have been assumed for design. Instrumentation and monitoring during construction should be carefully considered and planned to aid in confirming design assumptions and provide data input for safe and economical design of future similar projects.

2.4.7 Poles, Masts and Towers

The data that are required for the design and construction of poles and towers will be primarily concerned with support capacity. There will not usually be major excavations or dewatering, but there may be consideration of corrosion or decay, erosion protection or soil or rock modification.

Since poles and towers may have high wind loads, the evaluation of soil or rock support capacity will often have to consider lateral resistance for poles and masts, uplift capacity for structural towers, and guy wire anchorages. These considerations will generally be a function of the properties that are determined for lateral earth pressure calculations, but theoretical analyses of side-bearing resistance or friction capacity may not accurately model the field condition. In some cases large scale *in situ* horizontal bearing or vertical or inclined pull-out tests may be warranted.

2.4.8 Culverts and Pipes

Large box culverts will generally require data comparable to that for bridges and viaducts, with particular concern for lateral earth pressure and excavation support and dewatering. Large span metal arches and pipe arches are dependent on foundation support at the haunches, and good quality, highly competent backfill on the sides of the pipes. Smaller culverts and pipes are less dependent on foundation support, but in the case of high fills or deep trenches, settlement and/or excavation bottom stability assume more importance. Corrosion is a concern for buried underwater metal.

2.5 MAINTENANCE MANAGEMENT

Transportion system maintenance budgets are generally developed on the basis of regional experience dealing with normal traffic, the effects of vehicle accidents on a statistical basis, and incidental remedial treatments of a more or less unpredictable nature. Maintenance in the last category usually involves the repair of facilities damaged in some way by the elements. Some of the repairs can be assigned to causes of a geological or geotechnical nature and the more obvious types of geologically-related damage are easily identified by design engineers. Maintenance personnel can readily identify various forms of slope movements that disrupt traffic flow or create displacements in the roadway. Geotechnically-related problems can usually be associated with deficiencies in design or construction; they are usually hard to detect, subtle and may be difficult to assign to a specific cause.

Structural and highway design personnel can do much to assist in the detection of causes for geologically and geotechnically-related damage by developing programs which catalog examples of related damage factors. District-wide briefings to both design and maintenance personnel should be held, and identification of such natural causes should be stressed. Geotechnical personnel should be able to list from experience, many similar factors underlying recurring maintenance expenditures.

Data appropriate to the geotechnical aspects of maintenance management will include physical evidence of any problems or distress that can have geotechnical origin. Groundwater seepage, slope or structure movement, and pavement distress are obvious concerns. Recorded evidence should include identifiable soil and rock exposures, weather conditions, and the geometry of the problem area.

Some natural damage will require rapid assessment, remedial design, and award of a competitive-bid contract for repair outside of the Agency force account. In times of natural disaster, repair funding often requires special legislative or Federal appropriations. Such a requirement is commonly found in damage to State and Federal highways lying on Federal lands, such as those administered by the U.S. Forest Service and the U.S. Bureau of Land Management.

One of the most important facets of geotechnical participation in maintenance management is development of a standard method of recording maintenance stemming from natural causes. From such experience, methods of design-avoidance should become apparent and occurrence frequencies for various types of natural damage should be reduced over a period of years.

Once the geological sources of recurring maintenance problems have been detected, it will usually become apparent that the causes are predictable on the standard method of regional physiography and will be more pronounced in some Districts of larger state or provincial Agencies and may also overlap between adjacent districts, states or provinces. A source of these regionally important geological factors are the yearly proceedings of the Highway Geology Symposium and the Idaho Symposium on Engineering Geology and Soils Engineering.

2.6 REHABILITATION PROJECTS

With the completion of the Federal Interstate Highway Program, attention has been turned to the problem of rehabilitation of the older segments of these routes, as well as other primary and secondary roads. The rehabilitation program is generally involved with resurfacing, rehabilitation, and reconstruction. A separate FHWA program also addresses the rehabilitation of bridges. Subsurface investigation techniques are an important part of planning for rehabilitation projects.

The underlying objectives of rehabilitation expenditures are to restore the functional use of transportation routes, with the application of optimal funding, to make full use of existing structural components of each route. Geotechnical personnel are capable of providing significant input into the planning and management of rehabilitation projects. Since the goal of optimization of expenditures requires maximum use of existing structures, geotechnical personnel should be called upon to inspect and record evidence of failure or distress in rehabilitation candidates projects. Most of the damage requiring rehabilitation is the result of the following:

- Overstressing by vehicular traffic
- Aging beyond the life of the component
- Improper construction techniques
- Improper construction materials

- Obsolescent design provisions
- Failure of natural materials or subgrade units below or adjacent to the roadway or other structure under consideration
- Upgrading of route dimension, layout, and traffic requirements since construction

With exception of the final reason stated above, the reasons underlying need for rehabilitation work can be detected and recorded in the field by trained geotechnical observers. The evidence that will appear is that of surface and pavement pitting; pavement cracking; pavement edge sloughing and erosion; erosion of structural supports for bridges and viaducts; broad roadway surface depressions (settlement-induced), lateral movement of fills, supporting embankments and cut slopes; deterioration of concrete due to disintegration of mineral aggregate, and frequent erosion and runoff debris falling or flowing into the roadway.

The evidence of distress and damage can be detected and recorded by engineering geologist and geotechnical engineers on a base plan reproduced from the design/contract or as-constructed plans for the project. For projects which are not presently supported by record plans, geotechnical personnel can compile simple pace-and-compass plots of key areas of damage, supplemented by site-related photographs. Maps at 1:200 scale are ideal for recording most evidence of wear and distress of transportation systems.

Geotechnical participation in rehabilitation planning can be accomplished in an orderly manner, providing support from the beginning of planning. Some of the usual steps in the procedure are as follow:

1. Locate and review existing records of the project such as the design and as-built plans.
2. Make a rapid reconnaissance of the site or segment, combining the expertise of highway planner, bridge and structure engineer and engineering geologist or geotechnical engineer; determine the objectives of observations to be made in more detail.
3. Make a geotechnical assessment map at 1:200 or other specified scale, carefully noting the physical nature and orientation/location of all types of distress; take hand soil and rock samples where necessary; take photographs and relate them to the assessment map.
4. Plan for supplementary subsurface explorations to verify or determine the nature and extent of conditions in the roadway subgrade, supporting embankments, and adjacent cuts, that may be related to the observed distress.
5. Conduct the borehole sampling, pavement coring, geophysical surveys and laboratory testing.
6. Develop an assessment of the nature and extent of geotechnical influences relating to route and structure distress; integrate this assessment into the on-going planning and structural evaluation of the damage noted during the geotechnical assessment mapping and subsequent observations by other transportation specialists.

The end product of field, office, and laboratory assessment should be a thorough understanding of the nature and extent of the requirements for rehabilitation as well as the development of actual and specifications for the required rehabilitation work.

2.7 ENVIRONMENTAL ASSESSMENTS

The environmental review process initiated in the United States in 1969, with the passage of the National Environmental Policy Act (NEPA), has had a profound effect on new transportation systems constructed. It is imperative that Agency design personnel take appropriate action to determine that they have not overlooked or otherwise devalued environmental factors that will be affected by construction and operation of each transportation project.

The basic requirements for incorporating geotechnical contributions into environmental reports are covered in Section 15 of this manual. Each Agency should take steps to determine that proper coordination exists between geotechnical managers and design personnel who are tasked with future transportation needs. Projects have been defeated when apparent negative aspects have been portrayed that the Agency has either not detected or which the Agency has not gathered sufficient data to prove for nonsignificant impact. Before making decisions relating to commitment of significant field investigation resources, it may be desirable for Agency management to call together its experts and consultants to discuss regional experience and to develop a plan to identify potential negative impact factors and to investigate their natures and magnitudes.

It has been previously stated that geotechnical data for environmental assessment can include almost all of the data that is necessary for project planning and design, although ordinarily in less detail.

As in the case of maintenance management, recurring experience in environmental impacts of a geological and geotechnical nature is also an important source of data which can be brought to bear in planning environmental assessment efforts.

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3.0 CONDUCT OF INVESTIGATIONS

Geological and geotechnical investigations for major transportation projects eventually involve considerable expenditures of professional time and in-house or subcontracted subsurface exploration services. The investigations nearly always represent successive levels of effort, each based on the results of previous work. Careful planning of such efforts is required so that data are interpreted after acquisition and the findings are applied to optimize each succeeding work task.

3.1 TRANSPORTATION PROJECT PLANNING

Transporation agencies conduct their most detailed geotechnical investigations in association with major, new projects. The overall planning procedure for conduct of these major projects has been described in TRB Synthesis Report 33. The procedural steps are:

- Corridor Study
- Route Selection
- Preliminary Design
- Final Design
- Advertising and Bidding
- Construction

It is the viewpoint of this Manual that geotechnical and geological personnel should be involved in providing basic data for decision makers at each procedural stage. Many transportation agencies and key officials prefer to employ this expertise selectively rather than routinely. Section 3 cites examples whereby geological and geotechnical input can be cost effective at all stages.

Geological and geotechnical information is basic to the design process; it must be produced in a timely fashion and be made available as one of the first-received packages of data. Transportation systems must be designed to accommodate the natural properties of soil and rock as well as the user's needs. Geotechnical data should be considered influential or even critical in all stages of each project.

Geotechnical personnel should consider interim release of data. Such releases, however, must be carefully described in terms of their provisional nature. Each stage-related or interim report or data release should reflect available data. Many products, such as geologic maps and subsurface profiles can be continually revised and updated to portray more accurate or completed interpretations, based on increased information and verified interpretations.

Final reports should generally include a summary of previously submitted data and interim reports. If agency policy permits, previous data may be considered superceded and should be discarded; later ambiguities resulting from multiple reports will then be avoided. The final geotechnical report should also present a clearly integrated summary of geological and geotechnical conditions and thereby remain as a single-source reference. Such a report should be made available at the start of final design and may, if agency policies permit, serve as a reference document for contract bidders.

3.2 ALTERNATE ROUTE SELECTION

Geotechnical personnel are in a position to provide a variety of preliminary assessments which can be made as the result of literature review, photogeologic interpretation, and limited field reconnasisance. In those agencies not now using geotechnical participation at the alternate route location phase, a trial example of such a product should be sufficient to gain acceptance of the concept.

An example of alternate route geotechnical mapping has been developed by the Soils and Geology Section of the Kansas DOT. Alternate route area maps such as these portray the distribution of geological and soil units that may be used by design engineers and others. The mapping is usually accompanied by a brief report pointing out the desirable and undesirable features of each map unit, as well as any geologic constraints (See Section 5).

In Kansas, such mapping is produced routinely as part of the corridor analysis of the Environmental Services Section and hence serves many uses, including the Environmental Impact Statement.

3.3 GUIDELINES FOR MINIMUM INVESTIGATIONS

Occasionally a question will arise regarding the level of investigation appropriate to a particular transportation structure or road segment. The question should be considered from the standpoint of what may be required of subsurface investigations as a function of natural conditions. If the geologic framework of the site or alignment is expected to be simple and geologic constraints (Section 5) are minimal or lacking then a minimal investigation may be warranted. An example of this might be a site in the great mid-Central till plain, in such states as Illinois, Iowa, or Indiana, where the till is of predictable grain-size nature and bearing capacity. However in valleys in the same province, localized, often softer, bodies of post-glacial deposits may present variable and less desirable conditions.

Experienced geotechnical personnel in transportation agencies generally agree that about 0.7 to 1.0 percent of total construction costs should be allocated for an "average-condition" subsurface investigation. For sites or alignments in areas which are underlain by poorer quality soil and rock units, or which may be impacted by geologic constraints, an increased level of expenditure should be budgeted. Costs of subsurface investigations, as a percent of construction cost are usually higher for rehabilitation projects. Typically, the subsurface investigation will break down to about 75 percent for engineering and about 25 percent for subsurface explorations.

Sites in areas underlain by predictable subsurface conditions and minimal or non-existing geologic constraints can probably be safely explored by subsurface investigations funded at about 0.50 percent of total estimated construction cost. It is believed, however, that few sites can be properly engineered on the basis of subsurface investigation expenditures of less than this amount.

3.4 PLANNING AND PHASING

Geotechnical investigations are sometimes difficult to manage and control from a scheduling and fiscal standpoint. In most projects, as soon as the socio-political questions of basic need and financing are answered, the design team is asked to initiate rapid determinations of siting, routing and general feasibility. Seasonal considerations are an important factor in performance of field investigations. Unfavorable weather conditions can easily add 15 to 25 percent to costs of field investigations.

Agency planning teams should include a geotechnical representative so that proper lead times and initial inputs are received and considered. This geologist or geotechnical engineer will be able to convert concepts into geotechnical impacts on the basis of his/her regional experience. The geotechnical representative will be able to provide conceptual planning information.

Conceptual planning requires only a minimum of information to begin formulation of the costs and schedules required for developing the entire geotechnical data package. As soon as the need for the project is recognized and the end points or general location of construction are identified, the Agency geotechnical staff should, within a matter of days, be able to present a synopsis of impact factors, as identified in this Manual. There are two levels of impact factors that can be identified:

- *Level-One Geotechnical Impacts:* These are well-recognized regional geologic and geotechnical conditions that will probably be encountered on most projects, regardless of size. Examples include areas of poor bearing capacity, geologic constraints found in the region and potentially adverse environmental impacts.
- *Level-Two Geotechnical Impacts:* Further considerations of the effect of geological conditions on planning, design, costs and environmental impact are those which are related to project size or magnitude. Examples of such impacts or considerations are stability of large cuts, the costs associated with developing and transporting construction material or with disposing of excavation waste, the costs associated with siting of large facilities in urban areas, and the costs of designing large embankments in seismic risk zones or areas of marginal bearing capacity or highly compressible foundation soils.

Level-One impacts will generally be recognizable to experienced geotechnical personnel from the very beginning of site or route identification. Level-Two assessments will begin to be identified as soon as the geotechnical team begins its project-related evaluations. Level-Two data will continue to appear throughout the subsurface investigation and must be identified and assessed immediately and reported to the overall design team as soon as possible. Level-Two data are generally crucial to final siting, dimensioning and elevation positioning of critical structures along the transportation project.

Phasing, as a means of controlling the direction and speed of field investigations, can be effectively utilized from the beginning of any project.

3.5 CONDUCT OF INVESTIGATIONS

Phasing of subsurface investigations can be developed on the basis of identification of the major components of the transportation system under consideration and the degree to which the size and magnitude of the component structures interrelate with expected geologic conditions. A generalized sequence of initial use of each of a number of subsurface investigation activities is discussed below. Although a particular activity is introduced in sequence, it may be necessary to repeat that activity later in the investigation.

3.5.1 Literature Search (Review of Existing Information)

The term "Literature Search" will be used in a broad sense to describe the accumulation of all existing information on a particular project prior to field investigation for the project. This "literature" may be print such as reports, journal articles, reports, maps, or non-print such as aerial photographs or geophysical logs, or even personal communications such as telephone conversations or letters. The sources of the "literature" may be well recognized public sources such as the United States Government Department of the Interior, Geological survey (U.S.G.S.) or the U.S. Department of Agriculture, Soil Conservation Service, state geological surveys or other state or municipal sources, professional journals or societies, project reports either in-house or otherwise, aerial photographs (remote sensing), well logs, and personal communication with individuals with local knowledge.

The level of effort expended on this review is established by the size and complexity of the project. However, regardless of the size of a project, some review should take place prior to going into the field. The minimal effort is to procure a plan and the topography of the site. The same document may serve both purposes. It may be either a plan surveyed for the project or a photographically enlarged (blow-up) portion of a topographic map. An expanded discussion of the sources of existing information appears in Section 3.7.

3.5.2 Study of Preliminary Plans

Many transportation projects are planned in phases, recognizing that unknown geologic and geotechnical conditions will be encountered and defined during preliminary site reconnaissance and exploration. Geotechnical personnel and the transportation system planners should maintain a close liaison, dealing with developing findings. The coordination should begin during concept development and continue through selection of all elements of the system alignment. Throughout this period, the geotechnical personnel should provide information on the expected nature of site conditions. Much of the geotechnical response should be forthcoming within days or weeks. Literature searches, Agency files, and a basic photogeologic interpretation can produce results often as detailed and useful as those depicted in the photogeologic interpretation of Figure 4-1. Project team discussions will define the alternatives to major structures such as bridges, viaducts, and tunnels, often identifying possibilities for shortening or reducing the size of such structures.

3.5.3 Formulation of Tentative Field Exploration Plan

At the completion of the office reconnaissance, the project team should be familiar with the expected rock and soil types in the project area; the general effect of topography, vegetation and near-surface groundwater conditions on site exploration plans, the probable depth ranges for borings; the need for supporting engineering geophysical surveys; and requirements for hydrogeological studies. It should be possible at this stage, for experienced geotechnical personnel to develop a scope of field exploration and field and laboratory testing that will meet design requirements, to within about 25 percent by cost.

3.5.4 Field Reconnaissance

A field reconnaissance can be planned on the basis of known project concepts and requirements and on the basis of findings from the literature search and image interpretation that represent the first activity of subsurface investigations. The reconnaissance should be based on formal objectives; that is, to determine the nature and areal extent of major geologic units, to gain an appreciation of their engineering characteristics and to develop the site region or site-area (within 8-km or 5-mi radius, or other better defined limitations) geologic detail. The other very important aspects of the field reconnaissance are to discover fatal flaw information which would limit siting or raise construction costs to an unacceptable level and to determine the nature and accessibility of required subsurface explorations. The final results of field reconnaissance are:

- Compilation of a preliminary geologic map of estimated geologic conditions over the entire area of interest;
- A scope of estimated field exploration activities and their locations
- A means for conducting a briefing on geologic conditions for the planning/design/environmental impact team

3.5.5 Field Geologic Mapping

While the subsurface investigation equipment and personnel are being readied, field geologic mapping (See Section 4 for details) can begin to answer the requirements identified during the field reconnaissance stage and those which have developed out of meetings with the planning/design/environmental impact team. Since all that is required by the geotechnical team is an appropriate topographic basemap and aerial photographs, this work can generally be started within days of establishing the project team guidance; seasonal weather conditions permitting. In areas of appreciable surficial soil overburden, the geologists assigned to mapping will probably also utilize the backhoe to augment visual inspections. Backhoe pits are placed (see Section 4.5) at critical structural foundation locations, in areas at which rock is to be exposed for detailed structural mapping, and in locations at which the nature of surficial geologic contacts are obscure and are needed to enhance the quality of surficial geologic mapping. The rate at which geologic maps can be produced is directly related to the level of detail and complexity of local geology. The geologic maps should be reviewed by the author and the field geologic supervisor each afternoon or evening, contacts inked, symbols checked and pencil coloring applied to insure correctness of overall map relationships.

3.5.6 Subsurface Explorations

Drilling, probing, and trenching should be undertaken only on the basis of a formalized plan. The plan should be based on geologic interpretations gathered to the time of initiation of field work and should be reviewed and updated according to findings during field geologic mapping and as a result of the subsurface investigation program itself.

Subsurface investigations should be reviewed on a daily basis by the field supervisor and brief discussions held between the geologists assigned to drilling rigs and other excavating equipment and the mapping geological team. Both teams should come away from the meetings with improved field plans.

3.5.7 Geophysical Surveys

Most geophysical techniques (Section 6) are employed on a linear basis and are anchored between or through outcrops or subsurface investigations in order to have a basis for interpreting the geophysical data. Most geophysical techniques require at least a hypothetical geological cross section, some physical property estimates, and an idea on the existence and depth to groundwater. Ideally, geophysical surveys should be initiated after the drilling program is about 25 percent complete. Later investigations may be required at locations on geophysical traverses that are open to question during interpretation of field results.

3.5.8 Hydrogeological Surveys

Traditionally, geotechnical engineers have been concerned about the presence and depth of groundwater in terms of its effect on construction conditions and its control over shear strength of soil and rock masses. These concerns are still with us, especially in regions characterized by relatively near-surface groundwater. With increased attention of the public and regulatory agencies toward environmental impact, hydrogeological surveys have taken on a new importance, as well as the location and definition of groundwater resources as they may be impacted by construction and operation of transportation systems. However, hydrogeological data necessary for design and construction generally suffices for environmental purposes. Frequently, the area of potential environmental impact of the system on groundwater is often broader than that of geotechnical concern.

Geotechnical workers often use two related professional specialty terms, *hydrogeology* and *geohydrology* in a synonymous sense, but they actually are two distinct specialities. Hydrogeology represents the expertise necessary to locate and define the presence and dynamics of general groundwater movement; geohydrology represents the more quantitative attempts to model or predict the occurrence and movement of groundwater on the basis of physical parameters developed by hydrogeologists. The fields of hydrogeology and geohydrology are staffed with professionals of a variety of backgrounds, generally in geology and civil engineering. However, most groundwater studies performed in the course of subsurface investigations are probably more related to geotechnical and environmental uses and are properly termed *hydrogeological* studies.

3.5.9 Materials Surveys

Construction materials are most valuable when they are located and made available within the construction site or ROW. The field reconnaissance and preliminary subsurface investigation should establish the general presence and quality of these materials. Most materials survey work in the site area can be accomplished at the time of the field investigation and probably should be phased to follow the previously mentioned activities. The subject of materials surveying is covered in Section 4.4.

3.5.10 Field Testing

As in other subsurface activities, field tests are frequently scheduled for performance in otherwise open borings and test pits. Field tests are conducted to determine the *in situ* strength, deformation, and permeability characteristics of key foundation soil or rock units. Since many field tests require the presence of a drilling rig, the tests should be scheduled as an integral part of the drilling program so as to avoid unnecessary remobilization of equipment.

3.5.11 Laboratory Testing

Laboratory testing is conducted to identify and correlate various soil or rock units and to determine their engineering properties. Most laboratory testing is undertaken on samples identified as being within the greatest zone of influence of foundation stresses, and for units which are felt to be so deformable as to govern foundation design at specific locations.

Many geotechnical staffs have operating procedures for identification and selection of samples for testing. One such favored method is to have drive samples or undisturbed piston samples arranged in order of sequence per boring and for a geotechnical engineer to view the exposed ends of each sample while reviewing the borehole log of the particular boring. Many geotechnical engineers prefer to use the torsional vane shear device or to simply make a thumb impression on the exposed tube or liner surface of the soil sample, to estimate soil bearing capacity and to enter this rough approximation on the boring log. Then, in overview, the most critical samples, representing foundation grades and other bearing surfaces are judged against estimated bearing capacity, and a selection of test samples and laboratory tests allocated against the budgeted scope.

Laboratory testing should begin as soon as specimens are made available and can be transported to the laboratory. Results should be processed and reviewed quickly and turned over to the office geotechnical staff for daily incorporation and review against boring logs and geologic maps. In locations in which the groundwater is high in total dissolved solids, especially in cases of brackish water, the specimens should be tested immediately in light of ongoing corrosion of the liner and cation exchanges present between soil/rock and liner; all of which tend to alter the engineering properties of the earth material.

3.5.12 Special Requirements

Many transportation systems involve structures of relatively large size in terms of the soil/rock-structural interaction, environmental impact, and susceptability to geologic constraints. Results of the field reconnaissance should have identified the possibility of natural conditions which may elevate project costs significantly or which may tend to make the project appear environmentally unacceptable. Actions should be taken during all field investigations to quantify these potentially negative aspects of siting and design and to provide insights or methods toward their mitigation. Most of the actions involve detailed geologic mapping, specialized geophysical surveying, and unusual or more detailed field and laboratory testing. These requirements, most of which are represented by methods and techniques discussed in the Manual, should be programmed and undertaken during the field investigation.

3.5.13 Photography

Photographs of the work in progress should be considered as a standard requirement for all subsurface investigations. The photography should be of a reasonably high quality and the agency should consider purchase of several medium-quality 35-mm cameras and provide basic instruction in their use to geotechnical personnel. As with other forms of permanent records, the photographs or color transparencies should be annotated with project number, stationing, date, and brief title. For conditions which may be difficult to describe, the use of stereoscopic photography as printed and included in the final report will be of use to many who use that document. Stereoscopic pairs may be made by focusing the camera on a center object, making one exposure, then stepping several steps to one side, focusing on the same center object and taking another exposure. The pair, when printed, should be trimmed so that the right-hand image is about 63 mm (2.5 inches) wide and spaced so that the left side of that image is spaced at about 63 mm (2.5

inches) from identical objects in the left-hand view. Test the pair for orientation using a pocket stereoscope, adhere the pair to a backing sheet, separate the pair by thin white tape and have a master photograph made of the stereogram. Care should be taken so as not to reverse the order of images and to produce a pseudo-stereoscopic image in which objects at depth (distance) appear to be artificially closer to the viewer.

3.6 REPORTS AND DRAWINGS

All activities undertaken by geotechnical personnel should be designed to provide data for specific use in the final report and its drawings. Design personnel should be tasked to provide adequate site and route topography and existing governmental topographic maps should be used to provide the basis for enlarged topographic coverage of the site area beyond the limits of actual construction. Topography and other cultural details should be photographically screened so that geological and geotechnical details stand out and apart from that background.

The degree to which geotechnical data are developed should be established with design personnel and the report language should be carefully chosen to avoid creation of impressions of conditions other than actually observed. Remarks concerning the absolute nature of existing or anticipated construction-related conditions should be avoided. That is, language that makes an absolute case for a specific condition is generally not warranted on the basis of the very limited nature of most geological or geotechnical observations and the extreme heterogeneity that is usually found in earth materials.

A general philosophy for establishing the scope of a subsurface exploration program is as follows:

By spending project funds, through its geological/geotechnical staff, the Agency hopes to procure an accurate and reasonably complete subsurface data package for use in design *and* as the basis for bidding by contractors. In spending agency money, geologists and geotechnical engineers have two primary goals, to provide:

- Data to produce a suitable and cost-effective design
- Data clear and concise enough to lead to a narrow spread of construction bids not containing large-risk dollar contingencies

The list of data elements likely required for even major construction projects is generally not long:

- Determination of perched water bodies and/or potentiometric groundwater surface
- Identification of engineering-significant soil/rock units
- Determination of a sufficient number of property tests to provide for reasonable design parameters
- Identification of top-of-rock
- Accurate recording of standard rock quality indicators
- Measurement of attitudes and other features of various structural discontinuities
- Recognition of reasonably apparent evidence of geologic hazards that could impact the project

3.7 SOURCES OF EXISTING DATA

3.7.1 USGS Quadrangle Maps

In the United States, the principal source of topographic maps and geologic reports is the United States Geological Survey. These maps are available in various scales, the most common is the 7.5 or 15 minute quadrangle. The appropriate quadrangle for the project may be located by reference to the state index map available from the USGS. Local vendors of maps are also listed on the index, as well as, deposit libraries.

Briefly, a topographic map is one that shows the size, shape and distribution of features on the earth's surface through the use of contour lines. A contour line connects points of equal elevation above or below a stated datum plane. The contour interval is the difference in elevation between two adjacent lines, it is stated in the map legend. The interpretation of topography is a basic skill necessary for the interpretation of any geologic map.

A site may be located by longitude and latitude, or proximity to bodies of water, topographic feature, numbered highways, population centers, or any other landmark. The unit of mapping is called a quadrangle. In the United States, it is generally available as a 7.5 degree or 15 degree sheet, each sheet being bounded by 7.5 degree of latitude and longitude or 15 degree of latitude and longitude, respectively. The scale on 7.5 degree maps is generally 1:24,000 or one inch on the map represents 2000 feet. In addition to the stated ratio, there is also a bar graph printed on the map that may be measured with a scale and used to determine distances.

Longitude lines converge toward the north pole, so the actual area covered by the map is greater in the south (about 70 square miles) than near the Canadian border (about 50 square miles). Originally, USGS mapping used a 15 degree minute quad, many of whch still exist and which have a scale of 1:62,500 or one inch equals approximately one mile. A 15 degree quadrangle includes the area of four 7.5 degree quadrangles.

Quadrangle maps are updated or revised periodically. Changes interpreted from current aerial photos are overprinted in purple on the original map. No field investigation is usually conducted. The map then carries both the original date and the date of the latest photo revision (e.g., 1959–63). If a complete revision has been undertaken, the map date reflects that investigation.

Interim maps called orthophotoquads are available prior to a complete revision. An orthophotoquad can be described as a mosaic of monocolor aerial photos corrected for displacement of tilt and relief with little or no cartographic treatment; usually there are no contours or elevations. Information on available maps or the status of mapping may be obtained from the USGS National Cartographic Information Center located in Reston, Virginia, or from the various mapping center offices.

Older editions of topographic maps can provide information regarding pre-existing conditions such as stream courses, ponds and drainage patterns which may have been affected by man-made structures as well as filling and grading resulting in topographic and hydrologic change.

The accuracy of any given map is dependent upon the quality of the information from which it was derived and the care with which it was drawn. There are USGS standards for vertical and horizontal accuracy for topographic maps. For horizontal accuracy, no more than 10 percent of the well defined map points tested, shall be more than 1/50 inch (0.5 mm) out of the correct position at publication scales of 1:20,000 or smaller. This tolerance corresponds to 40 feet on the ground for a 1:24,000 scale map and about 100 feet on the ground for 1:62,500 scale map. The standards for vertical accuracy require that no more than 10 percent of the deviations of test points interpolated from contours shall be in error more than half the contour interval.

Quadrangle maps may be altered photographically, enlarged, reduced or screened. However, it should be recognized that enlarging the scale does not improve the accuracy or increase the detail. It is, however, perfectly acceptable to use an enlarged quadrangle map as a base or site plan.

3.7.2 Bedrock and Surficial Geology Maps

Two of the most useful types of maps published by the USGS are the surficial and bedrock geology series. These show bedrock and soil conditions, superimposed on the basic quadrangle map described in section 3.7.1. The data shown may include depth to rock; locations of rock outcrops; estimated thickness, composition and engineering properties of the various soil types, geologic history and groundwater.

Inquiries concerning the availability and purchase of these maps should be made to the nearest regional office of the USGS.

Many other types of maps (thematic maps) are also produced by the USGS. These include; land use and land cover maps of quadrangles or regions, hydrologic maps, landslide maps, maps produced as part of professional papers and bulletins, geologic folios, water supply papers which often have maps and cross sections, aeromagnetic maps, slope maps, mineral resource investigation maps, and oil and gas investigation maps. A particularly interesting series for those working in the area is the Engineering Geology of the Northeast Corridor, Washington, DC to Boston, Massachusetts. (I-514,A,B,C) All of the above illustrate some facet of the geology of a particular region.

The survey also produces geologic map indices for the various states which list all of the maps for a particular location, identifiable by latitude and longitude regardless of source (USGS, state survey, journal, article, etc.). Each of the above documents produces a map which represents graphically the specific information desired or necessary for that report. There is no standard date base. The absence of a particular type of information only indicates that it was not significant (or investigated) for that particular report.

In addition to the USGS, there are several other federal sources of maps as well as state, local, academic, and commercial. These sources include NOAA (National Oceanic and Atmospheric Administration) which produces climatological and navigational maps. The Office of Surface Mining of the Bureau of Land Management can provide information on surface/strip mining both active and abandoned in a particular area. This information can be very critical for foundation design or waste disposal. The U.S. Forest Service can provide considerable information regarding land in its custody; the National Park Service has the same type of information.

3.7.3 Soil Survey Maps

Soil surveys have been produced as a cooperative effort in the United States since 1899. The majority of these surveys have been compiled under the direction of the U.S. Department of Agriculture (USDA) in cooperation with the Land Grant Colleges. Soil surveys include maps that depict the distribution of agricultural-type soil bodies. The agricultural soils are formed in the uppermost layers of unconsolidated materials (engineering soil units). The maps are accompanied by a narrative description of the soils and interpretive tables that give the physical and chemical characteristics and the agricultural and engineering aspects of each of the soil units. As of 1979, there were approximately 1200 U.S. counties represented by soil surveys to meet the current needs of users. Many states, as part of the cooperative soil mapping program between the USDA and Land Grant colleges, have also produced a single Soil-Association map of the state, generally at a scale of 1:750,000. Georgia is such an example (Perkins and Morris, 1977).

Soil survey reports and maps represent the third most useful source of existing information for highway design, following U.S. Geological Survey topographic maps and the various types of geologic maps in degree of usefulness. The usefulness of soil surveys varies greatly with the nature of the engineering project, with the age of the soil survey, with the expertise of the mapper, and especially with the background and experience of the user. Most of the information contained in soil surveys must be interpreted for engineering purposes despite the fact that the more recent surveys provide a variety of standard geotechnical information.

The crucial point of understanding regarding soil surveys is that generally the standard unit mapped is named for a *soil series.* The unit mapped represents an area on the landscape made up mostly of the soil or soils for which the unit is named. Most map units include small, scattered areas of soils other than those that appear in the name of the map unit. Some of these soils have properties that differ substantially from those of the dominant soil and thus could significantly affect engineering use of the map unit. These soils are described in the description of each map unit. More than 12,000 soil series have been identified in the U.S. (McCormack and Flach, 1977).

3.7.3.1 Development of Soil Surveys in the U.S. From 1899 to 1938, county soil survey maps were compiled on a basis very similar to Quaternary or surficial geologic mapping. Many of the maps were compiled by geologists and a tradition of association of the soil series with parent geologic materials was established. Most of the maps were produced at a scale of 1:62,500 and were printed in color. Soils were classified using soil series to represent the central concept for each soil. Class limits were poorly defined. Each soil series was named for the geographic location at which it was first described and the mapping generally followed Marbut's 1913 description of a soil series (USDA, SCS, 1964):

> "A group of soils having the same range in color, the same character of subsoil, particularly as regards color and structure, broadly the same type of relief and drainage, and a common or similar origin."

In 1955, work began on a soil classification system that would have more precise categories to enable more quantitative and reliable interpretation of soil surveys. This resulted in "Soil Taxonomy" (USDA, SCS, 1975, Agriculture Handbook No. 435). Properties used to define classes in "Soil Taxonomy" have precise quantifiable limits and are generally properties that influence use and management. From 1951 through 1965, soil taxonomy was refined, resulting in what is known as the 7th Approximation with its six categories:

1. Order
2. Suborder
3. Great Group
4. Subgroup
5. Family
6. Series

The series is the lowest category in the system and, as such, provides the most site-related information of geotechnical importance. The use of soil series data of any age can be useful in the exploration and design of highways.

3.7.3.2 Soil Survey Mapping Philosophy. Soil survey maps are the product of an attempt to depict the areal coverage of parcels of soil with a similar, average solum or soil profile to about 2 m of depth. The maps are compiled first by photointerpretation techniques and are then field checked by foot traverse, and observations of road cuts, exploratory auger cuttings, and test pits. Aerial photographs are used as the map base. The soil scientist preparing the map looks initially for surface-visible indictions of the nature of the underlying soil solum. These indicators are landform, slope, vegetation type, surface water or moisture, and geomorphic position. Photointerpretation techniques aid the mapper in determining soil boundaries. Nearly all SCS soil survey mapping is now printed at scales ranging from 1:15,840 to 1:24,000, with much of the mapping being performed at 1:20,000. County surveys also contain useful summary soil maps at scales equal to or small than 1:62,500.

Map units on most soil maps are named for phases of soil series. Soils that have profiles that are almost alike make up a soil series. Except for allowable differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement. Soils of one series can differ in texture of the surface layer or in the underlying substratum and in slope erosion, stoniness or other characteristics that affect their use. On the basis of such differences, a soil series is divided into phases. The name of a soil phase commonly indicates a feature that affects its engineering use or mangement. Phase designation will allow for geologic interpretation of the parcels into different surficial geologic units.

In most cases, soil survey mapping will subdivide an area into more detail than a corresponding geologic map of the same area, and the geologist or geotechnical engineer will be faced with the prospect of lumping or combining soil series or phase parcels into surficial units of interest to engineering design studies.

3.7.3.3 Conversion of Soil Survey Classifications. In order to make full use of soil maps with map units comprised of soil phases, the geotechnical engineer or geologist must review the physical description and textural classification of the soil map unit. The dominant horizons of each series is given a textural classification. The textural terms used are defined using a textural triangle. Of more help to the engineer is the particle size and mineralogical class given for the family of which the series is a part. Data for these classes are averaged and given for the subsoil as a whole. Conversion of the USDA texture and particle size classification to engineering classifications can be aided by comparison with textural triangles relating percentages of sand, silt and clay to each established soil type. Figures were prepared by Handy and Fenton (1977) as an aid to this conversion. Use of these diagrams gives an idea of the composition of each soil series, and the interpreter should bear in mind the changes in the system with each modification. The soils of modern soil surveys have been classified according to the AASHTO and USCS schemes eliminating the need for this conversion.

To convert soil series mapping further for engineering purposes, one may use the textural triangle developed by the US Army Engineer Waterways Experiment Station which is keyed to the 1940–1965 period surveys. For further comparison of soil survey unit properties with those of the AASHTO and USCS schemes, Handy and Fenton (1977) have developed additional triangles relating sand content to Atterberg limits and a triangle depicting the size-content basis for the AASHTO classification.

In all of these schemes for conversion of properties, one must bear in mind that design-related geotechnical data will never be generated by interpretation. The conversions are appropriate only for the purpose of grossly categorizing each map parcel into broad quantitative engineering categories, such as:

- Cohesionless vs. cohesive
- Plastic vs. non-plastic
- High permeability vs. low permeability
- Dense vs. loose
- Hard vs. soft
- Wet vs. dry
- Easy to excavate vs. hard to excavate

The most confusing aspect of conversion of soil survey data to engineering usage is that of *loam*. Loam is essentially a cohesive soil of intrinsic value for agricultural purposes. The term itself is meaningless for engineering purposes and should be translated according to the scheme of Handy and Fenton (1977), with verification coming from grain-size analyses.

3.7.3.4 Engineering Data from Soil Surveys. USDA soil surveys prepared after 1965 generally present a considerable amount of engineering property data collected at identified locations for single soil series. These data are related to specific depths in the solum or are keyed to major soil horizons of the solum. As with conversion of geologic map data for engineering purposes, soil survey data can be used to obtain a relative appreciation of average engineering properties. Some soil survey data represent egineering property determinations that are developed by the SCS on the general engineering characteristics of soil series within individual counties. Other engineering property data are provided by State DOTs and other organizations. Among the engineering data that are commonly provided in modern County Soil Surveys are:

- Seasonal moisture content
- Density
- Texture; refers to the (−) 2mm fraction; terms such as fine silty, etc.
- Percent coarse fragments greater than 3 inches
- Percent organic matter
- Atterberg limits
- Clay mineral type
- Reaction (pH)

The Michigan Department of State Highways and Transportation has been cataloging the engineering properties of standard SCS soil series for more than 30 years. Programs have subsequently been initiated in South Dakota (Crawford and Thomas, 1973), Ohio (Johnson, 1973), and Wisconsin (Alemeier, 1974). The Wisconsin system carries the average soil series properties in looseleaf handbook form relating updated series properties for an average solum, along with what may be generally expected for the series in various Wisconsin locations.

3.7.3.5 General Use of Soil Survey Data. Soil survey data are most useful in the preliminary planning stages of the project. The data should be used to consider the relative cost and suitability impacts of alternative routes and to plan the general nature of the subsurface explorations that will follow. The general estimates of conditions in the various soil parcels of mapped areas are as follows:

- General suitability/unsuitability
- Depth range to bedrock
- Groundwater conditions
- General slope stability
- Erosion susceptability
- Excavation characteristics
- Frost susceptability
- Heave or collapse potential
- Potential borrow areas
- Degree of uniformity or complexity of soil conditions

Soil Survey maps are also excellent sources of projecting data beyond the normally-mapped right-of-way, especially in locating borrow materials and estimating environmental impact such as surface and culvert erosion and related sedimentation.

3.7.4 Other Sources of Information

Other sources of geologic maps are the individual state geologic surveys, state DOT's, municipal highway or public works departments, regional authorities such as River Basin Commissions, Turnpike authorities, and airport commissions. Professional society proceedings and journals, academic departments, and various specialty organizations can often provide information of use. Much of the above information may be in manuscript, that is, unpublished.

The type of information produced by the state geologic surveys will be similar to that of the USGS. Individual surveys have their particular specialties, as well as level of activity. Fairly often the state geologic survey may be part of the environmental mangement group or located in the state university. Boring logs for water wells are often archived by state surveys. Rock core collections are also sometimes retained.

Local historical societies, historical commissions, libraries, and tax assessor's departments may also be a source of maps or reports, particularly old atlases whch illustrate pre-existing uses of the site. The type of information of interest in an urban site is quite different from that of a rural site. Obviously, the extent of office research is determined by the scope of the project and the ease of acquiring information.

The agency or consulting firm itself may have considerable information in-house as the result of previous investigations of the same or an adjacent site. Not only completed projects but those that were proposed but not completed should be consulted.

Remote sensing or photointerpretation is discussed in Section 4.0. A minimal level of effort of photointerpretation is appropriate for a small site, however a right-of-way for several miles of highway would merit a thorough complete photointerpretation.

Finally, one of the most elusive sources of information yet one which may be extremely valuable is the personal communication. Stated simply, it is a few telephone calls either to other geologists, geotechnical engineers, government officials or anyone else familiar with the site to find out what they may know about it. Obviously, information gathered casually, must be verified carefully.

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4.0 FIELD MAPPING

Field geologic mapping is a means by which all subsurface data are made useable for design engineers. The engineering geologic map portrays the estimate of conditions at any location in the site area or along the ROW. Field Mapping represents an interpretation of observations by the engineering geologist or geotechnical engineer to produce a two-or-three dimensional representation of the geologic fabric of the project area. This mapping is carried on at all scales and with variations in symbols and detail to answer specific design needs.

4.1 General

Field maps are a record of the observational coverage of an area. The area of interest may be several kilometres of preliminary transportation routing, alternate corridors for routing, the face of a rock quarry or gravel pit, the interior of a pilot bore or inspection shaft for an underground transit station, or the site of a roadway failure requiring immediate attention. Maps are two-dimensional representations of the extent of units of earth materials of similar properties. A variety of symbols are used to portray the nature of the materials, their discontinuities and other flaws, and the presence of all manner and types of indicators of geologic phenomena (such as geologic constraints; see Section 5) that affect transportation system design. Varnes (1974) has compiled a treatise on the development and philosophies of geologic mapping. The techniques of various types of field mapping are discussed in the present chapter.

4.2 RECONNAISSANCE MAPPING

In the early stages of project planning and feasibility studies, the primary factors controlling cost and environmental acceptability of projects are usually unknown. Reconnaissance mapping is the first step to obtain project-related physical data to shape the investigations and design program.

4.2.1 Purpose

Reconnaissance mapping should be undertaken as the first step in gathering project-related subsurface data. The techniques of reconnaissance mapping are the same for all geologic and geotechnical mapping; careful observation and accurate graphical reporting of all pertinent information and observations. Reconnaissance mapping involves limited foot traverses of the area of interest, using aerial photographs and topographic maps as a data collection base. The goal is to generate general classifications of material type, landform characteristics, the nature of surficial geologic and soil units, general groundwater conditions, and an assessment of geologic constraints.

4.2.2 Levels of Effort

Reconnaissance mapping should suit the needs of the project and of the specific members of the design team. Prior to leaving for the field, the assigned staff members should discuss the data needs of the individual who has requested the mapping. Mapping may be accomplished on field-compiled sketch maps generated by pace-and compass methods, by use of soft, colored pencils on matte-surfaced aerial photographs, by inked lines on rapid-development photographs, or suitably enlarged topographic basemaps.

4.2.3 Office Reconnaissance & Literature Search

A thoughtful preparation of field work will remove many time-consuming obstacles to efficient reconnaissance efforts. The project engineer should be consulted as to the basic concept of routing, structure design or alternate corridor locations. The individual in charge of the field party should study available topographic coverage to determine access and foot traverse conditions and then develop a concept of conducting the reconnaissance. A group consultation with parties to the project is helpful. At this time the reconnaissance party chief outlines his program for conducting the field work, including the area(s) of interest to the planning and design team. Once the area of coverage has been mutually agreed upon, the reconnaissance party chief should continue to collect accessory information for continued use in the office, prior to entry into the field. (See Table 4-1).

Office review of all available materials (Section 3) will help to determine the level of reconnaissance effort and to identify key factors which should be examined in the project effort.

4.2.4 Field Reconnaissance

A well planned field reconnaissance is needed to verify the office reconnaissance. A field reconnaissance program should be undertaken only after the project team has a good concept of the main requirements of the project and after the geotechnical personnel have determined the apparent geological conditions in the site area. The reconnaissance mapping should begin by inspecting road cuts and drainage-courses and bank exposures adjacent to roads. The main objectives of these observations is to confirm the general types of soil and rock present in the site area. Almost always, geological formations have distinct lithologic characteristics and these generally control most of the resulting engineering properties of the materials. Therefore the project-related soil and rock stratigraphy usually results in a further amplification of existing classical geologic studies.

Foot traverses should be made next, to examine outcrops and landforms located on office photogeologic maps. A well-charted foot traverse should provide up to 25 percent of the data requirements of the project. The person(s) undertaking the foot traverses should do so with the final objective of being able to brief the project team about most of the key issues concerning route location and geotechnical design. These objective assessments are listed in Table 4-2, Fig. 4-1.

4.2.5 Field Reconnaisance Report

The field reconnaissance report should define most of the key planning and design issues and estimates of their effects on design and construction. The report should form the basis for the site investigation plan, its scheduling, its priorities, and its budget. At a minimum, the report should include the following elements (Table 4-3, Figure 4-2):

4.3 ENGINEERING GEOLOGIC MAPPING

Engineering geological maps are constructed for the purpose of identifying conditions which will affect the

Table 4-1.
First-Order Determinations from Non-Geologic Source Materials

Nongeologic source materials can be used to estimate the presence and nature of soil versus rock; cohesive versus cohesionless soil; the general origin of soil units (e.g., windblown versus alluvial; beach versus fluvial, etc.)

Topographic Maps	Landforms can be interpreted by slope angle, degree of planarity or convexity/concavity, contour irregularity, and stream cross section
Agricultural Soil Maps	Descriptions of soil associations generally provide an opinion as to the parent material of the soil, often that material lying directly below the soil solum
Aerial Photographs	As discussed in Section 4.5, an excellent source of information; the usefulness of photointerpretation is limited only by the quality and scale of the photos and the skill of the interpreter.
Well-Drilling Logs	Although these logs are extremely variable in quality, the basic differentiation between cohesive and cohesionless soils and rock are almost always obtainable; water levels at the time of drilling or well installation should also be available.
Engineering Soils Maps	Basic landforms are subdivided into units containing similar engineering characteristics such as origin, soil texture, drainage, and slope. Several States have prepared these maps on a statewide (New Jersey, Rhode Island), county (Indiana, Illinois, Washington) or corridor basis (New Mexico, Indiana, Maine). (See Mintzer, 1983).
Existing Borings	Borings from previous investigations in study area or in close proximity can be correlated to similar soil-terrain conditions in study area.

Table 4-2.
Key Exploration Factors Definable by Field Reconnaissance

Stratigraphy	Definition of most of the soil and rock units that will be ultimately encountered by sursurface exploration
Exploration Locations	Definition of the approximate location and traces of drilling test pitting, trenching and geophysical surveys; estimation of approximate depths
Accessibility	Specification of approximate routes of access into each of the exploration locations; determination of types of equipment necessary
Key Outcrops	Definition of outcrops or exposures that warrant further investigation in terms of structural geologic mapping or petrologic classification
Water	An estimate of the general nature of groundwater and surface water regimes in the site area; development of concepts for further investigations
Existing Slopes	An assessment of the stability factors of major slope-forming geologic units
Material Sources	A tentative estimate of the nature and general availability of various categories of aggregate and borrow materials
Geologic Constraints	Identification of geologic conditions which may tend to adversely affect any of a number of project development plans; devise methods of investigating the degree of potential impact
Environmental Considerations	Identification of potential impacts of the project on water, soil and rock in the site area, based on the observed or presumed nature of each basic material type, the existing topography and the preliminary project development plan.

design, construction, maintenance and overall economics of the transportation system. They provide a means of combining outcrop geologic observations; drillhole, test pit and trench logging; photogeologic contacts; and geophysical survey results into a composite representation of the breadth and extent of each geologic soil and rock unit that is identified at the ground surface in the site area.

These geologic maps are similar in many respects to classical geologic maps, but differ essentially in their adherence to identification of individual map units strictly on the basis of observed engineering charac-

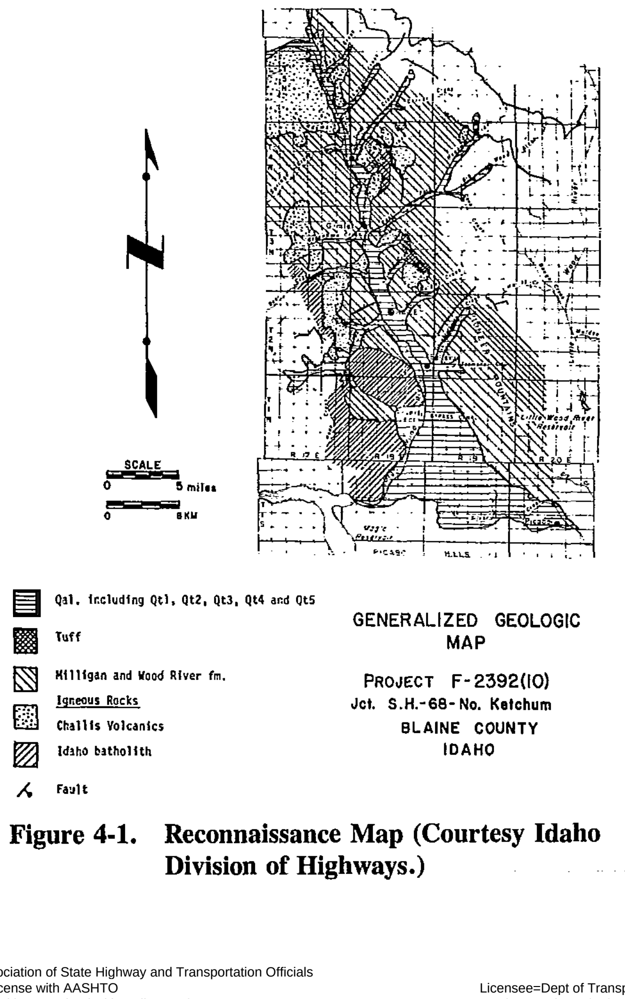


Figure 4-1. Reconnaissance Map (Courtesy Idaho Division of Highways.)

Table 4-3.
Elements of the Field Reconnaissance Report

- A summary of the geologic framework of the site area.
- A stratigraphic listing of soil and rock units expected to be encountered in field explorations and subsequent mapping as well as a draft geologic map legend with tentative lithologic names and map symbols.
- A sketch reconnaissance map on site-area scale. This scale is one level of scale smaller than the ROW map that will be used in site or alignment mapping.
- Locations, numbers and depth ranges for recommended or suggested exploration activities; boreholes, test pits, trenches, geophysical surveys, observation wells, etc.
- Locations or areas requiring special attention in field mapping or subsurface exploration.
- Basic questions to be answered relating to groundwater environmental concerns and geologic constraints.
- An opinion relating to the probability of locating and developing significant quantities of construction materials in the site area.

GEOLOGIC RECONNAISSANCE - LOCATION (16-610)

16-611 General

Definition: The initial study made approximately five years before the confirmed project alignment based on a geologic reconnaissance of a selected corridor.

Report To: District Engineer
District Location Engineer
District Landscape Architect

Copies: Environmental Planning & Corridor Study
Materials Supervisor

Field Review:

	Attendance
District Materials Engineer)	1 of 2 required
District Geologist)	
Supervising Geologist)	required

16-612 Purpose

The report provides estimating data for the District Location Engineer and background data for the Environmental Impact Study Report. The report provides information relative to one or more lines within the corridor under study.

16-613 Report Composition

The report will cover the following subject matter:

16-613.1 Introduction

16-613.2 Conclusions

16-613.3 Evaluations

1. General Geology
   - A. Stratigraphy
   - B. Topography
   - C. Soils & Vegetation
2. Drainage
3. Groundwater
4. Geologic Hazards
   - A. Existing and Potential Landslides
   - B. Slope Stability
   - C. Fault Influence
   - D. Joint Systems
   - E. Flood Plain Deposition and Influence
   - F. Seismic Risk Assignment
5. Construction

(Example Report)

Figure 4-2. Outline of Geologic Reconnaissance Reports prepared by the Idaho Division of Highways.

teristics. The characteristics used to differentiate map units are those visually-apparent during mapping: hardness, degree of weathering and alteration, basic lithology, grain size, color, and degree of induration. Other characteristics related to rock mass properties, such as nature, continuity and frequency of discontinuities should be considered when identifying separate geologic units. Although the basis for map unit definition is strictly one of engineering character, the rock or soil is described according to established geological and engineering terms with the use of correct geological terms relating to the makeup or lithology of the rock.

Two main types of surficial maps represent the state-of-the art engineering geological mapping for transportation systems: engineering unit maps and engineering geomorphological maps. Both techniques recognize that the landform observed is the key to the nature and origin of the rock or soil material underlying the ground surface at that point, and that the lateral and vertical extent of the unit is indicated by the form and lateral boundaries of each landform unit. Because of this relationship, most mapping begins with photogeologic interpretation of the right of way (ROW) photos obtained by the Photogrammetric Department of the DOT, as well as other standard photographic coverage of the region, such as that of U.S. Department of Agriculture, or other governmental agencies. Individual DOT regions may have established the general geomorphological and stratigraphic relations of the physiographic regions in which they operate.

Prior to field mapping, it is desirable to establish an informal project-related mapping specification noting the general symbols to be used and the rock types to be expected. The symbols and units can be modified or extended on the basis of observations made during mapping, but uniformity of Agency mapping will be maintained.

The general procedure for engineering geological mapping is as follows:

1. Using aerial photographs or other remote images;
   a) identify separate landforms
   b) define the area of individual bodies of various surficial geologic units
   c) assign tentative origin and physical properties to each geologic unit
   d) complete site-area photogeologic map
   e) plan for a field reconnaissance
2. Conduct a field reconnaissance of outcrops and road and drainage cuts.
   a) devise a list of expected engineering soil and rock units and symbols for continued field mapping
   b) select key locations for briefing of the field mapping team
   c) plan for the priorities of mapping; traverses and key locations for inspection
   d) develop a tentative subsurface exploration plan
3. Conduct the engineering geologic mapping of the ROW and the site area
   a) locate outcrops and define areas at which each engineering soil or rock unit is described as being representative
   b) review the ROW geologic map and revise the exploration plan to inspect key highway structural sites and locations believed to be important to the geologic and geotechnical interpretation

4. Conduct the field exploration plan, relying on drill rigs for observations at depth in areas of primary emphasis (bridge, piers and abutments, major cuts and fills, bodies of poor quality rock or soil, etc.)
   a) log the explorations, relate the observations to the geologic map units
   b) modify geologic contacts to reflect findings of borings and test pits and trenches
   c) determine the need for additional explorations including geophysics, to refine mapping in areas of question
   d) conduct these support explorations
   e) revise the geologic mapping

4.3.1 Project Area Geologic Maps

About the smallest scale of geologic mapping that will be appropriate for transportation system work will be that prepared for routes. The map should be at a scale small enough to show the interrelationships of geologic units over a wide enough band or strip to offer some degree of routing choice. Project area geologic maps are usually constructed along a rather narrow strip of land, just wide enough to contain the roadway, cuts, fills, and the adjacent areas to be impacted by construction. The mapped strip should also be wide enough to contain some prospects for aggregate and borrow locations. The average scale for route maps is about 1:6000.

4.3.2 ROW Geologic Maps

Geologic maps of the alignment of roads and rail lines are essential to development of slope stability assessments, bearing capacity, roadbed settlement computation, rock excavation plans, control of groundwater, location and qualification of borrow and aggregate sources, and a number of other critical design-related judgments. ROW geologic maps are generally prepared at about 1:600 scale, or at about ten times the detail of the site area geologic maps. Since ROW maps are by nature much longer than wide, scales larger than about 1:600 become too cumbersome and the engineer using the data begins to lose the appreciation of geologic relationships along the ROW. Most ROW geologic maps are not cluttered with detail because of the relatively low density of exploration locations, at this rather large scale.

4.3.3 Site Geologic Maps

The methods of compiling site geologic maps are less standardized than those prepared for routes. The site maps should be compiled strictly for the purpose at hand, such as bridge foundations and a variety of remedial treatments of natural damage. Often the site is so restricted as to be without significant topographic relief or it must be covered at a scale far larger than existing topography. Judgments must be made as to the level of detail that is required to formulate the design of remedial treatment necessary at the site.

If specific design recommendations and quantities are not required, a geologic summary can often be made on the basis of a Polaroid-type photograph backed up with a finer resolution panchromatic or color negative to be developed and printed later. The instantly-developing photograph can be used for annotation of field notes.

Wherever field geotechnical recommendations are to be developed for immediate remedial action, a more accurate portrayal of site geometry is usually required. In the event that appropriate topographic maps or formal survey assistance are not available, a surprising amount of detail and accuracy can be achieved using a geological compass (Brunton) and stadia rod. However, planetable and alidade mapping, or more precise mapping by terrestrial photogrammetry, may also be required if a structure is to be installed and stability considerations are apparent.

4.3.4 Other Special Geologic Maps

Most special geologic maps produced for transportation projects are nonrepetitive in nature. Some maps, such as those compiled for evaluation of off-ROW borrow and aggregate sites, consist mainly of planimetric sketches or simple geologic maps developed in enlarged USGS topographic basemaps or enlarged aerial photographs. The main objective of materials survey maps is to estimate the areal extent, depth, and volume of recoverable materials of certain specifications.

Segments of the ROW that may encounter severe groundwater problems (drainage) or which must be considered for impact on abutter's wells will probably be analyzed with the assistance of observation wells so that the existing piezometric surface can be determined. In this case, the ROW geology is plotted, along with the extend of the projected cuts and fills and a before-construction and after-construction estimate is made of the level of groundwater, along with the anticipated directions of flow and the equipotentials representing the piezometric surface (See Section 8). The main objective is to predict the shadowing effect of road cuts on groundwater flow patterns and to define the expected seepage conditions along the cut faces.

Metropolitan area transit authorities who are now undertaking serious planning measures for initiation or extension of subway systems often contract for broadscale geologic evaluations of the areas of their main traffic-flow patterns. The past decade has seen release of several of these studies, generally in cooperation with State surveys or the U.S. Geological Survey.

4.3.5 Integration with General Project Photointerpretation

Most agencies have photointerpretation needs that extend beyond the primary design information collected during geologic mapping. Land utilization patterns and utility corridors affect both the cost of construction and the environmental acceptability of most transportation projects. Kansas DOT has developed a *corridor analysis* methodology which is applied through its Environmental Services Section as a first action in the planning stage of a route project. A photomosaic is prepared at 1:24,000 scale, directly overlaying existing USGS topographic coverage. A series of derived maps are prepared as line overlays to the photomosaic. The primary overlays are:

- Soil and geologic conditions
- Drainage divides
- Utilities
- Land utilization

The analysis is completed in the form of the corridor maps and an explanatory text. Locations of special interest may be located on the imagery and further illustrated by enlarged aerial photograhic stereograms. Maps such as these represent a useful method of assessing route alignment alternatives, which are compared simply by overlaying the subject route on each of the corridor maps.

4.3.6 Special Methods of Geologic Mapping

Geologic maps are made as a representation of field geological observations. Nearly all geologic maps are made up of geologic contact lines which are drawn on the field basemap on the basis of observations taken at outcrops and other direct indications of contacts between geologic units. A variety of subsurface exploration techniques are used to supplement the surface observations at outcrops. These accessory explorations are especially useful whenever the contact relationships are obscured by vegetation and the surficial soil mantle.

Wherever slopes are gentle and offer little indication of changes in soil or rock units, backhoe pits or trenches can be used as the basic form of supplementary information. Pits and trenches permit direct visual inspection and study of a continuous horizontal and vertical section of earth materials. Examples of some special methods of geologic mapping techniques are given in the following subsections.

4.3.6.1 Test Pits. Test pits are generally excavated by backhoe and logged by visual descriptions of the excavated spoil and pit walls. Good sense and Federal regulations (Occupational Safety and Health Administration, 1974) dictate that the geologist should not enter unshored test pits greater than 1.5 m (5 ft.) in depth.

Test pits can be placed at locations where the geologist wishes to locate approximate contacts between surficial soil units or to determine depth to rock or the nature of weathering between top of rock and the surficial soil units. Small backhoes, which are mounted on rubber-tired tractors, can reach to about 4 m (12 ft.) of depth. The usual bucket capacity is 0.3 m^3 (3/8 cu. yd.). For depths in excess of 4 m and for dense soil units, larger backhoes will be required. Between eight and twelve pits can frequently be located, dug, logged, samples, observed for water inflow, and backfilled in a day. A sample test pit log is included in Appendix A. If such are present, the number and approximate total volume of boulders in the pit should be noted. This may be of crucial importance in planning site development and subsequent use of spoil for earthwork. Shallow observation wells can be installed in test pits, but the geologist should be aware of the possibility of the looser replacement spoil of the pit acting as a collection sump during heavy rainfail, thus giving erroneous groundwater levels.

4.3.6.2 Exploration Trenches. Trenches are lengthwise extensions of test pits. Their use is discussed in detail by Hatheway and Leighton (1979).

4.3.6.3 Exploratory Shafts. Underground structures, such as some subway stations in urban areas, are often designed with complex geometries. This can result in unfavorable stress concentrations in wall rock, making structural assessments of the rock an important facet of site exploration. When little is known of rock structure in the site area, a combination of expensive oriented core borings (see Section 7), reorientation of unoriented rock core, and exploratory shaft mapping may be required to adequately assess the nature, attitude and spacing of rock discontinuities.

One or more exploratory shafts may be churn-drilled or calyx (a large-diameter rock core) drilled, wedge-separated and lifted from the boring) drilled to create an accessible shaft about 1 m (3 ft.) in diameter. Logging of such shafts, at scales of 1:10 to 1:15 can provide an excellent summary of relationships not otherwise seen even in oriented rock core.

4.3.7 Rock Structure Mapping

Rock structure mapping entails observing, locating, measuring and recording lithologic contacts and various rock discontinuities which provide information on the orientation of rock masses and their bulk engineering characteristics. As discussed in Appendix E, the engineering properties of rock should be considered at two levels, those of intact rock (or the hand specimen and laboratory sample level) and those of the rock mass. The manner in which intact-rock and rock-mass engineering data are evaluated and used in engineering geological and geotechnical analyses lie outside the scope of this Manual, however.

Two primary methods are commonly used to present the results of a rock structure mapping program; 1) geologic maps showing the location so lithologic contacts and the presence and orientation of contacts and discontinuities, and 2) statistical plots of structural geologic measurements. The measurements are typically made with a Brunton compass and consist of strike and dip of faults, joints, foliation, shear planes, zones of broken rock, dikes, sills, veins, and contacts. Each of these geologic features should also be described and classified according to the methods presented in Appendix E.

Maps can be compiled in the field and each observation should be station-related to fieldbook notes describing the nature of discontinuities. Observation stations consist of individual rock outcrops or test trenches excavated to bedrock surface. The structural geologic observations are tabulated according to station, attitude, and characteristics (i.e., bedding, joints, shears, etc.) and should be provided as part of the raw data of the final report (Section 10).

An example of a site area geologic map prepared for an Interstate Highway Extension project is portrayed as Figure 4-3. The mapping scale should always be selected such that the geologic data obtained during field mapping can be presented in sufficient detail to clearly delineate site geology. If the scale permits, limits of rock outcrops and test trenches should be indicated, to separate the interpretive from the observational data. Depending on the size of a project site and the regional geology, it is possible that structural geologic conditions may vary significantly across a study area. If this is the case, similar geologic observations and measurements from various stations may be grouped together into *structural domains.* Approximate boundaries for the structural domains may be presented on the geologic map, as shown on Figure 4-3. During subsurface explorations, the geologic map may be improved with additional structural geologic data obtained from core borings. Fault traces or geologic contacts may be projected with greater certainty utilizing the boring information. Also, the oriented core technique of rock coring can be utilized to supply supplemental strike and dip measurements of subsurface rock discontinuities.

Spherical projections provide a convenient tool for graphical presentation of geologic data. Field measurements of rock discontinuities may be plotted on an upper hemisphere equal area steronet and may then be interpreted statistically to provide preferred orientations of joint sets, foliation, shears, etc. to be used in engineering design analyses. The equal-area plot is made up of poles lying perpendicular to planes represented by measured strike and dip of discontinuities. Each measurement is represented by one pole, and the origin of the pole is the center of the hemisphere. An example of such a plot is shown as Figure 4-4. Different symbols can be used for the various types of discontinuities, or all discontinuities can be represented together using only one symbol. Comprehensive discussions on the use of sterographic projections has been given by Hoek and Bray (1974) and Goodman (1976).

In order to identify preferred orientations of systems of discontinuities, contouring of polar point density may be performed. To arrive at a density contour plot, the polar point plot is divided into patches of equal area and the occurrence of observations in each patch is counted and translated into percent density. A contoured upper hemisphere polar point density plot of rock discontinuities is shown as Figure 4-5. If an extensive number of strike and dip observations have been obtained during field mapping, it may be appropriate to use a computer program with an automatic plotting routine, such as developed by Mahtab et al. (1972). In addition to providing polar point plots and polar point density plots, the computer programs can output statistical parameters and mean orientations for "clusters" of observations representing bedding, joint sets, etc. The amount of scatter for each joint set should be evaluated in engineering design analyses.

4.3.8 Tunnel Silhouette Photography

Tunnel silhouette photography is a means of recording single-station, cross-sectional shape, and overbreak through flash-photography. The technique is applied primarily to rock tunnels excavated by conventional drill-and-blast techniques. In drill-and-blast excavation, deviations from the design tunnel cross-section are likely to occur and are of concern to

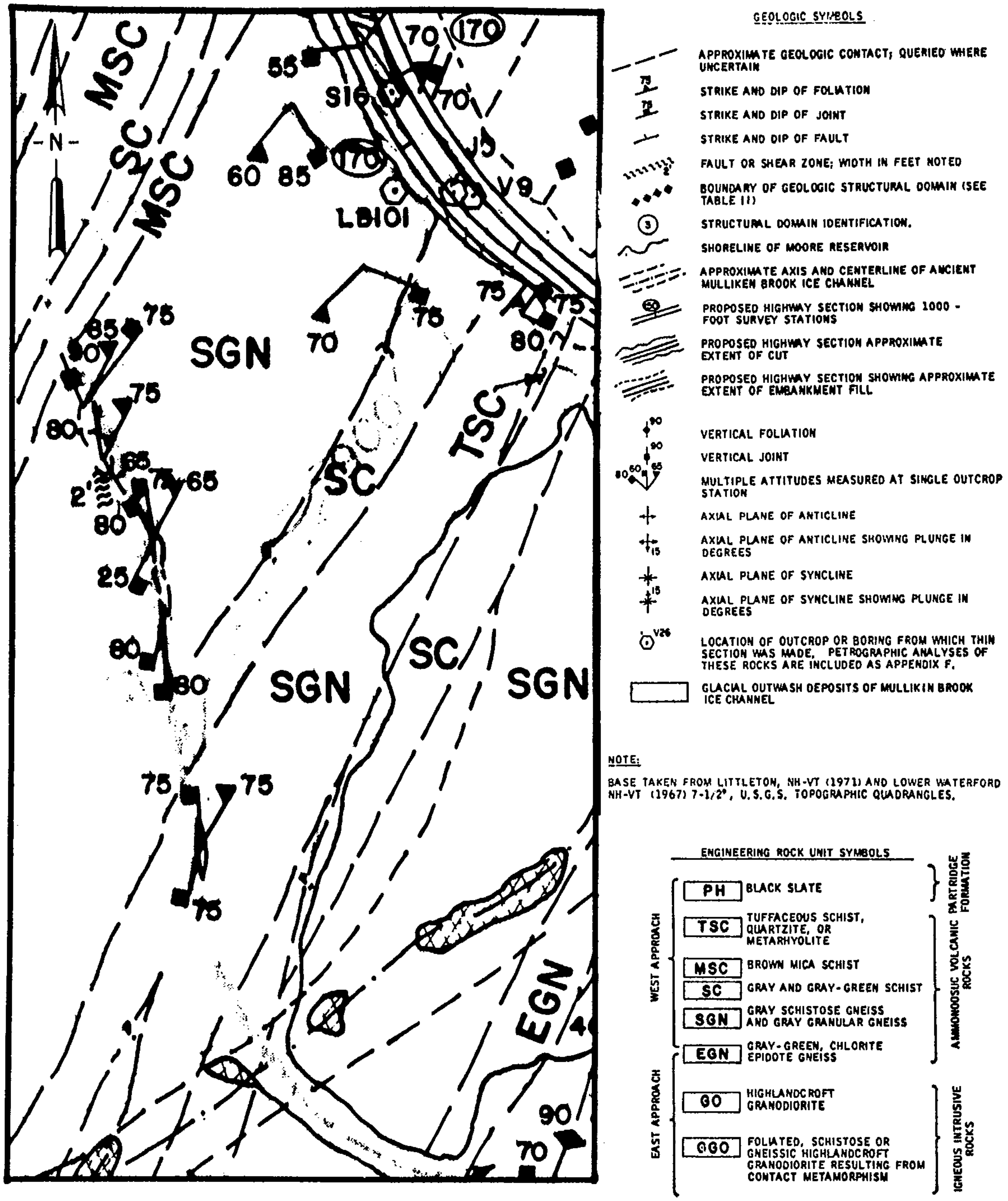


Figure 4-3. A site area geologic map prepared for an interstate highway (Haley & Aldrich, Inc.).

design engineers, owners, and contractors. The purpose of generating tunnel silhouette photographs is to provide a convenient means to study the extent to which the blasting program and geologic features control the excavated cross-sectional geometry. With a specially designed light source, it is also possible to quantify overbreak or underbreak, at given stations, in terms of cross-sectional area. The amount of overbreak or underbreak can be expressed as a percentage of rock over or under-excavated, compared to the design tunnel cross-sectional area.

Silhouette photography as applied to tunnels is not a new concept. Hillan (1955) used one form of silhouette photography in connection with an Australian hydroelectric project. Fellows (1976) constructed a light source patterned after the Hillan work, but increased the light-source intensity. Fellows compared the photographic method with two surveying tech-

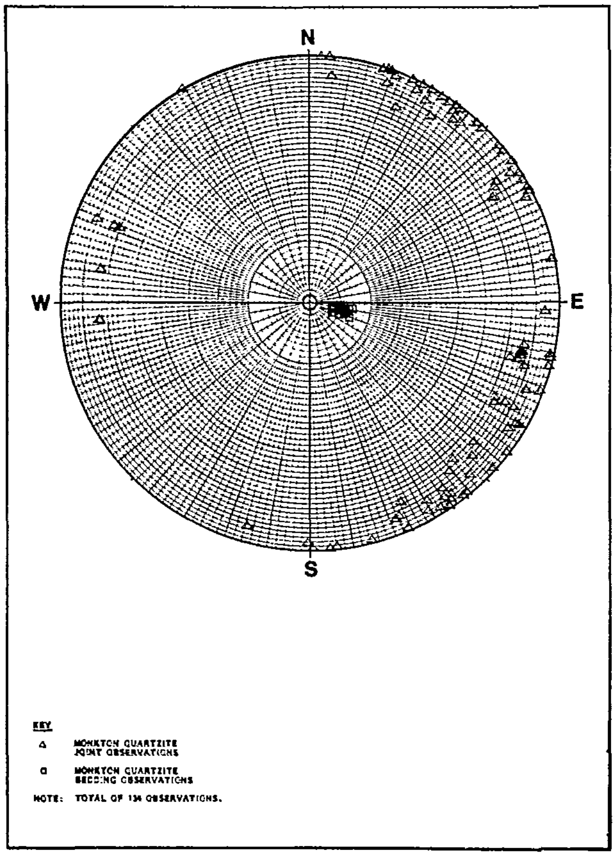


Figure 4-4. Upper hemisphere polar point plot of rock discontinuities.

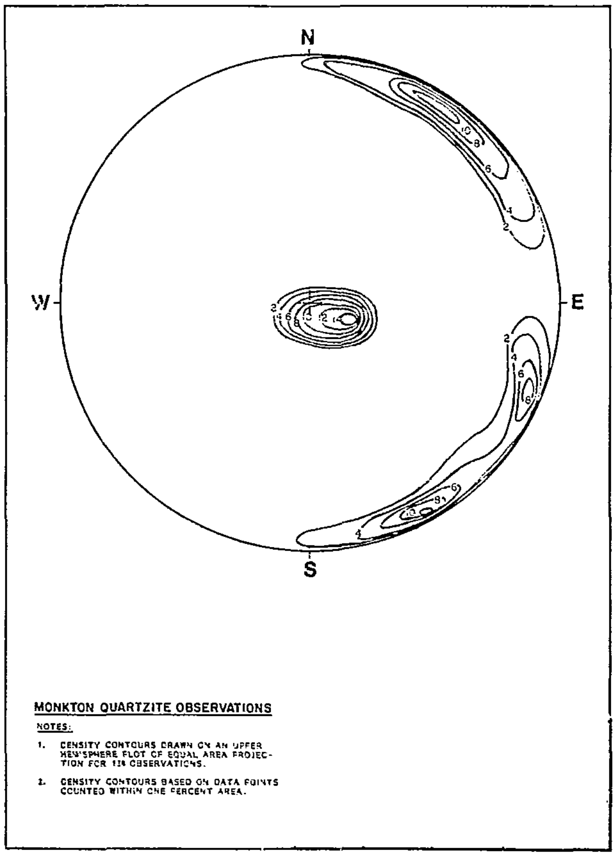


Figure 4-5. Upper hemisphere polar point density plot of rock discontinuities.

niques for measuring tunnel cross sections and concluded that the photographic method was the most accurate and quickest technique. The drawback of the Fellows method is the special photographic and surveying equipment required.

Until only recently, tunnel silhouette photography has not been widely used in the United States. Law Engineering Testing Company employed tunnel silhouette photography qualitatively in 1977 on a rapid-transit pilot (exploratory) tunnel project in Atlanta, Georgia, to evaluate typical tunnel cross-sectional geometry as affected by geologic features. Haley & Aldrich, Inc. (1976) has developed a further-simplified silhouette photographic procedure.

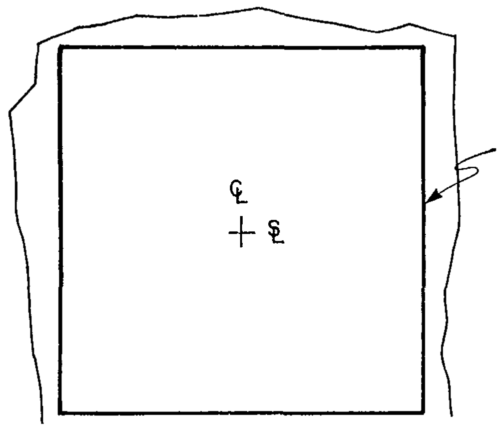
A typical tunnel silhouette photograph made by the Haley & Aldrich procedure is compared herein with the results of profile-survey measurements at an individual station. Survey measurements of tunnel cross section were taken from a square wooden templet. These measurements were obtained with a crew of four persons and took approximately 10 to 15 minutes per station to obtain.

Figure 4-6 has been plotted from survey data at the same scale as silhouette photograph enlargement (Figure 4-6). Survey data are compared directly to the tunnel silhouette photographs. The amount of overbreak above the design invert level at the station shown on Figure 4-6 was then estimated by polar planimeter and agrees with survey data within two percent.

4.4 MATERIALS SURVEYS

Considerable attention is given in highway layout and design to create balanced sections of cut and fill, thereby minimizing the use of imported materials. Design engineers can predict the overall balance of cut and fill, and geologists and geotechnical engineers must evaluate the rock and soil components of the excavated materials inventory as to suitability as construction materials. It is unusual that a particular construction project goes through design without the anticipated requirement for location and qualification of a borrow source of some kind. Materials surveys are the medium of assessment of the borrow sources.

A materials survey should attempt to provide several types of design-related data. The data should be grouped according to similar bodies or geologic deposits which can be identified by landform or structural character.



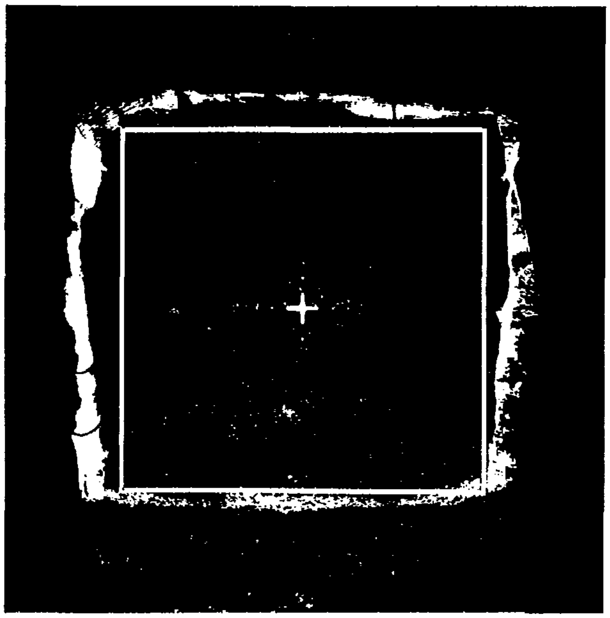


Figure 4-6. Tunnel overbreak photograph and plot used to determine volumes of excavated rock. Agreements of ± two percent have been obtained by use of this technique and the transit-survey method (Haley & Aldrich, Inc.)

Statewide materials surveys are often undertaken with the expectation that suitable sources are scarce in the region and that the DOT may wish to acquire some reserves in anticipation of future requirements. Such states as Arizona, Kansas and Tennessee have undertaken these statewide surveys, and have done so on a basis of a uniform evaluation system. The Tennessee system is illustrative of this type of broad-area assessment effort (personel communication, Dr. R. W. Lounsbury, Memphis State University, November, 1979).

The Tennessee study, under contract to the Division of Soils and Geological Engineering, DOT, made use of representative samples of defined geologic rock units, as exposed in operating quarries throughout the state. The samples and quarries were chosen for geographic coverage and to provide specimens from nearly all of the lithologies of aggregates which have been traditionally used in state highway construction. Due to the fact that the study was statewide, a methodology of analysis was adopted. The Tennessee DOT method is as follows:

- Definition of the stratigraphy of expected samples
- Field examination and sampling
- Petrologic examination (hand specimen)
- Petrographic analysis (thin-section)
- Assign a lithologic and engineering rock name
- Determination of grain size and texture
- X-ray diffraction analysis for clay mineral and other layer silicate discrimination
- Performance of Differential Thermal Analysis; for clay mineral and layer silicate confirmation
- Examination of the insoluable residue.

Large amounts of data often call for computer storage of analysis data and later access for statistical correlation studies.

No system of materials evaluation can offer an absolute qualification of suitability. However, important ranges of properties can be developed so that engineers are aware of the general degree of suitability or unsuitability of certain types of aggregates and to what extent such materials must be tested in order to qualify for consideration on a particular construction project.

4.4.1 County Wide Materials Surveys

An alternative method to the Tennessee DOT spot location of existing materials sources has been applied by many State DOTS. These are generally organized on a county-wide basis and use the statewide, planimetric basemaps fostered by the FHWA. Most of the work is accomplished on a cooperative basis with the Federal Agency and results in single county reports of use not only to transportation agencies but to the aggregate industry. A positive side benefit of the mapping is that there is some stimulus for private development of aggregate sources which may be available at an attractive cost to future transportation projects. The reports are a one-source compendium of geologic resource information for the county. A

Table 4-4. Suggested Outline for County-Wide Materials Inventory

Abstract	Summary of the physiographic, hydrologic, geologic and hydrogeologic features of the county • Drainage and transportation map • Aerial photographic index map
General Geology	County-wide, color geologic map at 1:250,000 to 1:400,000 scale, geologic time scale, stratigraphic column, Quaternary time scale, summarized geologic history and review of geotechnical considerations for construction in the county
Materials Inventory	The main section of the report: • Table of materials and availability • Description of materials by geologic unit, including outcrop stereograms • Tabulated engineering properties • County materials map, by area segments, at 1:31,250 scale, with explanatory legend • Site Data Forms; one per identified existing or potential source.

TYPE Material and Geologic Source	USE	AVAILABILITY
LIMESTONE		
Altamont Limestone Formation	Concrete and bituminous aggregate. Light type surfacing.	Moderate source in eastern part of county.
Hertha Limestone Formation	Concrete and bituminous aggregate. Light type surfacing and riprap.	Good source in eastern part of county.
Swope Limestone Formation	Concrete and bituminous aggregate. Light type surfacing and riprap.	Moderate source in eastern part of county.
Dennis Limestone Formation	Concrete and bituminous aggregate. Light type surfacing and riprap.	Good source in central part of county.
Iola Limestone Formation	Concrete and bituminous aggregate. Light type surfacing and riprap.	Limited source in northwestern part of county.
Plattsburg Limestone Formation	Light type surfacing.	Very limited source along western edge of county.
SAND AND GRAVEL		
Undifferentiated Quaternary Terrace (Nebraskan-Kansan?)	Concrete and bituminous aggregate. Light type surfacing.	Very limited source on higher topography along Neosho River.
Illinoisan Terrace	Light type surfacing.	Very limited source along Neosho River Valley.
Quaternary Alluvium	Concrete and bituminous aggregate. Light type surfacing.	Moderate source in Neosho River Valley.

Figure 4-7. A county-wide materials inventory summary, part of the Kansas DOT statewide materials inventory.

table of contents for a Kansas county Construction Materials Inventory is shown in Table 4-4.

Figure 4-7 is an example of the table of materials and availability of a Kansas Countywide Materials Inventory.

Sampling and testing for materials inventories are usually conducted in accordance with AASHTO and state standards (Kansas DOT, 1973).

4.5 REMOTE SENSING

Remote Sensing is the acquisition of information about an object without physical contact. The normal use of remote sensing usually refers to the gathering and processing of information about the earth's environment, particularly its natural and cultural resources, through the use of photographs and related data acquired from an aircraft or satellite (Colwell, 1983). The aerial data collected by remote sensing systems include *photography* (obtained by a camera), and *imagery* such as satellite, multispectral, infrared and radar (obtained by systems other than a camera).

4.5.1 Types, Availability, Advantages, and Limitations of Aerial Data

A variety of aerial remote sensing data exist. This discussion is limited to those data which have some application to terrain analysis and geotechnical exploration, are readily available, and reasonably economical. These include aerial photography, satellite data, infrared and radar imagery.

4.5.1.1 Aerial Photography. The most useful and available of the remote sensing data is aerial photography. It is available in various film types, formats and scales. The *film types* include: (1) black-and-white (B&W)—panchromatic and infrared; and (2) color—natural and infrared. The most common type used is the B&W panchromatic film. However, both color films have proved valuable for terrain analysis studies and have been used more frequently in recent years. The common photographic *formats* include vertical (camera perpendicular to the ground) and oblique (camera tilted from the vertical). Vertical photography is the predominant format used for interpretation and mapping; obliques are valuable for evaluating valley walls and sidehill slopes. Typical *scales* of photography include: (1) ultra-high altitude (>1:80,000), (2) high altitude (1:40,000–

1:80,000), (3) medium scale (1:20,000–1:40,000, (4) large scale (1:6,000–1:20,000), and (5) very large scale (<1:6,000). All of these scales have been applied in various ways for terrain analysis and geotechnical exploration. A program of special value for terrain analysis is the National High Altitude Photography (NHAP) program under the coordination of the U.S. Geological Survey (USGS). In this program, B&W coverage at 1:80,000 and color infrared at 1:58,000 are obtained for the conterminous United States on a 5 to 6 year cycle. The first cycle started in 1980 and the second in 1985.

Availability. Access and availability of existing photography are excellent in the United States and Canada and in much of the rest of the world. The National Cartographic Information Center (NCIC) of the USGS, maintains an index to all acetate-base film obtained from the mid 1940's in the United States by Federal Agencies, many State Agencies, and some commercial firms. Information on available photography can be obtained from NCIC in the USGS Offices in Reston, Virginia, Rollo, Missouri, Denver, Colorado, Menlo Park, California, and Sioux Falls, South Dakota. Microfiche listings of the holdings for individual states or regions can be obtained at a reasonable cost. The National Archives in Washington, D.C., is the depository of all nitrate-base photography collected during the years 1936–1941. In Canada, information can be obtained from the National Airphoto Library, Canada Department of Energy, Mines and Resources in Ottawa. These resources just provide a listing of the available coverage. To obtain copies of the photographs, one has to contact the organization holding the negatives; this process may take a month or more. For those States or organizations not included in the listing, direct contact has to be made with the organizations to determine their coverage. The price list from NCIC, effective July, 1987, indicates that 9.0″ × 9.0″ B&W prints are $6.00 each, and color prints $16.00 each.

Advantages. Aerial photography provides a three-dimensional view of the terrain showing the conditions existing at the time of photography, and the interrelationships existing between various natural and manmade features. Better results are obtained from analyzing photographs collected several different times over a period of a year, rather than just once. For example, photographs taken in the spring during the wettest time of the year, would provide more information on soil/rock types and presence of high water table and seepage; photographs taken in the fall would indicate tree and vegetation differences that might be related to soil/rock and water conditions. Together, they provide a more complete picture of the terrain conditions.

Limitations. Availability, access, and date of photography may limit its value. It might take a month or longer to obtain existing photography; the photography might be too old and photographic coverage takes long range planning, as much as a year in advance; although general non-mapping photographs can be obtained rapidly by renting a plane and taking pictures with a hand-held camera—weather permitting. In heavily forested regions it may be difficult to interpret terrain conditions because the ground is not visible except in scattered areas.

4.5.1.2 Satellite Imagery. The vast majority of satellite data available includes multispectral (MSS) and video (RBV) imagery obtained in the Landsat program (Short, 1982). Some satellite photography is available from manned Skylab and Gemini programs; however, this coverage is limited and sporadic. The satellite imagery is small scale (~1:1,000,000). Each image covers a ground area of about 115 miles × 115 miles. Only limited stereoscopic coverage is available. Five different Landsat satellites have been in operation since the first one was launched in July 1972. The products available from Landsats 1 and 2 include 4 bands of MSS data (in B&W), color infrared composites from the MSS data, and a limited number of RBV images. Landsat 3 added a fifth thermal infrared MSS band and improved RBV coverage. The resolution of the 4 MSS bands is 79m, and the infrared MSS band 240m. Landsats 4 and 5, in addition to the MSS system, has a 7 band thematic mapper (TM) with bands 1–6 having 30m resolution, and the thermal infrared band 7 having 120m resolution. When only one satellite was in operation, repeat coverage of an area could be obtained every 18 days. When two satellites were in operation simultaneously, repeat coverage could be obtained every 9 days.

Availability. The distribution of Landsat data has been turned over to a commercial firm, Earth Observation Satellite Company (EOSAT). Inquiries about available Landsat data can be made by calling EOSAT at 1-800-367-2801. The information they require is either the latitude and longitude of the area of interest, or the path and row of the images covering the area obtained from the "Index to Landsat Worldwide Reference Systems (WRS)." The WRS index maps are available from NCIC. Other information requested is acceptable image quality (5, fair; 8, good), and maximum acceptable cloud cover (10%, 30%). The cost of a paper print (7.3″ × 7.3″) of a B&W MSS image is $80, B&W TM image $150; a color composite already prepared for an MSS image $150, and a TM image $360. If a color composite has not been previously prepared, then there is an additional fee of $200 to generate an MSS image, and $300 to generate a TM image. All prices are as of July, 1987.

Advantages. Can obtain repetitive, and multispectral coverages of areas of interest. Very useful for regional natural resource studies (e.g., geology, vegetation, landuse); or to note changes that have occurred over a period of time due to natural or manmade factors, temporal changes, or changes due to catastrophic occurrences. Excellent source of recent images of various parts of the world at reasonable costs.

Limitations. Small scale, lack of continuous stereoscopic coverage, and added cost for color composites if images not previously prepared. It also takes longer to obtain these images. Stereoscopic satellite images can be obtained from a recently launched European satellite called "SPOT," but this requires a special order which is expensive.

4.5.1.3 Infrared Imagery. Thermal infrared imagery can be obtained in certain regions or windows of the infrared region within the electromagnetic spectrum; these are at the 3.0–5.0 and 8–14 micrometer wavelengths. Daytime or nighttime imagery can be obtained, but the nighttime imagery has proven more useful for terrain analysis (Rib and Liang, 1978).

Availability. Infrared imagery has been obtained for various research projects by organizations such as NASA, USGS, DOD, and several Universities. The data obtained by NASA and USGS is available through the USGS, EROS Data Center, in Sioux Falls, South Dakota. Thermal infrared imagery can be obtained by contract with commercial firms such as Daedalus Enterprises Inc., Ann Arbor, Michigan, and Teledyne Geotronics, Long Beach, California. Costs for special flights can be expensive and probably only justified for extensive or critical exploration programs.

Advantages. Infrared imagery offers some unique information that can not be obtained directly from the analysis of aerial photography. The combination of aerial photography and infrared imagery provides a more accurate portrayal of terrain conditions than can be obtained from either system alone.

Limitations. Cost, limited availability of infrared systems and data, the necessity of more ground information for interpretation of the imagery, and the need for better weather conditions for flights, are some of the factors limiting the use of this data. The resolution of infrared imagery is not as good as for aerial photography. A knowledge of infrared principles is necessary for the proper interpretation of infrared imagery.

4.5.1.4 Radar Imagery. In contrast to aerial cameras and multispectral and infrared scanners, which are passive sensors that rely on natural illumination or heat emission, radar is an active system that produces microwave radiation to illuminate the surface. Thus, radar is a day-or-night, and virtually all-weather imaging system. The most common type of radar imagery collected is that using a sidelooking airborne radar (SLAR) system, using either K-band or X-band wavelengths, and horizontal (transmit)-horizontal (receive) or "(HH)" polarization. Radar flights can cover large areas very rapidly, each flight strip covering a band about 12 miles wide at scales of 1:250,000 or 1:400,000. Image resolution for the X-band systems is about 10m. It is also possible to obtain stereo radar coverage (USGS, 1985).

Availability. Radar imagery is available from the USGS for selected projects in the conterminous United States and Alaska. Radar strips or mosaics for these project areas can be obtained from the USGS, EROS Data Center, Sioux Falls, South Dakota. A SLAR Microfiche Reference System showing the areas of coverage is available from the EROS Data Center. Paper prints of radar strips are $30 each, 1:250,000 mosaics $85 each, and 1:1,000,000 mosaics $60 each. (All prices are as of July, 1987.) The Goodyear Aeroservice Company in Litchfield Park, Arizona is the repository for radar imagery collected by various DOD Agencies. The availability of strip radar coverage can be ascertained by providing latitude and longitude coordinates. Copies can be obtained at reasonable costs. A limited amount of radar imagery has been obtained from satellite systems. Coverage of the Seasat program and Shuttle Imaging Radar program are shown in the Geologic Applications section of the Manual of Remote Sensing (Williams, pp 1698–1703, 1983). Radar imagery can be obtained by contract with some commercial firms such as Goodyear Aeroservice Co., Litchfield Park, Arizona, and ERIM, Ann Arbor, Michigan. Costs for special flights are expensive and only justified for very large areas of investigation.

Advantages. Radar can be obtained for large areas, day-or-night, in virtually all weather conditions and under a constant illumination. It is especially useful in areas with constant cloud cover where it is difficult or impossible to obtain any other form of aerial data.

Limitations. Small scale, cost, and limited availability of existing data are some of the limiting factors in the use of radar imagery. A knowledge of radar principles is necessary for the proper interpretation of radar imagery.

4.5.2 Uses of Aerial Data

Aerial data provides an aerial overview of the project area that is far superior to an actual overflight, and provides information that might not be easily discernible by ground reconnaissance. Unlike ground reconnaissance, photo interpretation allows for the inspection of the terrain without the obstruction of topographic relief, vegetation, and cultural features. Aerial photography provides the interpreter with a stereoscopic view of the route or site along with the surrounding terrain. Using the standard two-power pocket stereoscope, a useful vertical exaggeration of about 3.5 times, accentuates topographic relief. Variations in topographic expression, along with the elements of drainage and erosional patterns, tone and texture, and vegetation and cultural features form the basis for terrain analysis. An added factor in increasing the accuracy of interpreting aerial data is that of analyzing data collected at different times of the year over the project site. Differences noted at different times of the year over the project site. Differences noted at different times of the year can be related to specific terrain conditions and thus help in the identification of unique terrain features.

High-altitude and ultra-high photography, satellite imagery, and radar imagery have proven useful for the initial stages of terrain analysis. They provide an excellent overview of the terrain and the interrelationships of the natural and cultural features. Regional geologic structure, traces of major faults, lineament delineation, and large regional instabilities are best represented on these data types (e.g., Alfoldi, 1974 was able to delineate landslide susceptible terrain on a satellite image).

After the broad regional terrain analysis is performed on the small scale photography and imagery, medium and large scale photography should be used for planning the geotechnical exploration investigations. Rib (1967) concluded after evaluating various aerial remote sensing data, that the best single system for delineating soils and soil conditions was large scale natural color photography; multispectral imagery provided additional soils/terrain information, but at a greater cost in time and money. Various other investigators have similarly reported the value of color photography for terrain and geotechnical investigations (e.g., Chaves and Schuster, 1964; Mintzer and Bates; 1975, Stallard, 1965). Very large scale photography is not as useful because of the limited coverage per image. It may be difficult to gain an appreciation of the interrelationships of geology and topography as they influence project layout and design.

Aerial photographs of forested regions are undoubtedly the most difficult to interpret. The vegetative cover may become so dense, as in tropical areas, as to preclude direct interpretation except in scattered areas and for certain elements of geology and terrain. For heavily vegetated regions, color infrared is probably the best film type to use. The different color tones associated with different vegetative types and conditions (e.g., hardwoods from softwoods; undergrowth, pasture and grasses from forested areas; water from vegetation) can assist in relating the vegetation types to the underlying soil, rock, and water conditions.

Thermal infrared imagery offers some unique information not directly interpretable on other aerial data. Various temperature and emissivity (efficiency of an object in absorbing and emitting energy) features of the terrain can be related to specific geotechnical conditions. Rib and Liang (1978) have listed some valuable information that thermal infrared imagery provides for landslide investigations: (1) indication of surface and near surface moisture and drainage conditions; (2) indication of the presence of massive bedrock at or near the surface; and (3) distinction between loose colluvial materials and solid bedrock. Tanguay and Chagnon (1972), and J. Buckmeier of Texas Instrument, have reported success in using thermal imagery to locate the high moisture and seepage zones in unstable areas. This aided them in planning exploration programs to stabilize these unstable areas. Thermal infrared imagery has also been used in active volcanic areas such as in Yellowstone National Park to aid in avoiding heated water and soil zones.

4.5.3 Image Interpretation

The entire process of image interpretation is based on the fact that geologic units (rock and soil) possess enough differences in physical characteristics as to represent distinct regimes with respect to roughness of the ground surface, type of vegetation, shape and slope of, and the way in which the unit affects the presence of groundwater and surface water. These aspects of the physical properties of the site affect the *texture, tone, pattern,* and *color* (if color imagery is available) of the image. Additionally, linear features of a non-cultural nature are added indicators of engineering significance. Virtually all natural processes result in landform and vegetational elements which are strictly nonlinear and nonuniform in shape and area extent. Features which take on a linear aspect should be identified and interpreted on the basis of whether or not they represent some form of previous stressing or breakage of the underlying natural materials. Geologists are aware of the fact that even at the microscopic level, linear traces represent mineral fracturing or stress concentrations and their related microdisplacements. Such is also the case at the macro-level of remote imagery.

Tone and texture are rarely quantified in terms of their expression or lack thereof; rather, the photointerpreter notes differences and similarities and places boundary lines (geologic contacts) on the interpretation overlay. These contacts are the interpreter's assessment of the differences and similarities between texture and tone of adjacent areas. When the interpreter adds the third element of landform and the successive elements of vegetation and evidence of surface or groundwater, the type of soil or rock becomes more and more apparent.

Entry-level personnel in the Agency should avail themselves of such excellent references as Ray (1960), Miller and Miller (1961), Lillesand and Kiefer (1979), Way (1978), and Scovel and others (1965), in order to independently develop their skill at image interpretation. Actual transparent overlay interpretations should be made, in which the geologist or geotechnical engineer goes to the point of making a photogeologic interpretation or terrain analysis of the photo.

Some of the essential steps in image interpretations are listed below:

4.5.3.1 Orientation. Arrange the flight lines of overlapping aerial photographs in sequential order of exposure, find north, and begin to locate the center points of the photographs, or *nadirs,* on available topographic maps. Once the average scale of the imagery has been worked out, cut out a cardboard template in the square or rectangular format of the photographs and to the same scale as that of the topographic map base. Mark the coverage of each image on the topographic basemap.

With this accomplished, locate the area of interest to the project and determine which of the images provide the best coverage.

4.5.3.2 Initial Scan of Imagery. Quickly scan the imagery to detect the major aspects of image quality, the general nature of landforms, and the relationships between existing cultural features and the main topographic features. A good way to begin the mapping is to pick an area which represents a type of geology or landform with which the interpreter is most familiar. Often a good place to begin the interpretation is a well defined contact between valley-fill alluvium an surrounding bedrock or glacial-drift-mantled hills.

Continue to scan the photographs gaining an appreciation for the degree of variation in tone and texture representing the various soil and rock units that begin to appear to the interpreter. Complete the scan of all of the photographs making up the primary areal coverage of the route or site; return to area that appeals to the interpreter as being the "best place to start."

4.5.3.3. Compilation of the First Interpretation. Using soft and carefully-sharpened color pencils (Mars Omnichrome) begin marking apparent geologic contacts between soil and/or rock units. If the markings can be made without damage to the emulsion surface of the photographs, it is often best to do so, leaving a photogeologic interpretation on alternate stereoscopic prints. If damage is apparent or markings are not allowed by custodians of the prints, then the use of a light-weight, semi-matte surface, acetate film laid over one print of the stereoscopic pair is appropriate.

Colors are useful indicators of the nature of each element of the interpretation. For example, yellow can be used to outline soil units, green for rock, red for structural features such as discontinuities and folds, blue for groundwater and surface water, and brown for cultural features and manmade fill. Note locations and areas that appear to be very important for field verification.

Within each area of soil or rock unit, place a first-approximation symbol indicating its expected physical nature (rock or soil type) and information dealing with its expected geologic origin, depth and possible underlying material.

4.5.3.4 Assessment of the First Interpretation. Transfer the interpretation to a transparent overlay and have a blueline facsimile made. Finalize the symbols, develop a draft map legend and color the map according to the symbols. The act of coloring a photogeologic or geologic field map often makes errors and discrepancies instantly apparent. It must be remembered that this product is not a true map, but a form map, since the various points traced from the photographs may be at different scales and not in their true map position.

Plan a route of access and traverse across or along the project alignment, such that all critical outcrops or landforms are visited on the first field trip. The colored version of the photogeologic interpretation can be annotated as to priority of each stop and an indication of what will be investigated at the stop. The process of identifying the critical locations and defining the questions to be answered is the best method of planning for effective field mapping.

4.5.3.5 Field Verification. Field verification, should be used to confirm the nature and location of geologic and cultural features identified on photogeologic maps. This mapping is often best accomplished on the site-area route maps, generally about 1:6000 in scale. If the site-area topographic base has been enlarged from existing government topography, it can be printed as a screened reproduction. The base itself will have a reduced visual impact due to the screening. If the photogeologic contacts have been optically transferred (by use of an enlarging-reducing tracing device) from the aerial photographs to the site-area basemap, the photogeologic interpretation can be annotated and revised directly during the field verification. During this field investigation a rudiment of the ensuing field exploration plan can be developed, along with estimates as to the kind of equipment that will be required, the probable production rates for exploration and general routes of access in terrain limited by topographic relief or relatively thick vegetation.

4.5.3.6 Finalization of the Photogeologic Interpretation. The return to the office, the photogeologic form map becomes, essentially, the first project geologic map. This information should be transferred to a topographic base map and drafted, at least on a provisional basis, for use as the main exhibit for project briefings among other geotechnical personnel and with the design and planning engineers. The photogeologic map will next be enhanced by field mapping, probably at a larger scale (1:500 to 1:1,000) during actual field exploration.

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5.0 GEOLOGIC CONSTRAINTS

Transportation systems are subject to a variety of rare, high-impact natural phenomena. Most of these forces have a geologic cause and involve some disruption of the terrain under or around the transportation network. The disruption is often created by dislodgement of masses of earth materials onto or from a roadway cross-section, or by strain-type displacements of roadways and other key structures. Such forces have been termed *geologic hazards*. Recognizing that most of the hazards can be predicted in some way, geologists have recently adopted the term *geologic constraints* to indicate that, when properly identified as to presence or potential, such hazards can be engineered toward minimal impact.

Transportation systems subsurface investigations should identify potential geologic impacts early in the office or field reconnaissance and define their key aspects so that planning and design personnel can provide the engineering response. The engineering solutions may range from minimal-cost measures to consideration of serious economic impacts on the basis of alternate design concepts or rerouting.

5.1 PROVIDING DESIGN-RELATED DATA

Geologic constraints should be considered in terms of the magnitude of strain or disruption to the transportation system represented by the occurrence of each event. Geotechnical engineers and geologists should detect the potential geologic constraint, assess its risk of occurrence, and estimate the magnitude of the impact. Some constraints are continuous and ongoing (such as swelling soil or rock), others are single-event threats (such as unstable soil and potential landslides), others are frequency-dependent and recurring (earthquakes and hydraulic damage).

For constraints that are continuous or single-event in nature, the constraint is considered to represent a 100-percent risk potential. Design should then be based on attainment of a level of assurance that the constraint is compensated for in design.

Constraints that represent frequency-dependent natural events must be countered by measures to divert the forces (e.g., flood waters or snow avalanches) from the system or to strengthen the system so that it will retain acceptable minimal function (e.g., earthquakes) immediately after the event has occurred.

Table 5-1 lists the more common geologic constraints, the general nature of their occurrence, the type of threat that they represent, the basis upon which they are assessed for transportation system design, and the design-level requirements placed on geological and geotechnical personnel in the transportation organization.

Design personnel should be able to define the nature of most threats due to geologic constraints. After definition of the threats, a philosophy of containment or avoidance should be developed on the basis of feasibility and cost. Once the decision is made to plan for containment or avoidance, the site specific design-level recommendations can be developed for either or both choices. Separate subsections of this section discuss the nature of major geologic constraints, and how each can be defined by Agency personnel to a degree sufficient to warrant alignment and design decisions.

5.2 DETECTION OF GEOLOGIC CONSTRAINTS

Geologic constraints may be passive or active, predictable, quasi-predictable, or unpredictable, and of a single-event or recurring nature. Nearly all transportation projects will encounter some form of geologic constraint; those of least impact can result in infrequent roadway hazards and intermittent maintenance costs; the more serious constraints would be capable of causing severe losses in human life and property damage, and could result in relocation of a facility or

Table 5-1.
Identification and Accommodation of Geologic Constraints in Transportation System Design

Constraint	Occurrence	Nature of Threat	Assessment Basis	Design-Level Requirements
SUBSIDENCE	Single-event; Predictable (active mineral extraction) Unpredictable (abandoned mineral extraction activities) Quasi-predictable (growth faults and carbonate dissolution)	Mass displacement; open voids in roadway	Mapping, drilling, geophysics	Depth, geometry and areal extent of mineral extraction activity; devise methods of avoiding or inactivating the process
LANDSLIDES AND OTHER MASS WASTAGE	Single-event; Predictable	Mass displacement of roadway or debris onto roadway	Remote sensing, mapping, drilling, geophysics, instrumentation	Geometry of mass; definition of driving forces; method draining water or reduction of driving forces, relocation if remedial treatment is not cost-effective
UNSTABLE SOIL AND ROCK	Single-event; Predictable	Slow and continuous displacement of roadway and structures; adjacent slope sloughing	Mapping, drilling, petrology, petrography, known detrimental geologic formations	Estimate potential for volumetric change under moisture variation and exposure to elements; define need to remove, drain, isolate, or overcome by structural resistance
FLOODING	Frequency-dependent	Removal or obstruction of roadway or structures	Mapping, hydrologic statistics	Estimation of erosion susceptibility, determination of level or path of flow
SEICHES AND TSUNAMIS	Frequency-dependent; Risk basis	Disruption of traffic; erosion of roadway; debris obstacles to traffic	Assess geometry of roadway; use of hydrologic statistics, empirical seismic relationships	Define risk-related level of runup and area of inundation
TIDAL INUNDATION	Predictable	Inundation of roadway, lodgement of debris	Review hydrographic records, assess site geometry	Estimation of runup area, erosion, susceptibility of soil/rock
VOLCANISM	Quasi-predictable; Frequency-dependent	Blockage of roadway (lava flows, ash falls, mudflows) disruption of drainage upstream; noxious gases	Mapping to include presumed volcanic vent; stratigraphic basis for frequency; seek evidence of each type of roadway blockage unit	Estimate path, volume, depth, rate of advance of each type of roadway blockage unit; consider diversion schemes
AVALANCHES	Predictable within each season; on observation	Disruption of traffic, lodgement of debris, structural damage	Mapping of previous accumulations and flow paths	Estimation of maximum mass volume and rate of flow; most likely flow paths
POLLUTANTS FROM SPILLS	Unpredictable	Incidental to use of the roadway; hazard until cleared; possible long-term envi-	Determine volume and area of spill; position of ground water; fluid	Estimation of volumetric distribution of spill; rate and path of migration;

Table 5-1. (Continued)
Identification and Accommodation of Geologic Constraints in Transportation System Design

Constraint	Occurrence	Nature of Threat	Assessment Basis	Design-Level Requirements
		ronmental impact	transmission properties of substrate	assist in cleanup program design
EROSION	Predictable	Loss of roadway; disruption of roadway drainage facilities	Mapping, drilling, sampling, testing; determine surface water flow paths	Estimation of susceptibility to erosion; devise drainage provisions; recommend soil/rock surface treatment

route. It is generally possible to anticipate geologic constraints, through knowledge of the particular physiographic region or natural problems associated with construction in similar terrain, or from engineering geological predictions associated with the geology of the site or route.

Environmental impact assessments should detect major geologic constraints. These are the constraints which lie largely external to the project and which may be activated by or otherwise affect the project. Examples are major landslides, volcanism, or floods. Other constraints may be equally important but more localized and are activated strictly by the presence of the roadway. Examples of these phenomena are unstable soil and rock material, subsidence, the smaller landslides, and erosion of slopes and embankments produced during construction of the roadway.

Part of the preliminary planning for a route, exclusive of the environmental impact statement, will be a literature search. This review should define the general lithology of rock units, along with previously-noted occurrences of geologic constraints. Maps depicting mineral and groundwater extraction activities are common and well fields, mines, and quarries should be identified early in the study. Table 5-1 may be used as a checklist for possible geologic constraints in the region. In reviewing the constraint categories, geologists should consider the possibility that construction of the transportation system may introduce new physical parameters such as adverse stress distribution and pore water accumulation that have not been present previously. New geologic constraints could be activated; threats which are reasonably predictable, but which have not been previously encountered in the region. However, in almost all cases, if a site or route is evaluated for impact against the key factors for each geologic constraint, the potential for occurrence of individual geologic constraints will emerge, and these factors can be verified or discarded on the basis of field mapping and subsurface explorations.

In addition to the literature review and search for physical factors that may indicate the presence or activation of geologic constraints (such as clay mineral type, low soil density, abandoned coal mines, or frost susceptibility), photogeologic studies can detect many of the constraints which have developed in the past. Examples of these features are sinkholes or karst topography resulting from carbonate dissolution, as well as landslides, shore or riverbank erosion, frost heave patterns, and mudflows. With a limit of resolution of about three feet, for 1:20,000 scale photography, significantly large areas of constraint-affected ground can be detected before entry into the field.

5.3 SUBSIDENCE

The phenomenon of downward sinking of the ground surface in generally bowl-shaped configurations is termed subsidence. Subsidence is induced through the removal of either the pore fluid or solid components of earth materials below the ground surface or, more rarely, by long-term naturally occurring volumetric shrinkage of relatively thick sequences of unconsolidated sedimentary soils.

Subsidence generally affects transportation systems in some way because once underway, the volumetric decrease usually occurs over fairly large areas; as much as 13,000 km^2 in the Houston-Galveston area (Holzer, 1980). Although the lowering of the ground surface is generally at a slow rate, developing stress fields can lead to strain accumulation at depth and the formation or reactivation of fault-like earth fractures. These fractures may open at the ground surface in a matter of minutes, and may be large enough to create obstacles to automobile and truck traffic, deform rail lines, or damage or alter gradients in drainage systems.

Subsidence is usually not anticipated by transportation designers in areas in which it has not previously been noted. However, it may be logically expected to occur in some areas, due to groundwater and mineral extraction activities. The potential for and character of subsidence may be generally predicted on the basis of certain factors:

- Relatively thick sequences of younger, unconsolidated or semi-consolidated sedimentary materials
- Irregular bedrock topography jutting upward into the sedimentary sequence
- Heavy ground water withdrawal for irrigation or water supply
- New or on-going petroleum extraction activities
- Active or abandoned mines
- Highly variable aquifer and aquitard stratigraphy in the sedimentary section
- Near surface, soluble rock formations, (for example: limestone) in which groundwater has flowed or is currently flowing

Some guidelines relating to mechanisms causing subsidence are presented in the following sections. They may be used as indicators for the detection of potential subsidence problems along transportation system alignments.

5.3.1 Fluid-Withdrawal Effect

Three-dimensional volumetric shrinkage of soil or otherwise unconsolidated sediments is termed *consolidation* in the engineering context of this Manual. Relatively loose cohesionless soils and relatively soft cohesive soils can contain a significant percentage of void spaces which, below the water table are filled with pore water. These soils are capable of losing significant percentages of their pore water under imposed structural loads or from drainage or well pumping. As the pore water drains or is forced out of the matrix voids, the soil mass loses a portion of its volume as individual grains adjust positions and move together.

Subsidence is generally not considered to result from foundation loading; this type of volumetric shrinkage is routinely analyzed as consolidation under load, with resulting settlement. However, fluid withdrawal activities also create volumetric shrinkage, but in these instances the volume change is centered within the stratum from which fluid is being withdrawn by oil, gas, or water wells. The consolidation begins at the well points and extends out into the stratum, appearing at the ground surface as subsidence.

Subsidence due to petroleum pumping in the Long Beach, California area became observable in the 1930's at which time the U.S. Navy undertook measures to build sea retention dikes at its Naval Shipyard. This subsidence later developed into a semi-elliptical bowl and the maximum subsidence has now (1980) reached more than 13 m. (40 ft.) below the original ground surface. Hydrogeologists of the U.S. Geological Survey, in following this evidence of subsidence due to fluid withdrawal, began to detect significant amounts of subsidence in the San Joaquin Valley, here due to newly developed deep-draw water pumps supplying the developing agricultural industry.

At the present time, the U.S. Geological Survey continues to define the mechanisms of the various types of subsidence that are currently active in the Southwest and Rocky Mountain states. Holzer (1980) has reported on these on-going studies which have defined more than 22,000 km^2 (8,500 mi^2) of land that are affected by widespread fluid-withdrawal induced subsidence (Fig. 5-1).

The Houston-Galveston area of Texas has been known to be the seat of water-withdrawal subsidence since the 1950's. In this area alone, more than 150 active subsidence faults have been mapped, with an aggregate length of more than 500 km. (305 mi.). Damage to roadways and structures is slow and ongoing, but the dip-slip nature of these activated faults creates vertical displacements of up to 1 m. (39 in.). As of 1978 (Kreitler and McKalips) displacements now traverse two airports, eleven Interstate highway locations and railroad lines at twenty-eight places. Extensive drilling and careful borehole geophysical logging are used by consulting firms and agency per-

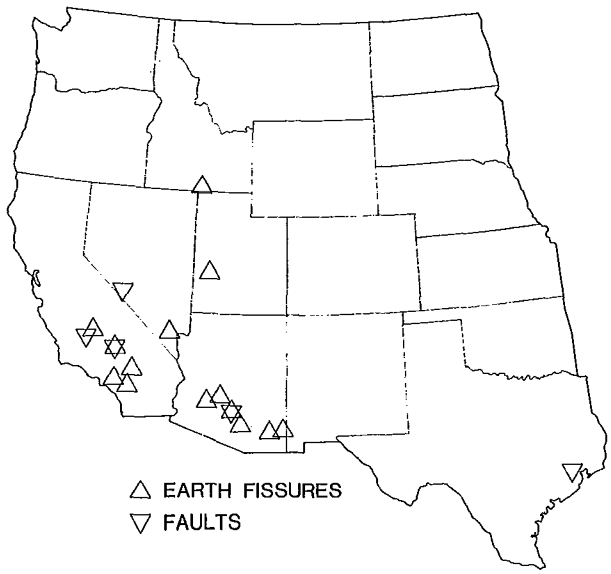

Figure 5-1. Locations in the U.S. of ground failure associated with groundwater withdrawal (Holzer, T.L., 1980).

sonnel to trace the position and extent of the faults. Kreitler (1976) and Kreitler and McKalips (1978) list five aspects of site-specific screening in the Houston-Galveston area:

- Evident topographic scarps
- Borehole geophysical logging
- Exploratory trenching
- Remote imagery lineations
- Horizontal electrical resistivity profiling

Many of the subsidence faults may be non-tectonic growth faults incapable of creating earthquakes (Section 5.3.4.).

Interstate 10, west of Tucson, Arizona, has been plagued by creep-type vertical displacements in the roadbed, since its construction in the early 1960's. U.S. Geological Survey personnel (Holzer, 1980) have mapped such fractures in more than 3000 km^2 (1150 mi^2) in two areas of southeast and south-central Arizona. Fractures, such as that shown in Figure 5-2, have been measured open to depths of 10 to 25 m. (30 to 80 ft.) deep and erosion-enlarged to a meter or more in width.

The mechanism by which subsidence-related fractures occur and open is believed to be largely shear and tension. Where near-vertical pre-existing faults occur in the area of fluid withdrawal, earlier displacements often create partial ground water barriers by juxtaposing aquifer against aquitard, thus limiting withdrawal of water to the aquifer side of the fault. As the aquifer adjusts to pumpage and consolidates, a stress differential builds up at the fault interface and eventually results in downward displacement along the side facing the aquifer. Some of the shear displacement is carried upward, and may be evident at the ground surface.

5.3.2 Mining-Induced Subsidence

Mining activities produce voids equal to the volume of extracted ore. Many mines are designed to remain open for the life of the facility, others are designed to remain open only in segments of the mine that are required as haulways and access and ventilation shafts. Portions of the mine openings left abandoned or otherwise not permanently supported, will fail with time. In bedded formations, such as those in which coal is mined, the failure comes with progressive deterioration of the support pillars of unmined coal, or with deterioration of intervening roof spans.

The extent to which the effect of failing roof strata

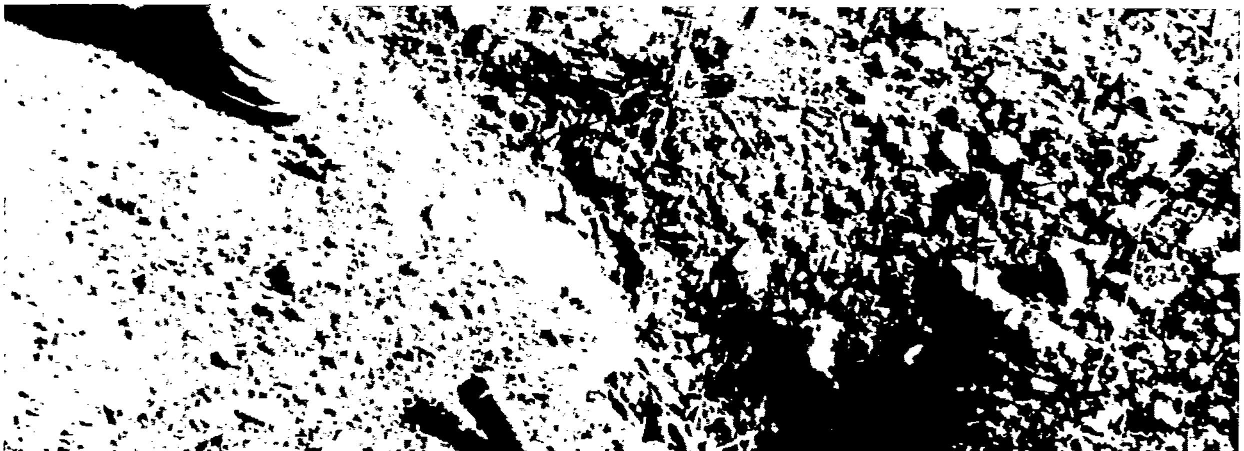

Figure 5-2. Incipient fissure enlarged by erosion; note geologist's pick for scale. (A.W. Hatheway)

or support pillars extends upward is mainly a function of the depth at which the mine has been developed. As pillar and roof rock fail, there is volumetric bulking. The void space is progressively filled by the expanding volume of rock rubble falling into the upward-moving void. Upward propagation of broken rock may cease short of breaking the ground surface, in which case ground surface subsidence may not occur.

Modern coal mining practice and Government mining regulations now call for careful planning for mine development, recording of the geometry of the mine workings, and measures to mitigate the tendency to create subsidence effects at the ground surface. New mines will probably not present threats to transportation system design and maintenance. Most of the ongoing mine subsidence damage, however, stems from the near-surface (generally less than 30 m. [100 ft.]) presence of abandoned mines, most of which were developed, mined and closed without records. They are also commonly the source of acid drainage waters that pollute streams in the area. State agencies have been involved in the past in monitoring coal mining. Transportation agency personnel can secure maps of coal mining regions that will outline the general extent and nature of the producing members of coal-bearing formations. Under the provisions of the Rural Abandoned Mine Program, county offices of the U.S.D.A. Soil Conservation Service may also be able to provide information relating to the existence of abandoned mines or waste dumps that might impact transportation system design.

The abandoned mines represent a subsidence potential in the form of surface fracturing of roadways, development of pits and sinkholes in roadways and undermining of embankments and sidehill fills. Detection of subsidence-prone areas should consider the use of coal mining district maps, geologic maps depicting the outcrop patterns of known coal-producing strata, remote-imagery interpretation of obvious surficial features indicative of subsidence, careful alignment mapping to detect mine openings, acid mine drainage, spoil piles, abandoned workings, and the selective use of geophysical techniques, such as gravimetric surveys.

Underground mining of salt by selective extraction of salt beds also produces upward-propagating failure of individual beds. Salt deposits of this type are found in the Province of Ontario and in Michigan. The surface outcrop pattern of subsidence is influenced by the geometry and age of development of the underground workings. As in coal mining, subsidence is unintentional and unwanted by the mining companies. Operations developed after about 1955 have generally considered this factor in the design of the workings and in the order of development of the mine. Areas known to have been worked previous to this time should be considered to be a potential risk for subsidence-related ground deformation.

In the copper mining industry, subsidence induced through the block-caving technique is an accepted facet of operation. Rock mechanics experts in the mining industry are not yet able to accurately predict the geometry of surface subsidence effects in block caving, but estimates should be available. At San Manual Copper Mine, about 80 km. (50 mi.) north of Tucson, Arizona, the positive economic state-wide impact of development of the mine, starting in 1955, led to the decision to abandon a state secondary road that originally crossed the present subsidence pit area.

5.3.3 Sinkholes

One of the major geologic constraints that is found throughout certain portions of the world is the presence and continued enlargement of subsurface voids in rocks (primarily limestones) that are subject to dissolution by the passage of moving groundwater. As dissolution continues with time, and individual cavities grow and coalesce, gravity-induced collapse often appears at the ground surface in the form of roughly circular, closed depressions. These depressions are most commonly called sinkholes, but they are known also by a variety of other terms with geographic, lithologic, or generic implications. The general term for the landform that results from dissolution is karst. Transportation structures have been adversely affected in the past by sinkholes which have enlarged and collapsed beneath roadways and structural foundations. Remedial measures include cleaning and infilling, or injection of stabilizing filler. A major review of the conditions of formation of sinkholes under all geologic and climatic conditions has been completed (Franklin and others, 1980).

The main minerals that are involved with dissolution are dolomite ($CaMg(CO_3)_2$), gypsum ($CaSO_4$), calcite ($CaCO_3$) and aragonite ($CaCO_3$), in order of increasing solubility. Sinkholes should be anticipated in any carbonate rock terrain in which groundwater lies close enough to the surface to produce cavities, the collapse of which would eventually reach the ground surface. Also of concern are cavities which may be large enough or could grow large enough to cause over-stressing under loads imposed by overlying transportation facilities.

Franklin and co-workers note that mean annual precipitation in the range of 81 to 142 cm (32 to 56 in.) has been noted to produce karst terrain in the United States. W. E. Davies (1970) has mapped the occurrence of rock units susceptible to sinkholes and related features of sinkholes. Inspection of topographic mapping and aerial photographs will assist in confirming the presence of sinkholes. Drilling conducted in carbonate rock should be done with careful observation of rod drops and poor core recovery as indicators of possible cavities, buried sinkholes, and dissolution enlarged joints and bedding planes in rock that may be later subjected to structural loads.

5.3.4 Growth Faults

During years of geophysical reflection surveying for petroleum exploration, in the Gulf Coastal states of the United States, geophysicists and geologists have detected unusual fault-like geophysical traces. These features were identifiable as stratigraphic displacements in the thick Tertiary-aged sedimentary sequence, but the throw (vertical displacement) became greater with depth. The usual appearance of steeply-dipping normal or thrust faults is that the displacement is constant over the trace of the fault, as viewed along strike and perpendicular to dip. Many of the traces were noted to approach the ground surface, but geophysical interpreters were not directly concerned about surface structural damage or possible seismic activity. Exploration drilling in the 1950s began to detect the upward-increasing stratigraphic displacements along these *growth faults*.

With the advent of nuclear power plant siting in the Gulf area and with increasing urbanization of these states, civil and structural engineers detected a growing occurrence of fractures in curbs, road surfaces, individual homes, and in a wide variety of engineered structures. According to Kreitler (1976) the growth faults had damaged more than 200 residences in eleven communities in Harris and Galveston counties, Texas, and are most observable on road and highway surfaces. Cracking of pavement and structural components with offsets of as much as 6 cm (2.5 in.) have been detected in Baton Rouge. Many of the fractures actually represent regional subsidence due to groundwater withdrawal. However, many others have occurred in areas in which groundwater-induced subsidence has not been observed or has not occurred to a significant degree. Unlike groundwater-induced ground fractures, which are mainly curvilinear in trace and oriented around centers of groundwater pumping, these ground fractures appeared to be predominantly linear. The damage has been widespread in Texas and in other states underlain by thick accumulations of Tertiary-aged sediments.

Kreitler (1976) gives four recognition criteria for growth faults intersecting the ground surface:

- Topographic scarps (vertical displacements of a few inches per decade have been observed)
- Geophysical traces (mainly from resistivity surveys)
- Intersection of faults through coring or trenching,
- Remote-image lineations

5.4 SLOPE MOVEMENTS

Slope movements represent a variety of natural and man-induced processes which result in gravitational displacements of very small to extremely large bodies of soil and rock. Transportation resources are continually taxed by the impacts of slope movements on roadways and critical support structures. Such earth and rock failures have been traditionally grouped together under the general term *landslides,* but their wide variety and frequent disassociation with sliding as a mode of displacement has led Varnes (1978) to propose the overall use of *slope movements* as a unifying descriptor. Some aspects of slope movements are illustrated in Figures 5-3 through 5-4.

The literature of slope movement processes is enormous. Collection of a large number of references will do little to help the engineer or geologist to discover and treat slope movement hazards. Rather, it is the familiarization with slope movement terminology, causative processes and indicators of existing or potential slope movements that is essential. However, acquisition of the following basic references is recommended; Schuster and Krizek, eds., TRB Special Report 176, 1978, Eckel, E.B., ed., 1958; (out of print); Coates, 1981; and Hoek, E., and Bray, J., 1976. Other appropriate references are given in the reference list for this manual.

Slope movements in the United States accounted for expenditures of $50,000,000 in 1973 (Fig. 5-5), mainly for remedial treatments (Fleming, Varnes and Schuster, 1979).

5.4.1 Classification of Slope Movements

Varnes (in Schuster and Krizek, 1978) has produced a comprehensive treatment of slope movement processes. An understanding of these processes is essential to recognition of slope movement hazards in transportation system planning and design. The use of the Varnes classification is recommended because it is very comprehensive and represents a terminology base that can be understood by design personnel.

The emphasis on slope movement awareness in transportation agencies should be placed on recognition of existing potentially unstable slope masses

Figure 5-3. Rock slide along adversely-dipping (angled out of the cut slope) bedding on an Interstate highway, during construction; note the particular planarity and relative smoothness of the bedding planes. (A.W. Hatheway)

which could later impact the function of transportation systems or which may be triggered or activated by construction, operation and maintenance of systems.

Varnes (1978) bases his classification on subdivisions of material type (rock vs. soil) and type of movement. The following types of movement are distinguished:

- *Falls and topples:* gravitational downward movement without shear displacement
- *Slides and spreads:* gravitational downward movement of rather intact bodies of soil or rock over an undisplaced lower boundary surface
- *Flows:* gravitational downward movement of nonintact masses of soil or rock over an undisplaced lower boundary surface
- *Complex movements:* combinations of the above movement types and their subtypes

5.4.2 Detection of Movement-Prone Areas

Most existing slope movements are near other similar examples, all clearly related to natural processes, cli-

Figure 5-4. A debris avalanche along California Highway 39 in the Angeles National Forest, San Gabriel Mountains, northeast of Los Angeles. (A.W. Hatheway)

mate, vegetation, hydrogeological conditions, and the engineering properties of various geologic formations and soil units. The single most helpful tool in detection is a review of local and regional physiographic characteristics with other workers in the area.

Second to the regional association of slope movements is recognition of their geomorphic indicators (Table 5-2). Slope movements, by virtue of their displaced earth and rock masses, always result in some degree of disruption of landform morphology; most notably in the production of scarps, swales, undulations, ground cracking, vegetational stress, slope seepage and disruption of natural drainage features. Where slope movements result in strain imposition on highways, rail lines, bridges, tunnels, or retaining walls, these man-made structures may bear evidence of rapid or creep-type deformation brought about by moving slope masses.

The detection process is usually aided by experienced photogeologic interpretation of air photos. Further, identification of regionally important areas of slope movement can lead to procurement of available aerial photographic coverage of such areas, for individual study. Photo interpretation practice strengthens the pattern-recognition ability for detec-

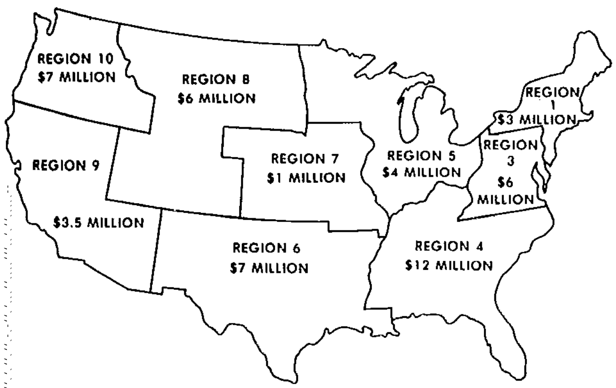

Figure 5-5. Costs associated with repair of major slope movements along Federal-aid highways in the United States, for 1973 (From Chassie, R.G. and Goughnour, R.D., 1976).

tion of areas which may now or may have previously undergone slope movement.

5.4.3 Geometry of Moving Slope Masses

An essential follow-up to recognition of potentially unstable slope masses, is the determination of their geometry. Without understanding of the actual or probable limits of such masses, it is difficult to advise planners in route or structure selection. However, even obvious slope movement masses affecting transportation routes may be quite difficult to define. Often, topographic mapping is necessary to achieve an accurate basemap on which to map morphological features and plot field observations from survey traverses of creep-monitoring stations, locations of boreholes and borehole instrumentation, seepage features and drainage measures. In this connection, orthophotographic, contoured basemaps often prove to be both inexpensive and available on short notice.

Determination of the subsurface geometry is extremely difficult, especially in cases in which the lower failure surface is actually transitional over tens of centimeters. This usually requires continuously-sampled boreholes and installation and monitoring of borehole inclination devices, which must be placed so as to intersect the failure surface. Obviously, most geometric detection efforts are undertaken in those instances in which the function of a roadway or structure has been impaired by a slope movement. By far the best product that can come from agency geological and geotechnical personnel is detection of potentially unstable masses early in the pre-design stage, so that mass movement impacts can be avoided through choice of alternate locations or alignments.

A series of examples of slope mass geometry for the *slump* sub-category of slide are shown as Figure 5-6 (Varnes, 1978). Most often, the failure surface will be complex in some fashion and not strictly circular, as shown. An example of surface morphology is shown as Figure 5-7, with a typical cross-sectional view shown in Figure 5-8.

Surficial details of moving slope masses can be accurately mapped by pace and compass or by reference to map contours, utilizing symbols such as are shown in Section 4.

5.4.4 Causes of Slope Movement

Some natural slopes are found in such geometry and slope angle as to be at a state of near failure. This is usually when the mass properties are marginally able to resist external and internal forces acting on the slope mass. This is especially true for slopes which are old enough to have been sculpted to their present form under climatic conditions and by natural processes which are no longer present. As Varnes (1970) reminds us, such slope masses may be triggered into creep or rapid movement by subtle or sudden changes in the natural environment surrounding the slope. All too often such changes are the result of man's activities: changing slope geometry, alteration of surface drainage or the internal groundwater regime, or simply logging or clearing of the slope.

Some of the specific causes of activated or continued slope movements are:

- Removal of lateral support at the toe by construction cut or on-going erosion
- Surcharge at the crown, by construction of structures, fills, roads, ponded or stored fluids, and material stockpiles
- Increase in pore-water content or pressure, especially by ponding or channeling surface water on or into ground fractures or otherwise porous zones of the slope mass
- Subjecting the slope mass to transitory stresses; seismic, blast, or machine vibrations

5.4.5 Data Requirements for Analysis and Treatment

Slope movement masses are generally analyzed by the various slip-circle methods (Fig. 5-6) of geotechnical engineering or by kinetic distribution techniques of geological engineering (Goodman, 1976; Hoek and Bray, 1981; Coates, 1981). Although the techniques of analyses lie outside of the scope of this manual, field exploration personnel should be aware of the analysis techniques so that they may properly map and sample slope movement masses.

Table 5-2.
Landform Types and Susceptibility to Slope Movements
(From Rib and Liang, 1978)

Topography	Landform or Geologic Materials	Landslide Potential[a]
I. Level terrain		3
A. Not elevated	Floodplain	
B. Elevated		2
1. Uniform tones	Terrace, lake bed	
2. Surface irregularities, sharp cliff	Basaltic plateau	1
3. Interbedded-porous over impervious layers	Lake bed, coastal plain, sedimentary plateau	1
II. Hilly terrain		3
A. Surface drainage not well integrated		
1. Disconnected drainage	Limestone	
2. Deranged drainage, overlapping hills, associated with lakes and swamps (glaciated areas only)	Moraine	2
B. Surface drainage well integrated		1
1. Parallel ridges		
a. Parallel drainage, dark tones	Basaltic hills	
b. Trellis drainage, ridge-and-valley topography, banded hills	Tilted sedimentary rocks	1
c. Pinnate drainage, vertical-sided gullies	Loess	2
2. Branching ridges, hilltops at common elevation		2
a. Pinnate drainage, vertical-sided gullies	Loess	
b. Dendritic drainage		2
(1) Banding on slope	Flat-lying sedimentary rocks	
(2) No banding on slope	Clay shale	1
(a) Moderately to highly dissected ridges, uniform slopes		
(b) Low ridges, associated with coastal features	Dissected coastal plain	1
(c) Winding ridges connecting conical hills, sparse vegetation	Serpentinite	1
3. Random ridges or hills	Clay shale	1
a. Dendritic drainage		
(1) Low, rounded hills, meandering streams		
(2) Winding ridges connecting conical hills, sparse vegetation	Serpentinite	1
(3) Massive, uniform, rounded to A-shaped hills	Granite	2
(4) Bumpy topography (glaciated areas only)	Moraine	2
III. Level to hilly, transitional terrain	Talus, colluvium	1
A. Steep slopes		
B. Moderate to flat slopes	Fan, delta	3
C. Hummocky slopes with scarp at head	Old slide	1

Note: This table updates Table 2 in the book on landslides published in 1958 by the Highway Research Board (Special Report 29, p. 91).
[a]1 = susceptible to landslides; 2 = susceptible to landslides under certain conditions; and 3 = not susceptible to landslides except in vulnerable locations.

Information which should be obtained during field mapping and analysis of slope movement masses should include:

- Topography of the slope mass (contoured at 0.5 to 2m intervals)
- All deformational features (scarps, fissures, cracks, etc.)
- Surface water and drainage conditions
- Probable subsurface slip surface
- Locations of all explorations, samples, instruments, survey traverses and monuments
- Longitudinal and lateral subsurface profiles
- Structural attitudes from adjacent rock, if possible
- Definition of major geologic/soil units involved in slope mass
- Description of rock/soil lithologies, mineralization, and weathering
- Description of groundwater regime in and adjacent to the slope mass

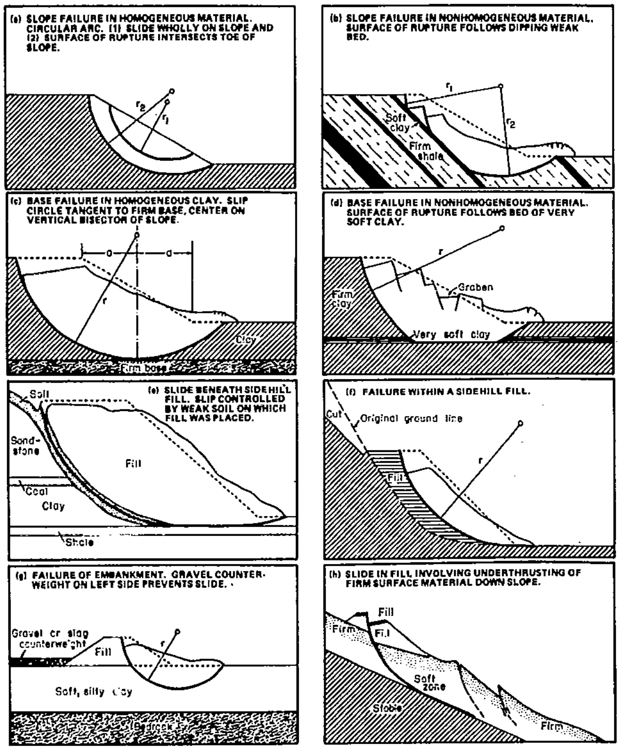

Figure 5-6. Varieties of slump-type failure that are common in soil units and in soft, weathered, or altered rock. (From Varnes, 1978).

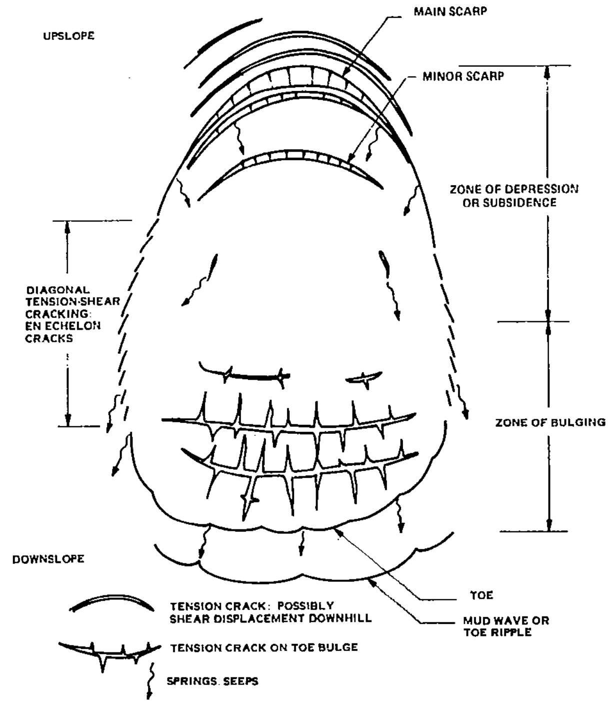

Figure 5-7. Diagram showing the elements of deformation of a slope movement mass. Most site reconnaissance maps of slope failures will show these elements in some form. (From Sowers and Royster, 1978)

5.5 UNSTABLE SOIL AND ROCK

Each year losses from deterioration of natural soil and rock foundation materials for various structures, including transportation systems, exceeds $5 billion in the United States alone. Damage from expansive soil subgrade material in 36 of the 50 states (Lamb and Hanna, 1973) results in more than $1.1 billion in damages per year (Jones and Holtz, 1973). This deterioration is caused by physical and chemical changes created by moisture changes and ion exchange. The deterioration results in one of three basic types of change in the earth material:

- Relatively rapid collapse of the natural soil/rock fabric, with significant decrease in volume; hence loss of structural support
- Less rapid volumetric swelling (expansion) of the earth material, leading to over-stressing of structures or structural members in contact with the affected soil or soft rock
- Slow slaking or disintegration of soft sedimentary rock involved in cut slopes, compacted embankments or load-bearing surfaces

As in the case with several other geologic constraints, unstable soil or rock can be detected or anticipated in most transportation system subsurface investigations. The key to anticipation is empirical data relating to the behavior of individual geologic formations or surficial soil units. Nearly all unstable rock is of the sedimentary variety and is generally represented by mineral types and size gradations reflecting

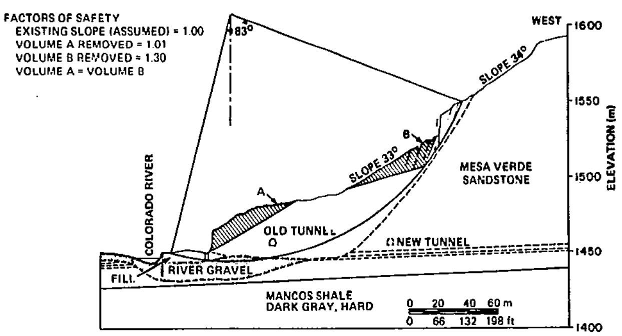

Figure 5-8. Cross-section of a typical semi-circular slope failure in jointed sandstone (Mesa Verde Formation) overlying less-competent shale (Mancos Formation) and affected by active river erosion. (Gedney and Weber, 1978)

its mode of deposition. To the degree that individual geologic formations or subunits of the geologic literature, such as members or named beds, can be identified, so can the mineralogy and depositionally-related fabric characteristics bed anticipated. An example of this is the geologic formational list of potential problem units complied by Snethen, and others (1975); certain units usually demonstrate a potential for expansion when under the effect of increased pore water (Table 5-3).

Table 5-3.
Tabulation of Potentially Expansive Materials in the United States

Physiographic Province No.	Physiographic Province Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map** Category	Remarks
1	Western Mountains of the Pacific Coast Range	Reefridge	Miocene	CA	1	The Tertiary section generally consists of interbedded sandstone, shale, chert, and volcanics
		Monterey	Miocene	CA	1	
		Rincon	Miocene	CA	1	
		Tembler	Miocene	CA	1	
		Tyee	Eocene	OR	3	
		Umpqua	Paleocene-Eocene	OR	3	
		Puget Gp	Miocene	WA	3	Interbedded sandstones and shales with some coal seams
2	Sierra Cascade	Cascade Gp	Pliocene	OR	4	Predominate material is volcanic
		Columbia Gp	Miocene	WA	4	
		Volcanics	Paleozoic to Cenozoic	NV	4	
		Volcanics	Paleozoic to Cenozoic	CA	4	Interbedded sandstones and shales may occur throughout, particularly in western foot hills
3	Pacific Trough	Troutdale	Pliocene	WA	3	Great Valley materials characterized by local areas of low-swell potential derived from bordering mountains. Some scattered deposits of bentonite
		Santa Clara	Pleistocene	CA	3	
		Riverbank	Pleistocene	CA	3	
4	Columbia Plateau	Volcanics	Cenozoic	WA, OR, ID, NV	4	Some scattered bentonites and tuffs
5	Basin and Range	Valley fill materials	Pleistocene	OR, CA, NV, UT, AZ, NM, TX	3	Playa deposits may exhibit limited swell potential. Some scattered bentonites and tuffs
		Volcanics	Tertiary	OR, CA, NV, UT, AZ, NM, TX	3	
6	Colorado Plateau	Greenriver	Eocene	CO, UT, NM	3	Interbedded sandstones and shales
		Wasatch	Eocene	CO, UT, NM	3	
		Kirkland shale	Upper Cretaceous	CO, UT, NM, AZ	2	
		Lewis shale			2	
		Mancos	Upper	CO, UT, NM,	1	

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Table 5-3. (Continued)
Tabulation of Potentially Expansive Materials in the United States

Physiographic Province No.	Physiographic Province Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map** Category	Remarks
		Mowry	Cretaceous	AZ	1	Interbedded
		Dakota	Upper	CO, UT, NM,	3	sandstones and
		Chinle	Cretaceous	AZ	1	shales
			Upper	CO, UT, NM,		
			Cretaceous	AZ		
			Jurassic-	CO, UT, NM,		
			Cretacious	AZ		
			Triassic	NM, AZ		
7	Northern Rocky	Montana Gp	Cretaceous	MT	1	Locally some
	Mountains					sandstone and
		Colorado Gp	Cretaceous	MT	1	siltstone
		Morrison	Jurassic	MT	3	Locally some sil-
		Sawtooth	Jurassic	MT	3	tstone
						Shales, sand-
						stones, and
						limestones
8	Middle Rocky	Windriver	Eocene	WY, MT	3	
	Mountains	Fort Union	Eocene	WY, MT	3	
		Lance	Cretaceous	WY, MT	1	
		Montana Gp	Cretaceous	WY, MT	1	
		Colorado Gp	Cretaceous	WY, MT	1	
		Morrison	Jurassic-	WY, MT	3	
			Cretaceous			
9	Southern Rocky	Metamorphic	Precambrian	WY	4	Montana and
	Mountains	granitic rocks			4	Colorado Gps
		Metamorphic	Precambrian	CO	4	may be present
		granitic rocks				locally with
		Metamorphic	Precambrian to	NM		some Tertiary
		granitic rocks	Cenozoic			volcanic and
						minor amounts
						of Pennsylvania
						limestone
						(sandy or
						shaly). Some
						mixtures of
						metamorphic
						rocks with
						sands and
						gravels of Poi-
						son Canyon fm
10	Great Plains	Lance	Pliocene	WY	1	
		Fort Union	Pliocene	WY, MT	2	
		Thermopolis	Pliocene	WY, MT	1	
		Montana Gp	Cretaceous	WY, MT, CO,	1	
		Colorado Gp	Cretaceous	NM	1	
		Mowry	Cretaceous	WY, MT, CO,	1	
		Morrison	Cretaceous	NM	3	
		Ogallala	Pliocene	WY, MT, CO,	3	
		Wasatch	Eocene	NM	3	Generally nonex-
		Dockum	Triassic	WY, MT, CO,	3	pansive but be-
		Permian Red	Permian	NM	3	ntonite layers
		Beds				are locally pre-
		Virgillian Series	Pennsylvanian	WY, MT, CO,	3	sent
		Missourian Series	Pennsylvanian	NM, SD, NE,	3	
		Desmonian Series	Pennsylvanian	KS, OK, TX	3	
				MT, SD		
				CO, NM, TX		
				KS, OK, TX		

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Table 5-3. (Continued)
Tabulation of Potentially Expansive Materials in the United States

Physiographic Province No.	Physiographic Province Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map** Category	Remarks
				NE, KS, OK, TX, MO		
				KS, OK, TX, MO		
				KS, OK, TX, MO		
11	Central and Eastern Lowlands	Glacial lake deposits	Pleistocene	ND, SD, MN, IL, IN, OH, MI, NY, VT, MA, NE, IA, KS, MO, WI	3	Some Paleozoic shales locally present which may exhibit low swell
12	Laurentian Uplands	Keweenswan	Cambrian	NY, WI, MI	4	Abundance of glacial material of varying thickness
		Huronian	Cambrian	NY, WI, MI	4	
		Laurentian	Cambrian	NY, WI, MI	4	
13	Ozark and Ouachita	Fayetteville	Mississippian	AR, OK, MO	3	May contain some montmorillonite in mixed layer form
		Chickasaw Creek	Mississippian	AR, OK, MO	3	
14	Interior Low Plains	Meramac Series	Mississippian	KY	3	
		Osage	Mississippian	KY, TN	3	
		Kinderhook	Mississippian	KY, TN	3	
		Chester Series	Mississippian	KY, IN	3	Interbedded shale, sandstone, and limestone
		Richmond	Upper Ordovician	KY, IN	3	
		Maysville	Upper Ordovician	KY, IN	3	
		Eden	Upper Ordovician	KY, IN	3	
15	Appalachian Plateau	Dunkard Gp	Pennsylvanian-Permian	AL, TN, KY, VA, WV, MD, PA, NY	3	Interbedded shale, sandstone, limestone, and coal
16	Newer Appalachian (Ridge and Valley)	Catoctin Gp	Precambrian	AL, GA, TN, NC, VA, WV, MD, PA	4	Metamorphosed rocks
		Lynchburg Gp	Precambrian	AL, GA, TN, NC, VA, WV, MD, PA	4	
		Switt Run Gp	Precambrian	AL, GA, TN, NC, VA, WV, MD, PA	4	
17	Older Appalachian	Carolina Slate Gp	Paleozoic	AL, GA, NC, SC, VA, MD	4	Metamorphosed and intrusive rocks
		Kings Mountain Gp	Paleozoic	AL, GA, NC, SC, VA, MD	4	
		Brevard Gp	Paleozoic	AL, GA, NC, SC, VA, MD	4	
18	Triassic Lowland	Newmark Gp	Triassic	PA, MD, VA	4	
19	New England Maritime	Glacial Till	Pleistocene and Ordovician through Devonian	ME, NH, VT, MA, CT, RI, NY	4	Glacial deposits underlain by nonexpansive rocks. Local

Continued on next page

Table 5-3. (Continued)
Tabulation of Potentially Expansive Materials in the United States

Physiographic Province No.	Physiographic Province Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map** Category	Remarks
						areas of clay could cause some swell potential
20	Atlantic and Gulf Coastal Plain	Talbot and Wicomico Gps	Pleistocene	NC, SC, GA, VA, MD, DE, NJ	4	Interbedded gravels, sands, silts, and clays
		Lumbee Gp	Upper Cretaceous	NC, SC	3	Sand with intermixed sandy shale
		Potomac Gp	Lower Cretaceous	DC	3	Sand with definite shale zones
		Arundel Fm	Lower Cretaceous	DC	1	
		Continental and marine coastal deposits	Pleistocene to Eocene	FL	4	Sands underlain by limestone, local deposits may show low swell potential
		Yazoo Porters Creek Selma	Paleocene through Pleistocene	AL, GA, FL, MS, LA, TN	1	A complex interfacing of gravel, sand, silt, and clay. Clays show varying swell potential
		Loess	Pleistocene	LA, MS, TN, KY	4	
		Mississippi alluvium	Recent	LA, MS, AR, MO	3 1	A mantle of uniform silt with essentialy no swell potential
		Beaumont-Prairie Terraces	Pleistocene	LA, MS, TX	1–3	
		Jackson, Claiborne, Midway	Paleocene	LA, MS	1–2 3	Interbedded stringers and lenses of sands, silts, clays, marl, and chalk
		Navarre, Taylor, Austin	Upper Cretaceous	TX	1–3 3 4	
		Eagleford, Woodbine	Upper Cretaceous	TX		
		Washita		TX		
		Fredricksburg	Lower Cretaceous			
		Trinity		TX		
			Lower Cretaceous	TX		
			Lower Cretaceous			

5.5.1 Expansive Soil and Rock

Soil and soft sedimentary rock containing relatively large amounts of expansive clay minerals such as illite and the smectites in general (the variety montmorillonite in particular) are themselves capable of exerting enormous swell pressures on engineered facilities. The magnitude of this damage to transportation structures over the years has been so large in the United States that the Federal Highway Administration (FHWA) has commissioned an intensive four-year study of expansive earth materials (Snethen, D.R., Johnson, L.D. and Patrick, D.M., and others, 1978). This work represents the state-of-the-art for detecting swelling soil materials, assessing their potential for swelling and designing subgrades to minimize such swelling.

Expansion is caused by the extraordinary ability of certain swelling clay minerals to take on unusually large amounts of water in relation to the bulk of their own individual mineral platlets and to molecularly bind such water. Next to clay mineral type, soil fabric is the most important factor in swell potential. *Fabric* refers to the structural orientation of the clay mineral platlets, in a range of orientations ranging from completely dispersed to completely *flocculated.*

The FHWA/WES study confirmed the traditional geotechnical viewpoint that expansive soil swelling pressure is activated mainly by construction activities or facility operating factors that expose the potentially expansive soil or soft rock to the elements and then allow an increase in moisture content to occur. When expansive materials are confined by high overburden pressures they are both protected from increases in soil moisture and restrained from expansion. As soon as the overburden is stripped, both atmospheric moisture (in non-evaporative climatic conditions), precipitation, and construction moisture can be absorbed into the clay mineral components of natural ground. Increases in compacted embankment moisture may also lead to swelling.

The mechanisms for intake of pore water all operate on the microscale. The criteria for activation of expansion are:

- Swelling clay minerals
- Exposure to an available source of porewater
- Removal of confining stress or placement of compacted materials at a moisture content which is dry of optimum moisture content
- Soil-fabric-related mechanisms to induce water intake at the platlet level and thereby create volumetric expansion

Field personnel should be aware of the general conditions under which expansion can occur and should be prepared to identify expansive materials. Snethen (and co-workers, 1975) compiled a series of five regional maps showing occurrences of swelling clay soils of the continental United States (Figures 5-9 through 5-13) and Table 5-3. These maps show areal distributions of geological units which contain argillaceous members and the clay mineral montmorillonite. The maps are keyed to FHWA regions for ease of use by transportation agency personnel. The map patterns denote generalized expansion potentials of high, medium, low, or nonexpansive. The user is reminded that the degree of expansion potential varies considerably even at a single site, depending upon the nature of individual beds of sedimentary rock encountered or upon the thickness and distribution of surficial soil units containing expansive clay minerals. The impact of the degree of expansion must also be measured against the type of roadway or structure contemplated. For example, areas in low-expansion potential may still represent distinct design considerations for design of some pavements and foundation slabs, drilled piers and grade beams.

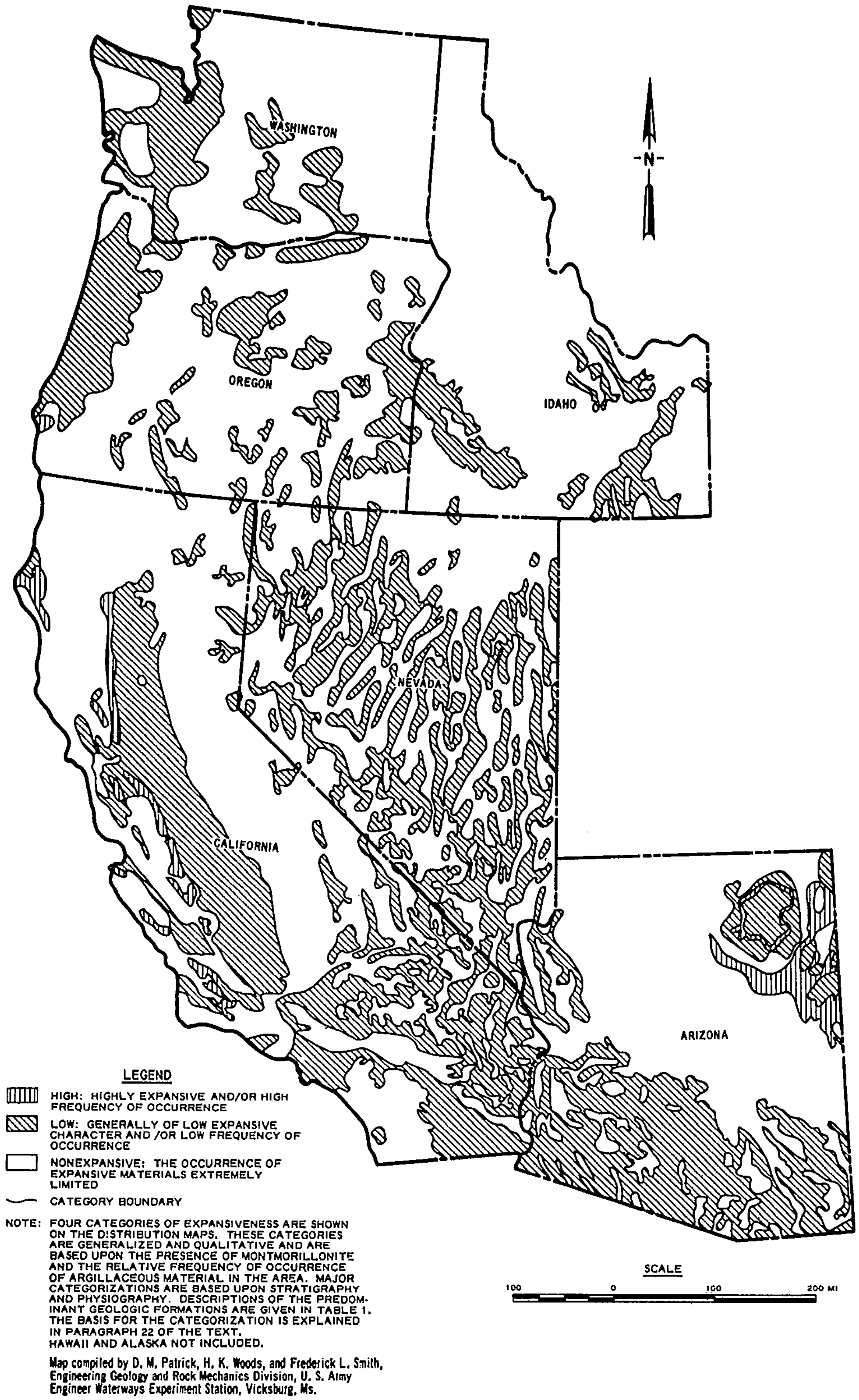
Keys to successful identification of expansion potential on individual project work are: (1) to be familiar with regional geologic literature, (2) to be aware of the presence and distribution of clay-rich, argillaceous rocks, and (3) to identify such in the course of reconnaissance mapping and later subsurface explorations. Additional determinants of use to field personnel in detecting expansion potential are as follows:

- Argillaceous (shaly nature)
- USCS classification of CH or CL
- Irregular, pebbly texture on exposed surfaces; resembles popcorn
- Closely-spaced desiccation cracks during dry periods
- Slickensides in freshly excavated or trenched materials
- Strain-related damaged to nearby structures (the four above-cited indications are from Snethen, 1979)

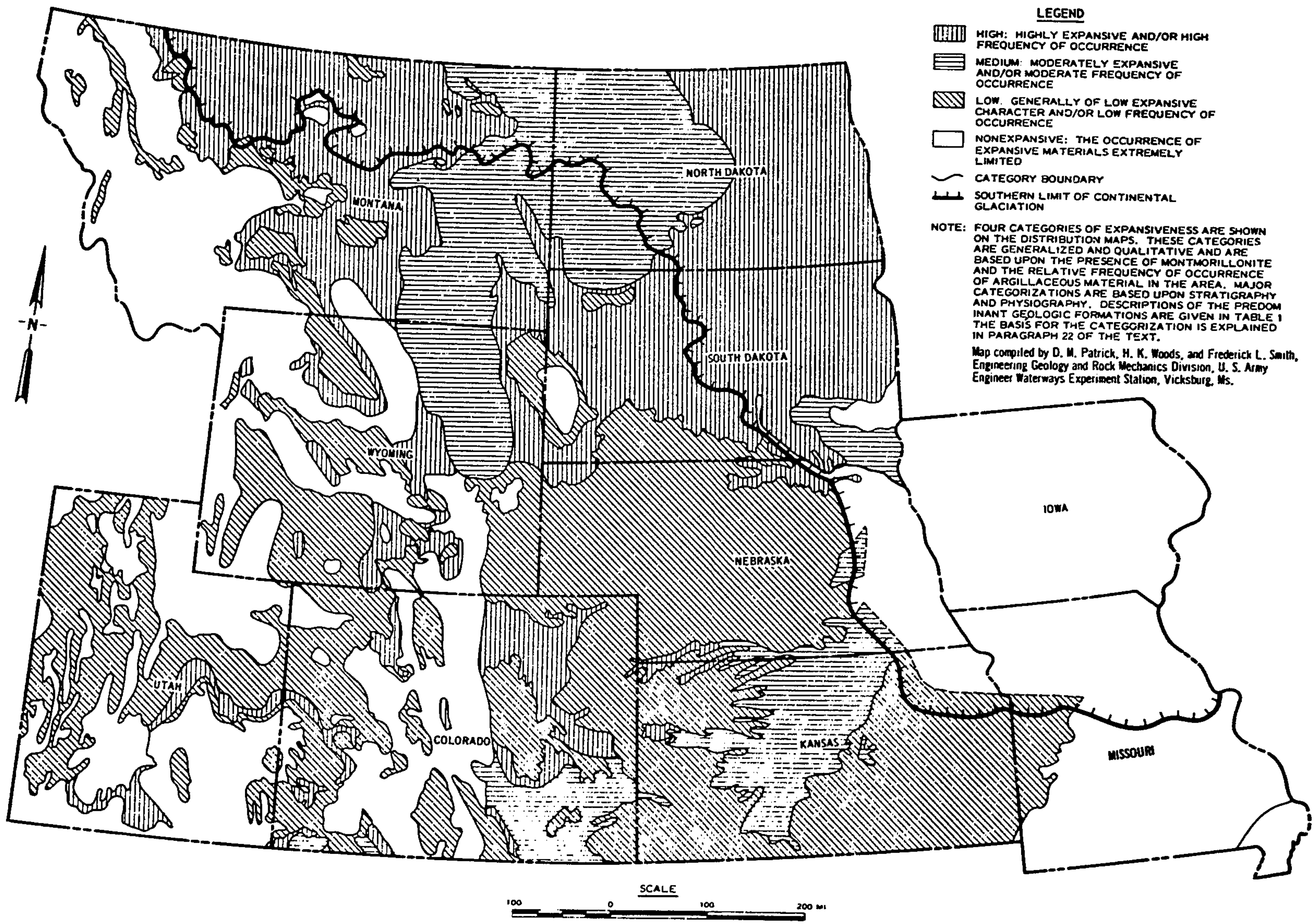
When an expansion potential is indicated by regional experience, by reference to the literature (and maps), or from field indications, additional laboratory testing should be considered. The broad categories of laboratory test indicators of expansion potential are as follow:

- Minus 200 screen fractions in excess of 80 percent (although lower percentages may also be important)
- 2-micron fractions in excess of 20 percent
- Plasticity Indices greater than 20, Liquid Limits greater than 30, Plastic Limits greater than 20
- Clay mineral determinations of illite or the smectites
- SEM (scanning electron microscope) determinations of flocculated or semi-flocculated clay mineral fabrics

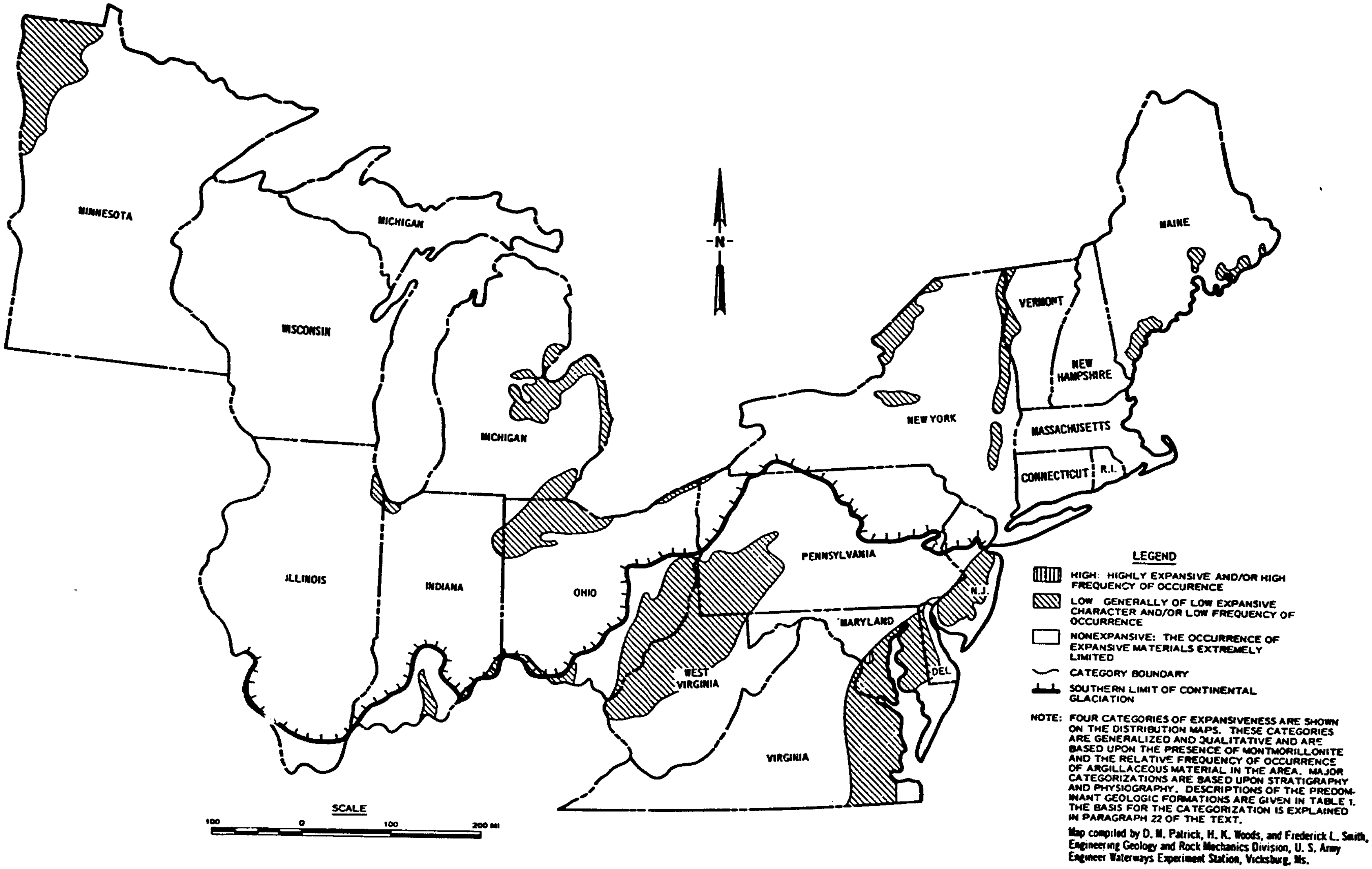
Geotechnical engineers will want to consider one or more of the laboratory tests to evaluate the swell potential of such soil or rock materials. The most useful of the tests are the one-dimensional, wetted consolidation test and the soil-suction thermocouple psychrometer test (Snethen, Johnson, and Patrick, 1977). These tests are discussed in Section 9 and are referenced in C.



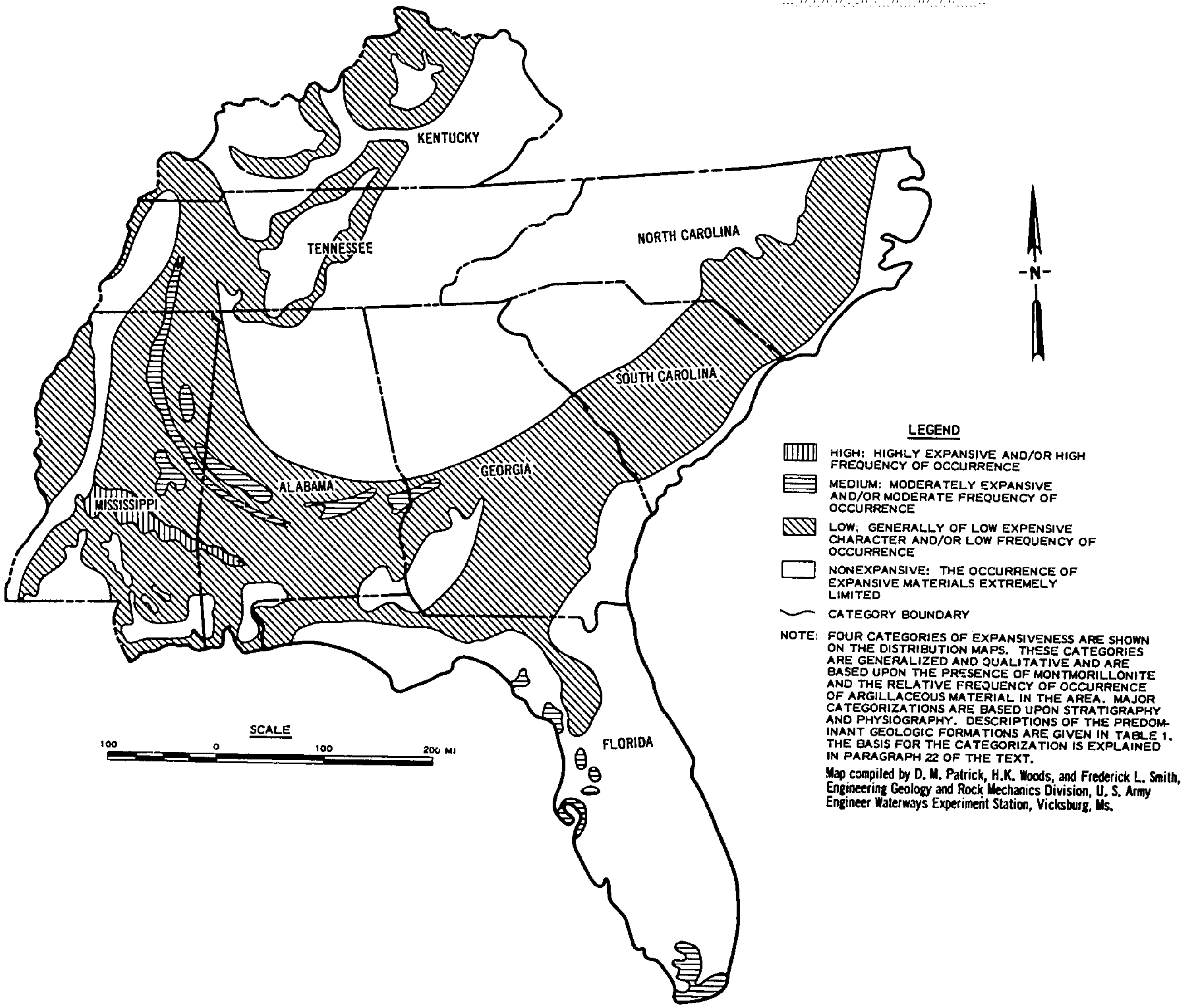
5-9. Distribution of potentially expansive materials in the United States: FHWA Regions 9 and 10. Snethan et al., 1975



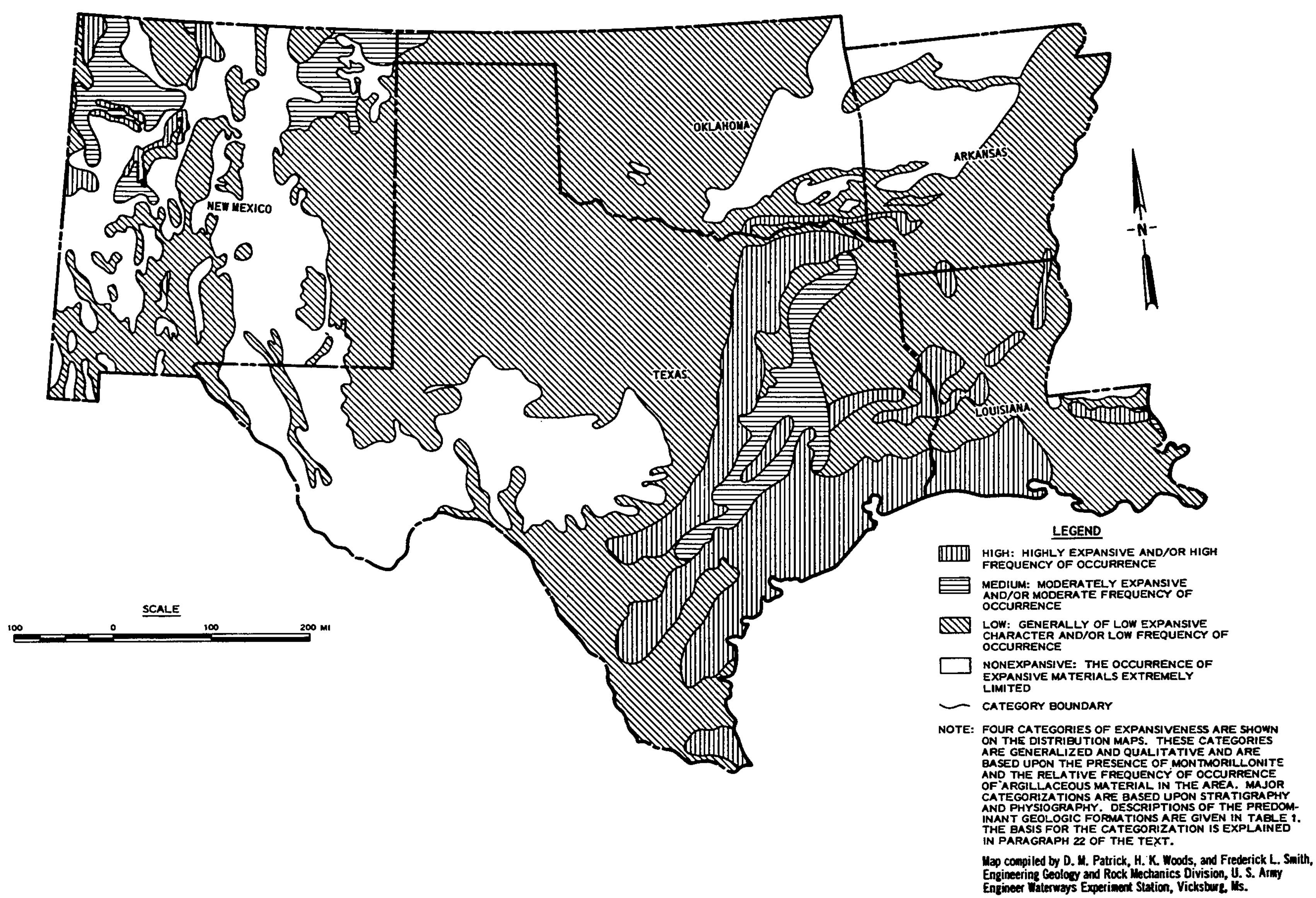
5-10. **Distribution of potentially expansive materials in the United States: FHWA Regions 7 and 8. Snethen, et al., 1975.**



5-11. Distribution of potentially expansive materials in the United States: FHWA Regions 1, 3, and 5. Snethen, et al., 1975.



5-12. Distribution of potentially expansive materials in the United States: FHWA Region 4. Snethen, et al., 1975.



5-13. Distribution of potentially expansive materials in the United States: FHWA Region 6. Snethen, et al., 1975.

5.5.2 Collapse-Prone Soil

Some underconsolidated engineering soil units of relatively young age (Holocene to Late Pleistocene) have a potential for instantaneous collapse. In most cases, the volumetric decrease is associated with porewater saturation and a dissassociation of silt grains and clay mineral platelets which had been previously arranged in an unstable soil structure. In order to fall into this collapse-prone category, soils are usually mixtures of fine sand, silt, and clay-sized materials, with the silt dominating.

As can be noted from the two cited figures, not all soils of similar size gradation and from the same region may prove to be collapse-prone. Slight differences in the mode of geologic origin, of previous saturation and loading experience of the soil, and the nature of its previous porewater chemistry appear to be the controlling factors.

Much of the collapse-prone soil units in the United States are associated with the Late Pleistocene loess blanket of windblown silt soils of the Pacific Northwest and those that lie adjacent to the Mississippi River and its tributaries. Other collapse-prone soil units are found throughout the United States (Dudley, 1970).

As Dudley notes (1970), one primary characteristic and one primary soil condition are generally responsible for the collapse-prone nature; a loose (large-void ratio) structure and a moisture content of less than saturation level. Dudley found that these loose states were represented by a spread of dry unit weight values of from 1100 to 1700 Kg/m^3 (80 to 104 pcf). Extensive deposits of collapse-prone soils are found in California's San Joaquin valley in the form of alluvial fan deposits spreading outward from ranges of low hills. Both highways and the California Central Water Project canal were designed for preconstruction wetting and treatment of these stretches of low-density soils.

Entry of water into freshly-exposed cuts or excavations in loessal soils leads to rapid widening of vertical microfractures into piping channels (Figure 5-14) and associated soil-structure collapse.

Identification of collapse-prone soil units should be made on the basis of low density and grain size with followup laboratory tests involving a saturated oedometer test. Instantaneous collapse on the order of 10 to 20 percent should be suspect. Arman (1973) has developed a recommended color-change field identification test now adopted by the Louisiana DOT, making use of addition of sodium hexametaphosphate (Calgon, or other brand name) with dry soil and addition of distilled water. Obviously, this test may not apply to other locations, but it does indicate that differences between stable and potentially unstable soil units are linked to distinct characteristics or properties of each soil.

Collapse-prone loess and water-lain, sheet-flood deposits along the western edges of the Great Plains can sometimes be identified by immersion of solid fragments of the soil in a jar of water. Potentially unstable soils generally disintegrate fairly rapidly with a noticeable sloughing occurring within seconds of immersion. The following references provide more information on collapse-prone soils: Gibbs and Bara, 1962; Holtz and Hilf, 1961; and Knight and Dehlen, 1973.

5.5.3 Shale and Clay Shale

Relatively fine-grained, indurated sedimentary rock materials, known broadly as shale and clay shale, often exhibit durability problems (Fig. 5-15) that can lead to a variety of roadway, cut slope and structural failures. These rocks are the rock equivalent of silts and clays of soil classification, and these soil types were the parent materials for shale and clay shale. Around the world, these rocks are also known as (Morgenstern and Eigenbrod, 1974) argillaceous sediments, claystone, siltstone, mudstone, and mudrock. The rock type itself is simple to recognize in outcrop and in borings; the difficult task is to assess which of the shales and clay shales exhibit unfavorable engineering characteristics. These detrimental aspects of fine-grained sedimentary rock are usually grouped as softening, shrinkage, slaking and swelling.

Underwood (1967) has produced a paper dealing with the classification and identification of all types of shales which is useful for geological classification of fine-grained rock for engineering purposes.

Shales and associated rock types tend to pose problems to engineered construction when they possess characteristics leading to some form of disintegration in service. The variabilities in slaking and related shale phenomena are so wide it is improbable that a universal test can be developed for use by all agencies. This recognizes that the factors responsible for unsuitable shale behavior are varied, as shown in Table 5-4.

Most geologic formational units of fine-grained rocks will exhibit similar engineering behavior throughout their areal exposure. For this reason, it is imperative for transportation agencies to record the engineering behavior and characteristics of fine-grained rocks as they are tested and utilized on various projects. The Virginia DOT, for example (Noble, 1977) found that Devonian-aged Millboro and Brallier Formation black shale would develop chemically-induced distress when removed from its natural *in situ* state of equilibrium. When exposed to the atmosphere and free water, minute pyrite crystals

5-14. The high erosion potential of loess is here depicted by a runoff-originated piping cavity formed in a 24-hour period of one inch (25 mm) of rainfall. (A. W. Hatheway)

begin to oxidize and produce sulfuric acid which leaches calcium from chlorite and carbonates minerals in the rock. The freed calcium can replace potassium ions in the illite, causing a minor degree of expansion. All in all, the black shale has a high potential for slaking and disaggregation. Virginia DOT geologists now take special care in identification of the presence of the black shale and outcrop samples are inspected carefully for the pyrite grains which begin the chain reaction of decomposition. If the pyrite is present, then laboratory determinations for clay mineral type should be performed to search for indications of the presence of chlorite and illite. Purdue University (Deo, 1977) has developed a slake durability test and a resulting classification of four grades of shale in terms of potential transportation system useage.

As in the case of all other geologic materials, the shales must be carefully mapped and classified in the field in order to identify which units appear to be distinct in terms of slake durability. The more definite discrimination between units, the greater the chances for suitable utilization of at least some of the strata encountered in a given project.

5-15. Slaking shale component of coarse-cobble slope protection. This silty shale has disintegrated in one season of exposure to the elements in the mid-Central states. (A. W. Hatheway)

5.5.4 Sensitive Clay Soils

Certain coastal areas of the northern hemisphere, notable in the St. Lawrence seaway region of the United States and Canada and in Scandinavia, have extensive deposits of marine clays and silts which were deposited at elevations above present sea level and in the recent geologic past. Although such clay soils generally appear quite hard and often stand up well as natural slopes, they possess a potentially unstable fabric of dispersed clay platelets. Changes in the geometry of natural slope areas, such as through road construction and urban development, have made many locales susceptible to catastrophic loss of shear strength and rapid flowage due to the imposition of additional loads, earthquake ground stresses or transportation-related ground vibrations. Figure 5-16 illustrates the striking contrast between undisturbed and remoulded strengths of these sensitive clays.

Field personnel should recognize such materials

Table 5-4
Geologic Factors Responsible for Unsuitable Shale Behavior

- Degree of preconsolidation through geologic time
- Nature of cementation bonding individual mineral grains
- Mineral content of the shale; presence of platy minerals such as micas, clay and swelling clay
- Degree of bedding present
- Degree of bedding lamination, alternation of coarse and fine laminae
- Compressive strength (ranges from 0.170 to 103 N/m^2; 25 – 15,000 psi; Underwood, 1967)
- Degree of chemical alteration present
- Nature and spacing of jointing

from general geologic relationships within a given physiographic province as well as their clay-rich and apparently stiff nature, their occurrence at elevations well above sea level as well as below sea level. The silts and select clay soils will liquefy when shaken or jarred in the hand. Laboratory strength testing of undisturbed samples will numerically define sensitivity. When properly remolded and re-compacted, these soils lose their sensitive character.

5.5.5 Frost Heave Susceptibility

Permafrost conditions exist widely in the northern hemisphere above 50 degrees north latitude. Many generalized maps of frozen ground distribution are available. Most of them subdivide frozen ground into two categories, continuous, seasonal permafrost and discontinuous permafrost. About 20 percent of the Earth's land surface is affected by permafrost conditions as is nearly half of the territory of Canada. Permafrost mitigating designs for transportation systems add considerably to their costs and routing and siting generally take this effect into consideration. Although the Northerly states, except for Alaska, do not have permafrost conditions they do experience seasonal ground heave associated with sub-freezing conditions, frost-susceptible soils and available water.

Gravel capillary break layers or pads, deep foundations and location on topographically high and free-draining soils are all methods of combating the heave and accompanying structural damage associated with freezing ground. In addition to frost heaving, thaw subsidence, soil creep and slope movements are all activated by ground frost. Additionally, soil shear strength is highly temperature dependent. Foundation soils may undergo long-term creep as the ice component of frozen soil deforms under the structural load.

Most authorities agree that frost susceptibility is linked most directly to soils made up predominantly of particles in the very fine sand through clay fraction. Soils with *in situ* permeability less than about 10^{-4} through 10^{-6} cm/sec are most frost susceptible. For permeability less than about 10^{-6} cm/sec the susceptibility begins to decrease due to the more limited water flow capacity of these fine-grained soils. Potentially frost-susceptible soils are usually considered to be those having more than 3 percent by weight finer than 0.02 mm. In the absence of hydrometer test results, there may be reason for concern with soils having more than 10 percent by weight passing the No. 200 sieve.

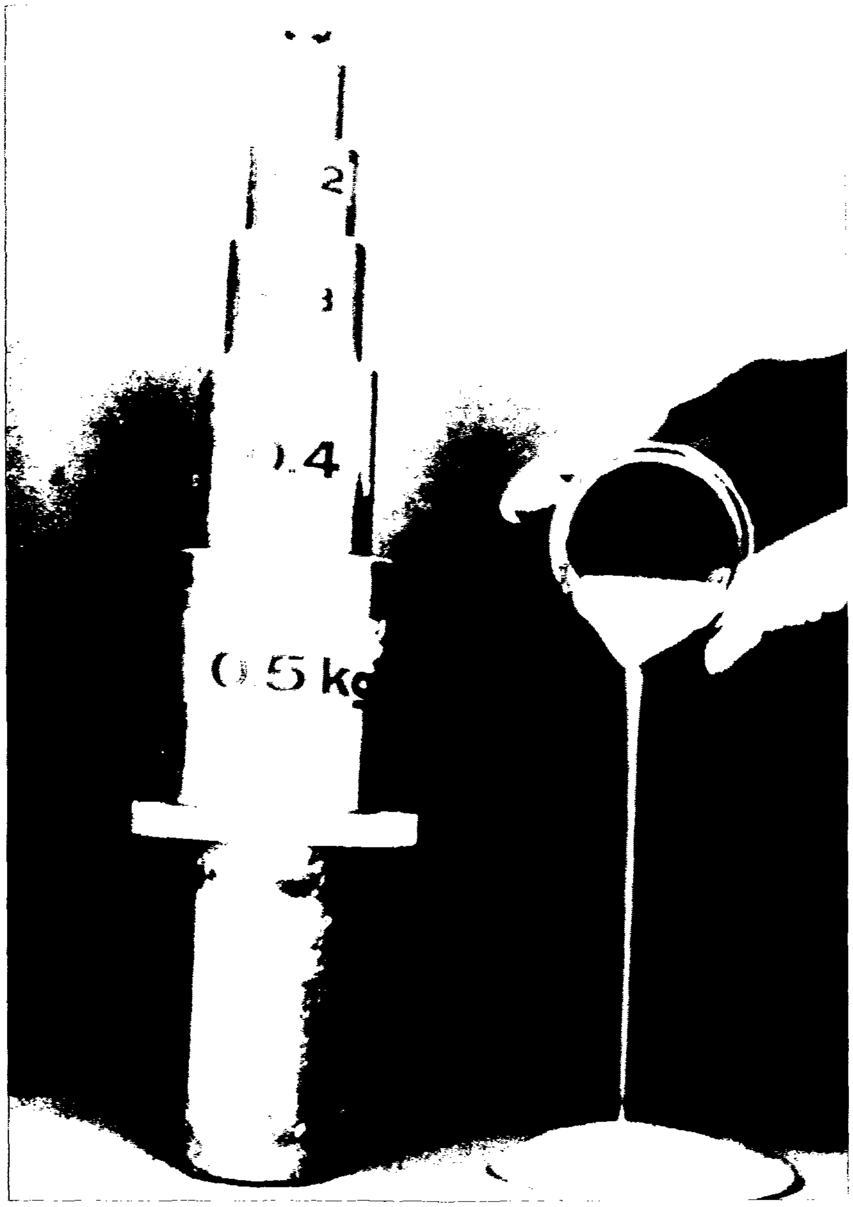
5.6 FLOODING

Flooding in the United States results in the loss of more life than from any other natural hazard. This loss is about ten times that suffered in the long term due to earthquakes. Hydrologic data relating to flooding has been collected by the U.S. Geological Survey since the 1902 Passaic River flood of New Jersey. Few locations in the country are without adequate means of estimating flood magnitude-frequency relationships. The nationwide data base contains information on historic magnitudes, frequencies and extent of flood-prone lands. Most of these data are developed and analyzed by hydrologists. Geotechnical personnel can, however, provide additional, site-specific interpretations from remote imagery and field mapping that will be useful in determining the actual flood extent for relatively small areas which may otherwise be shown incompletely or at a less than adequate detail for project planning purposes. The primary sources of information relating to local historic and predicted 100-year floods are the U.S. Geological Survey Hydrologic Atlases and flood hazard maps provided to users through the Federal Emergency Management Agency. As of 1979, more than 13,000 individual flood hazard maps had been produced.

In all instances, planning engineers and hydrologists should see that experienced geologists review the available flood maps in order to detect evidence of localized variations in formerly inundated areas, that would affect detailed design of a project.

5.7 EROSION

Sediment is a term covering all forms of earth and rock particles or fragments that are dislodged from



5-16. As shown in this view, the clay soil is stable under static loads, but on the event of dynamic excitation becomes liquified and viscous. (Courtesy Haley & Aldrich, Inc.)

their natural surroundings during construction activities. Most natural land surfaces have gained an equilibrium condition between slope degree, incident rainfall and snowmelt, stream flow, vegetation and the particular aspects of exposed soil and rock that make these materials resistant to erosion. Any disturbance of the land surface by construction promotes erosion, in the form of removal of earth and rock particles and fragments by natural agents, mainly flowing water and wind. Soil is far more susceptible to erosion than is rock. The main characteristics of soil that are important to erosion resistance and retention of sediment are:

- *Soil structure;* a measure of the physical and chemical bonds that hold constituent particles together in opposition to the action of flowing water and wind.
- *Soil texture;* the gradation of particle sizes making up the soil.
- *Moisture content;* a component of developed soil structural characteristics.
- *Porosity and hydraulic conductivity;* (permeability); the ability of a soil to retain or carry groundwater flow.
- *Organic content;* often provides favorable binding characteristics in terms of soil structure and

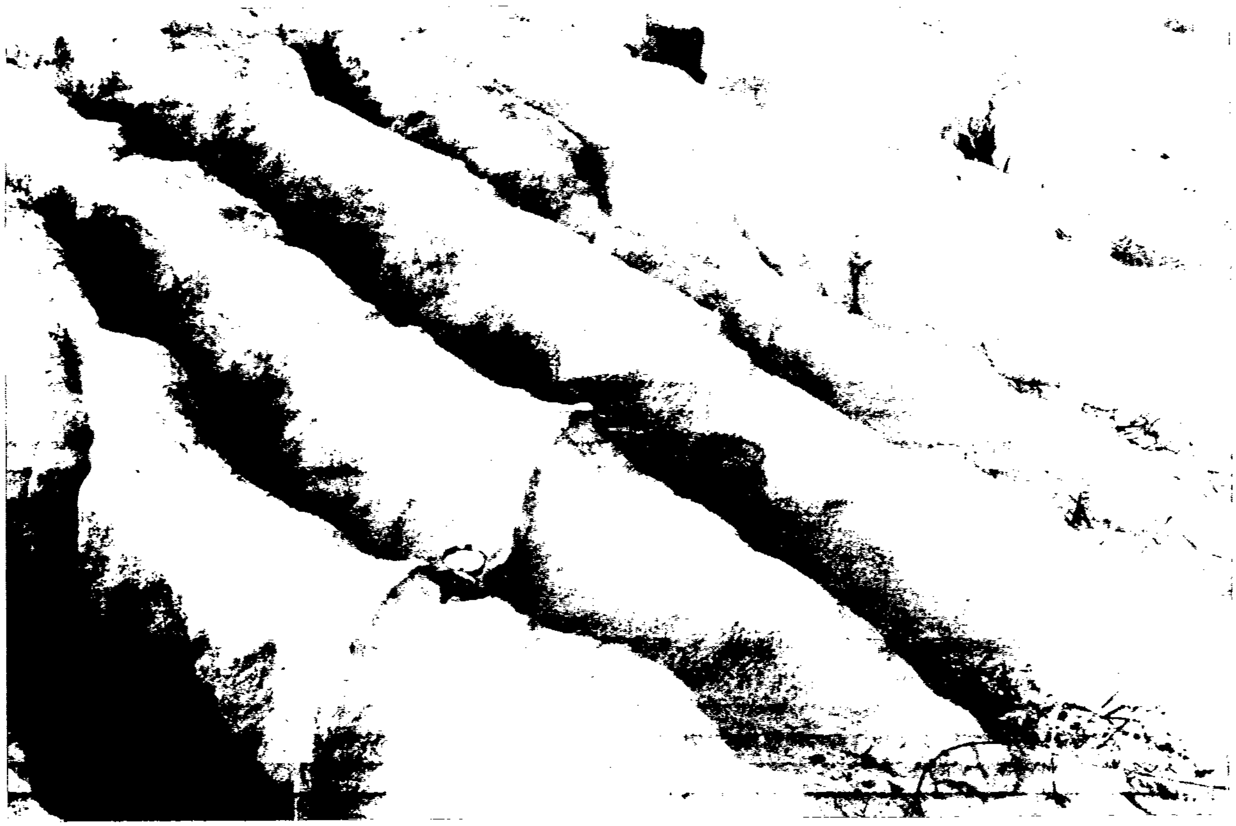


Figure 5-17. Erosion rills developed in clayey sand in a two-year period, over a slope that has not been protected by runoff provisions (note geologist's compass for scale). (A.W. Hatheway)

is influential in establishment of vegetation to assist in erosion control.

Sediment damage is estimated to run into the hundreds of millions of dollars annually in the United States. Minimization of erosion and control of released sediment is an essential aspect of transportation system planning. Planning and design considerations are usually managed by the general design team for transportation projects. Input from geotechnical personnel is important, however, for a number of reasons. The greatest single aspect of erosion potential is the nature of geologic units that will be exposed by construction. Some of this erosion occurs directly at the exposed surface, other erosion occurs as the result of internal flow of pore water in saturated silt and sands which result in piping. Both conditions can be anticipated on the basis of geological information developed in the course of site investigations.

Necessary construction activities will affect the geologic units that are encountered and defined during exploration. Specifically, design and construction activities that may lead to erosion and sediment production are as follow:

- Interception and concentration of runoff by roadways
- Exposure of soil and rock surfaces to the elements before surface treatment and/or revegetation
- Alteration of stream cross sections or bank conditions
- Improper sizing of hydraulic structures, with resulting surface flow and erosion
- Improper channeling, piping, and disposal of runoff collected on the route or site

The design team will need considerable geologic data to assist in mitigating erosion and the production of sedimentation. As geologic units are identified and defined, information relating to the erosion resisting characteristics noted above should be compiled and presented so that each mapped unit may be evaluated for its effect on the sediment control plan. As a guideline to this evaluation, the design team can minimize sediment production along the project alignment by considering the following factors:

- Effect of topography in exposing natural materials through cuts and fills and in channeling surface runoff and stream drainage toward, across and beyond the project

- Relative resistance of each exposed geologic unit to erosion, both surficial and internal
- Procedures to expose raw surfaces of rock and soil for only minimal periods of time
- Maintenance methods that will minimize the impact of sediment on sediment control structures installed as part of the project

The main types of erosion should be kept in mind during field mapping. Observation of examples of these types of erosion that are currently present in the site area form an important part of the environmental baseline and are important factors for the design team to take into consideration during formulation of its sediment control plan:

- *Sheet erosion:* dislodgment of soil particles by the impact of individual rain drops and transport of the particles by surface runoff.
- *Rill and gully erosion:* formation of semi-parallel drainage channels separated by ridges of about equal volume (Figure 5-17). Rills are the smallest and first indication of channeling of the runoff and removal of particles. Rills become gullies in the depth range of 0.5 to 1.0 m (1.5 to 3 ft.) and gullies become stream valleys and channels as they grow and coalesce.
- *Stream channel erosion:* the third stage in erosion channeling; individual water courses separated by much broader intervals of relatively uneroded terrain.
- *Wind erosion:* the more equal removal and surface transport of individual particles across relatively even surfaces of terrain.

In addition to qualitative observations of erosion susceptibility or potential in the field, some Agencies conduct studies of the erosion resistance that can be designed and built into engineered earthwork. The California DOT, for example, has constructed a rain simulation tower which is used to evaluate erosion resistance. Soils of various textures, bonded by different cementation additives and surface bonding treatments, and placed by various compaction techniques and energy levels can be tested.

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6.0 ENGINEERING GEOPHYSICS

Geophysical techniques applied to geotechnical investigations can be categorized into two general groups—investigations conducted from ground surface and those conducted in boreholes. Each group is further separable into two basic modes of data generation; measurement of either existing earth fields (passive) or measurement of fields induced deliberately for the purpose of the investigations (active). Investigations conducted from the ground surface typically provide information about the subsurface both laterally and to some depth, while most of the borehole investigations, with some exceptions, provide detailed information about materials only in the immediate vicinity of the borehole or between boreholes. The existing energy fields and induced energy fields pertinent to geotechnical investigations include:

Existing (Passive) Fields	*Induced (Active) Fields*
Gravimetric	Seismic
Electric	Acoustic
Magnetic	Electric
Thermometric	Electromagnetic
Nuclear	Nuclear

These fields represent those most useful in terms of the engineering requirements, but others exist that might be used under special circumstances (e.g., randomly-occurring seismic events, ground tilt, and natural electromagnetic fields).

The interest in existing energy fields occurs because the strength of the field at any particular point can reflect the geological conditions present between the point and the source of the field, such as proximity of bedrock, varying stratigraphic or hydrologic conditions, or mineral changes indicative of the stratigraphy present. The geologic conditions which result in measurable geophysical anomalies may be due to geologic conditions which are of little significance in a geotechnical investigation. Furthermore, interpretations of anomalies may be ambiguous since an anomaly may be due to natural geologic conditions or to the manner in which geophysical measurements are made.

Geophysical methods that rely upon the reaction of subsurface materials to energy introduced by some deliberate process are typically much more versatile for geotechnical purposes. These active geophysical techniques can be tailored to the needs of particular investigations. The appropriate equipment can be selected, the locations for investigation chosen, and the parameters measured in accordance with the specific project requirements (within the ability of geophysical techniques to provide such measurements).

Fundamental to the entire process of making geophysical measurements, and a concept sometimes overlooked, is selection of the method or methods appropriate to measure or derive the needed parameters, based on a knowledge of how the resulting data are to be used, and how the data should *not* be used.

In general, a single geophysical technique may not always provide the information needed for engineering investigations. A combination of several complementary methods usually provides more information and detail than might be expected. The purpose and limitations of any particular investigation should be clearly understood before selecting the approach to be used. This is because a moderate amount of additional effort in data collection may add a significant increase in the volume of additional information with somewhat broader application. All potential aspects of an investigation purpose should be considered in terms of what the geophysical methods can provide. A more cost-effective investigation can often be designed so that the need for later geophysical surveys can be avoided.

General texts describing the nature and measurement of the geophysical fields of interest include Rogers (1973), Stacey (1969), Parasnis (1966), Grant and West (1965), Dobrin (1960), and Nettleton (1940). Practical and theoretical bases for elastic wave propagation in the earth are given by Cagnaird (1962), Musgrave (1967), Ewimg, Jardetsky, and

Press (1957), and Love (1944). Gravity is treated thoroughly by Parasnis (1962), electrical phenomena by Van Nostrand and Cook (1966), Mooney and Wetzel (1956), and Guyod (1944), and electromagnetic waves by Wait (1962). Details of additional geophysical fields are presented below in the discussions regarding each particular method as applied to engineering investigations.

6.1 USE OF DATA

The data derived from geophysical investigations usually have to be interpreted by experienced geophysical analysts prior to use by engineering geologists or geotechnical engineers. Interpretations are both direct (calculations from established formulae or tabulation of data readings), or extrapolations based upon the experience of the individual data analyst.

In all but a few applications, such as reconnaissance investigations for example, the results of geophysical investigations should *always* be supported by direct observation of subsurface conditions by means of borings, test pits, trenches, outcrops and other geological information. Such direct measurements will assure that subsurface conditions not measured by the geophysical methods are discovered and support or negate interpretations made on the basis of geophysical methods. Each geophysical technique has facets that, if not recognized, can cause serious misinterpretation or misuse of the results. Awareness of the potential for error must be recognized annd anticipated so that proper "calibration" of the results is possible.

Measurements of the existing geophysical fields and resulting interpretations range from detailed gravimetric plan maps showing relative depth of bedrock (or actual depth if appropriately calibrated) to the identification of zones and flow rates of moving groundwater penetrated by boreholes. In each case, the density of surface observation stations (gravity, magnetics, electrical) or frequency of borehole recording or measurement points (electrical, nuclear, thermometric) establishes the resolution level of the data collected. The basic sensitivity of current instrumentation is sufficient to measure existing fields at the levels useful to geotechnical investigations.

Induced-field geophysical techniques are more widely used than passive techniques. Joint use of both induced and existing fields is common in some types of investigations. Selection of the method used in the induced case can be based upon a need for depth of coverage (seismic, electrical, electromagnetic), versus the specific type of information needed (seismic, acoustic, nuclear, electrical). Resolution capability is also selectable to some degree, with resolution increasing as the density of observation points or rate of observation is increased.

Table 6-1 lists the geophysical methods against common engineering parameters that can be provided from application of each, or where the method can provide closely related informationn that provides a strong contribution to the parameter identified. A numerical rating is included in the table to indicate whether the parameter is measured or calculated directly from the measurements made, if the parameter is more or less directly inferred from the measurements, or if the measurements simply provide contributory information that would not identify the parameter by themselves. Table 6-2 indicates which geophysical methods can be used to investigate geologic conditions which may be important in the siting of transportation routes. Limitations of some of the methods make several of those shown less useful than might be initially expected, and some comments regarding actual usefulness are reflected in the discussions of the following sections.

General references dealing with applications of geophysical methods for geotechnical investigations include: U.S. Army, Corps of Engineers (1943, 1979); Ballard and Chang (1973); Culley (1976); Enslin (1953); Golder and Soderman (1963); Griffiths and King (1965); and Bison Instruments (1977). Specific applications for highway investigations are discussed by Black (1973); Lawson, Foster, and Mitchell (1965); Love (1967); Malott (1967); Mayhew, Struble, and Zahn (1965); Patterson and Meidav (1965); and West and Dumbleton (1975). Tunneling applications are included in papers by Schwarz (1972) and Scott, and others, (1968), with representative investigations for dams discussed by Gogoslovsky (1970), and Cratchley, and others, (1972). A sinkhole problem examined by geophysical techniques is described by Enslin and Smit (1955), and a groundwater investigation by Foster (1951). The references above represent approaches that include use of more than one geophysical technique. Additional case histories and specific applications for particular techniques are also identified in discussions of individual geophysical techniques, given below.

6.2 SCHEDULING

Geophysical investigation techniques are generally applicable to some degree throughout a project lifetime, ranging from the initial investigative phases through the final design phase.

The widest use of engineering geophysics occurs as an integral part of the initial site explorations, especially in phased investigations or to generally provide

Table 6-1.
Geophysical Field Methods

	EXISTING FIELDS					INDUCED FIELDS				
	Gravimetric	Electric	Magnetic	Thermometric	Nuclear	Seismic	Acoustic	Electric	Electromagnetic	Nuclear
P-Wave Velocity	3					1				
S-Wave Velocity						1				
Resistivity/ Conductance		2						1	1	
Temperature		3		1				3		
Density	2	3	3		1	2	2	3	3	1
State of Stress						2	2			
Shear Modulus						3				3
Youngs Modulus						3				3
Poissons Ratio						1				
Corrosion Potential				3				2	2	3
Permeability	3	3		3	3	2		2	3	2
Saturation		3		2	2	2		2	3	1
Aquifers	3	3		2	3	1		2	3	2
Groundwater Table	3	2		2	2	1		1	3	2
Groundwater Flow		2		2	3			2		2
Soil/Rock Type	2	2	2	3	2	2	2	2	3	2
Depth to Bedrock	2	2	2	2	2	1	1	1	1	2
Material Boundries	2	2		2	2	1	2	1	1	1
Strata Dip	2	2	2		2	1	2	2	1	3
Lateral Changes	2	2		3	2	1	2	2	1	3
Obstructions	2	2			3	1	2	2	1	3
Rippability						2	2	2	3	3
Fault Detection	2	2	2			2		2	1	3
Cavity Delineation	2	2				2	2	2	1	3

1. Direct measurement, or calculated from measurement
2. Inferred from measurement or calculation
3. Combined with other data to develop inference.

information between widely spaced "point" observations (i.e., boreholes, test pits, outcrops, etc.). Preliminary geophysical explorations (following a review of geological, topographical, and ownership conditions) can lead to realignment or site rejection or can indicate the need for additional explorations. Table 6-1 is helpful in determining when various engineering geophysics techniques should be used (i.e., at what point in time a particular parameter must be known in the decision process). The need for some methods is sometimes also identified during the investigation of a site by other geophysical techniques.

For major projects, use of geophysics is ordinarily defined before the field investigations begin since the

Table 6-2.
Uses of Engineering Geophysics in Geological Investigations of Transportation Routes

Geological Conditions to be Investigated	Useful Geophysical Techniques	
	Surface	Subsurface
Stratified rock and soil units (depth and thickness of layers)	Seismic Refraction	Borehole Logging
Depth to Bedrock	Seismic Refraction Electrical Resistivity	Borehole Logging
Depth to Groundwater Table	Seismic Refraction Electrical Resistivity	
Location of Highly Fractured Rock and/or Fault Zones	Electrical Resistivity	Borehole TV Camera
Bedrock Topography (troughs, pinnacles, fault scarps)	Seismic Refraction, Gravity	
Location of Planar Igneous Instrusions	Gravity, Magnetics, Seismic Refraction	
Solution Cavities	Electrical Resistivity, Gravity	Borehole TV Camera
Isolated Pods of Sand, Gravel, or Organic Material	Electrical Resistivity	Borehole Logging
Permeable Rock and Soil Units	Electrical Resistivity	Borehole Logging
Topography of Lake, Bay, or River bottoms	Seismic Reflection (acoustic sounding), side-scan sonar	
Stratigraphy of Lake, Bay, or River Bottom Sediments	Seismic Reflection (acoustic sounding)	
Lateral Changes in Lithology of Rock and Soil Units	Seismic Refraction, Electrical Resistivity	

role of the eventual results is well known in the design process. On smaller projects the use of geophysical methods is sometimes deferred until it is determined that more traditional investigations cannot provide the required information, or that geophysical techniques will provide the needed information on a more timely or more cost-effective basis.

6.3 PRESENTATION OF RESULTS

The methods of presenting engineering geophysical results are as varied as the range of parameters listed. The specific need should be identified prior to investigation, and the method of presentation should be chosen on the basis of the needs. The following items should be considered when selecting the method of data display:

6.3.1 Site Locus Map

The site locus map is used to identify the geographical location of the investigations and to provide site orientation with reference to the project coordinate system.

6.3.2 Investigation Plan Map

A geophysical investigation plan shows the points of exploration activity within the project coordinate system.

6.3.3 Data Results

Maps, cross-sections and profiles are used as appropriate to display raw data measurements, results of calculations from the measurements, or interpretation of the meaning of the measured and calculated values. Many results may be contoured, or discrete levels of response can be patterned similar to geological mapping as a means of showing different physical conditions in both aerial and cross-section views. Borehole data may be presented in a continuous depth vs. response level chart or as tabulation of response at discrete depths or depth intervals.

All separate reports of geophysical investigation results should include an explanatory text that presents the following minimal information:

- Purpose and Scope of the Investigation
- Dates and Locus of Investigation

- Personnel and Organizations Involved
- Amount of Data Collected
- Quality (Reliability) of Data Collected
- Method of Investigation/Equipment Employed
- Method of Analysis and Interpretation
- Interpreted Results
- Summary and Recommendations (as appropriate)

Charts, figures, and tabulations should be used routinely to document the results.

Where such data are available, the report should also include correlative or contradictory results of other investigations, particularly if the data are important to interpretation of the geophysical results. The sources of such information must also be identified. Experienced investigators will also include comment about or an estimate of the accuracy of the results of their investigations to aid in resolving any conflicts between geophysical and supporting geological data.

6.4 MAJOR METHODS

Induced energy fields are the most widely employed geophysical approach. Seismic refraction and electrical resistivity are the techniques most familiar to the geotechnical community. Related methods, such as seismic reflection, seismic surface wave analysis, and electromagnetic techniques are ordinarily employed only where the need for the specific capability can be identified.

Gravity and geomagnetic measurements have experienced increasing use in recent years, primarily for projects requiring reconnaissance mapping of bedrock elevations for groundwater or geological/structural studies. These techniques are not generally familiar to the geotechnical community, but the basic concepts can be readily understood.

Electrical and nuclear borehole logging can provide a continuous measurement of material properties immediately adjacent to the borehole walls. Very detailed interpretations can be derived from the logs by experienced logging analysts. The seismic technique may also be employed in boreholes, and the technique is sufficiently advanced to permit computerized calculation of significant engineering parameters on a continuous basis throughout the depth of individual boreholes. Since the range of information beyond the borehole wall is limited to a few decimetres (a foot or so), geotechnical investigations require a good understanding of borehole logging techniques.

One of the more useful borehole techniques involves measurement of the transit times of seismic waves between boreholes. This "crosshole" method permits calculation of bulk engineering parameters (e.g., Young's modulus and Poisson's ratio) under dynamic stressing and under *in situ* conditions. Use of the method is becoming standardized for larger projects because the parameters derived are useful in the design of underground openings and in earthquake engineering.

Seismic reflection investigations on land have somewhat limited use because of energy generation problems. However, shock wave energy is transmitted very well in a fluid medium, and reflection techniques have wide use in marine investigations (lakes and rivers, as well as ocean surveys). The marine application of reflection is sometimes called "Acoustic Profiling," and it is very effective in providing an essentially continuous representation of bottom and subbottom boundaries between materials of different physical characteristics.

Other geophysical methods that have specific application in certain instances are available, and can be used for investigations of a somewhat unique nature. Devices that permit detection of subaudible rock noise can be effective in providing information about the state of stress in soil and rock bodies (from the strains that are occurring) or for establishing the rates of slippage as indication of impending failures. Borehole TV cameras are used to provide a direct visual image of material conditions in the borehole walls that might only be inferred from other techniques, especially in zones of shattered rock where core is not recovered.

While many other geophysical techniques are available in certain types of investigations, those above represent methods that have been demonstrated effective in geotechnical investigations. Several of the borehole techniques are currently in a stage of rapid development (gravimetry, electromagnetic crosshole procedures) and computerized treatments of data collected at ground surface (seismic and resistivity) are constantly in a state of improvement. Additional use of geophysical techniques for geotechnical applications can be expected as improvements in both field procedures and analytical/interpretational approaches develop.

6.5 SEISMIC METHODS

Seismic methods (refraction and reflection) involve the measurement of the transmission velocity of mechanical waves in soil and rock units. Seismic wave velocities are controlled by the density of the materials and the presence of discontinuities such as joints and faults. The density of an earth material is affected by the mineralogy, porosity (void ratio) moisture content, degree of saturation, and degree of fracturing of the material. Seismic wave velocities are indicative of the gross or bulk nature of these combined material characteristics.

To perform a seismic survey, energy is imparted to the ground by striking a plate on the ground with a sledge hammer or by setting off an explosive charge at the ground surface or in a borehole. Mechanical or seismic waves propagate from the energy source and are detected by *geophones* placed at known distances from the energy source. The travel times of the seismic waves from the energy source to the geophones are measured by a *seismograph.* The distances of the geophones from the energy source divided by the travel times indicate the *seismic wave velocities* of the materials through which the mechanical waves travelled.

The velocities of seismic waves in earth materials are directly proportional to the bulk densities of the materials. Seismic wave velocities of rock are generally higher than in soils and unconsolidated sediments. Intact rocks will demonstrate higher wave velocities than fractured rocks, and soils with low porosities (void ratios) will demonstrate higher wave velocities than soils with high porosities. The seismic wave velocities of saturated earth materials are usually greater than those of partly saturated earth materials. Table 6-3 demonstrates the differences in seismic wave velocities in different materials.

Two types of seismic methods may be utilized; *seismic refraction* methods and *seismic reflection* methods. Refraction methods utilize the refraction of mechanical waves at the interfaces of different materials. Reflection methods utilize the reflection of mechanical waves at the interfaces. For the purposes of engineering geophysics, refraction methods are best suited for use on land while reflection methods are best suited for use in aqueous environments.

6.5.1 Seismic Refraction Method

The essential parameters of the seismic refraction method are indicated in Figure 6-1. The figure represents a two-layer case with horizontal boundaries where the seismic wave velocity in layer 2 is higher than in layer 1. The interface between layers 1 and 2 may represent the top of bedrock, the groundwater level, or the contact of two geologic units. Between the shot point and distance Xc, the first seismic waves to arrive at the geophones are those that travel through layer 1. Beyond the distance Xc, the seismic waves to arrive first at the geophones are those that are refracted at the boundary and travel through layer 2, where the seismic wave velocity is higher than in layer 1.

Table 6-3.
Typical Seismic Wave Velocities of Various Earth Materials*

Earth Material	Wave Velocity Feet/Second	Wave Velocity Metre/ Second
Top Soil		
Dry	600–900	180–275
Moist to Wet	1,000–2,500	305–760
Clay, Dense and Wet	3,000–5,900	915–1,800
Gravel	1,970–2,600	600–790
Cemented Sand	2,800–3,200	855–975
Glacial Till	5,600–7,400	1,705–2,255
Weathered and Fractured Rock	1,500–10,000	455–3,050
Shale	2,600–12,000	790–3,660
Chalk	6,300–8,000	1,920–2,440
Sandstone	7,200–9,000	2,195–2,745
Phyllite	10,000–11,000	3,050–3,350
Granite		
Fresh	16,000–20,000	4,875–6,095
Highly Weathered	1,540	470
Fractured and Weathered	2,200–8,000	670–2,440
Open Joints Present	10,000–13,000	3,050–3,960
Basalt	9,000–14,000	2,745–4,265
Metamorphic Rocks	16,400–20,200	5,000–6,155
Water	5,000	1,525
Air	1,100	335

* Data from U.S. Army Corps of Engineers (1979).

The travel time is the time between the energy shot and the *first arrival* of the seismic waves at each of the geophones which is recorded by the seismograph. Plotting the travel time to each geophone against the distance of the geophone from the shot point puts the field data into a form from which the seismic velocities of layers 1 and 2 may be calculated, as well as the depth of the interface between the two layers. Figure 6-2 shows the time-distance plot of data which may be obtained from the situation depicted in Figure 6-1. The inverse of the slopes of the curves are equal to the seismic wave velocities in the two layers. The *critical distance,* Xc, is the distance from the shot point at which the first arrival is a seismic wave which has

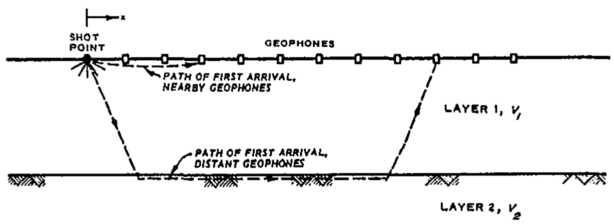

Figure 6-1. Essential parameters of the seismic refraction method for a two-layer case with horizontal boundaries. US Army Corps of Engineers (1979)

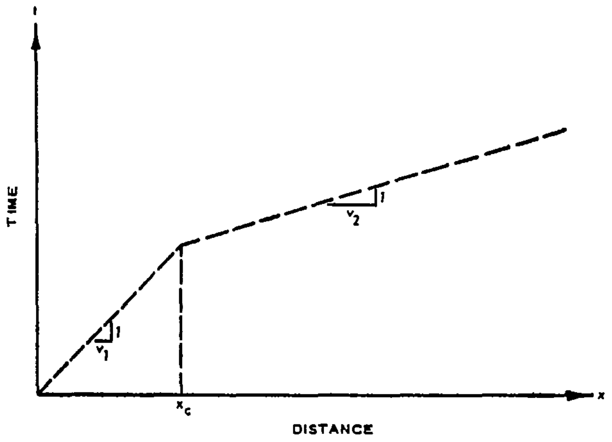

Figure 6-2. Time-distance plot of data obtained during seismic refraction survey of a two-layer case with horizontal boundaries. The inverses of the slopes of the lines are equal to the seismic wave velocities in the two materials. US Army Corps of Engineers (1979)

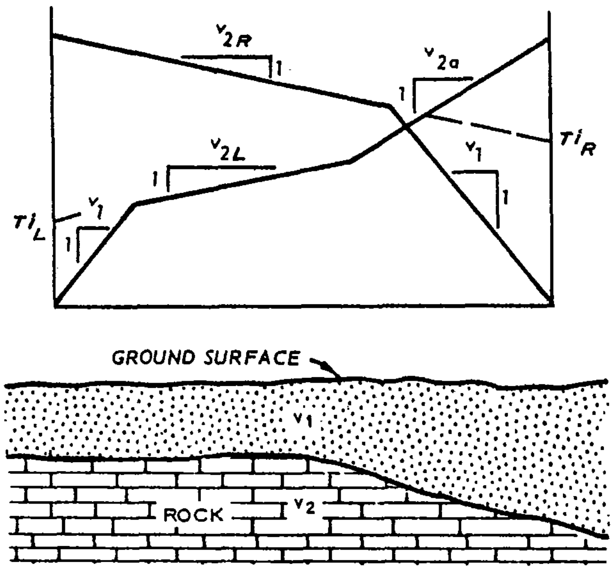

Figure 6-3. Time-distance plots of data obtained during seismic refraction surveys run in opposite directions above a two-layer system with a dipping boundary. US Army Corps of Engineers (1979)

travelled through both layers 1 and 2. The curve to the left of Xc represents the seismic wave velocity in layer 1 while the curve to the right of Xc represents the seismic wave velocity in layer 2. The curve to the right of Xc is flatter than the curve to the left of Xc, indicating that the seismic wave velocity in layer 2 is greater than in layer 1.

The depth (D) to the interface between layers 1 and 2 can be calculated using the equation:

$$D = \frac{Xc}{2}\sqrt{\frac{V_1 - V_2}{V_2 + V_1}}$$

where (Xc) is the critical distance and (V_1) and (V_2) are the seismic wave velocities in layers 1 and 2, respectively. An alternative method for calculating the depth to layer 2 is to use the intercept time, Ti, indicated on Figure 6-2, which is the intercept of the curve to the right of Xc with the time axis of the time-distance plot. In this case:

$$D = \frac{Ti}{2}, \frac{V_2 V_1}{\sqrt{V_2^2 - V_1^2}}$$

If the interface between two layers is not horizontal, the time-distance plots of refraction surveys run in opposite directions and will be different, as illustrated in Figure 6-3. The critical distance of the refraction survey advanced from the left to the right is less than the cirtical distance of the survey run in the opposite direction. This indicates that the depth to the interface between materials 1 and 2 is less on the left side of the profile than it is on the right side of the profile. The depths to the interface on either side of the profile are given by the equations:

$$D_L = \frac{V_2 Ti_L}{2\cos\theta} \times \frac{1}{\sqrt{(V_2/V_1)^2 - 1}}$$

$$D_R = \frac{V_2 Ti_R}{2\cos\theta} \times \frac{1}{\sqrt{(V_2/V_1)^2 - 1}}$$

where (D_L) and (D_R) are the depths to the interface on the left and right sides of the profile, respectively. V_2 and θ are given by the equations:

$$V_2 = 2(\cos\theta)\frac{V_{2L} \cdot V_{2R}}{V_{2L} + V_{2R}}$$

$$\theta = ½\left[\sin^{-1}\left(\frac{V_1}{V_{2L}}\right) - \sin^{-1}\left(\frac{V_1}{V_{2R}}\right)\right]$$

where (θ) is the slope of the interface and (V_1), (V_{2L}), and (V_{2R}) are as indicated in Figure 6-3.

The time-distance plots of data obtained from running seismic refraction surveys across a vertical subsurface interface (scarp) are shown in Figure 6-4. The curves are offset because of the steepness of the topographic break on the rock surface.

Profiles consisting of more than two layers may be analyzed using seismic refraction methods. The seismic wave velocities in the layers and the thicknesses of the layers may be determined from time-distance plots using equations which are more involved than those already presented. Detailed discussions of multiple layer systems are given in Dobrin (1976), Mooney (1977), and U.S. Army Corps of Engineers (1979).

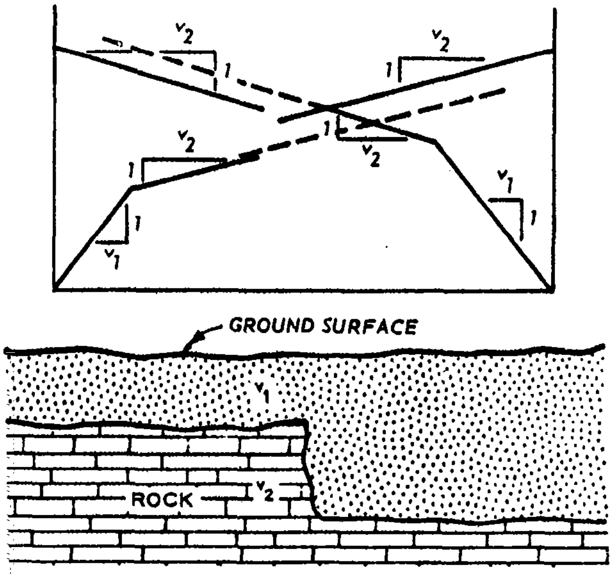

Figure 6-4. Time-distance plots of data obtained during seismic refraction surveys run in opposite directions across a two-layer system with a sharp vertical discontinuity in the boundary. The discontinuity may represent a sharp break in the bedrock topography or a subsurface fault scarp. US Army Corps of Engineers (1979)

6.5.1.1 Field Methods. When planning seismic refraction surveys, the source of mechanical energy, spacing of geophones, and direction of survey lines must be tailord to the geology of the site and to the information requirements of the survey.

Energy may be imparted to the ground by striking the ground with a sledge hammer, dropping a weight, or by setting off an explosion at or near the ground surface. Large-energy sources will result in deeper penetration of seismic waves and will allow for the analysis of geologic conditions at greater depths than if small energy sources are utilized.

The spacing of the geophones will be controlled by the depths and thicknesses of the geologic units along the survey lines. If individual subsurface layers are thin, small geophone spacings must be used in order to define the layers. If the subsurface layers are thick, wider geophone spacings can be used. The total length of the geophone lines should be at least as long as the depth of the deepest geologic unit of interest. Better results can be obtained if the geophone lines are at least three times the desired depth of penetration.

The method of performing a refraction survey will depend on the type of seismograph which is used. *Multiple channel* units consist of several (usually 6 to 24) geophones which are connected with a single seismograph. A single energy source is used and the travel times of the seismic waves from the shot point to the geophones are recorded by the seismograph. The field setup of a multiple channel unit will be similar to the arrangement of geophones shown in Figure 6-1.

A *single channel* unit consists of a single geophone which is connected to a seismograph. When using a single channel unit, the position of the geophone is kept constant while the distance between the geophone and the shot point is increased. The travel times of the seismic waves between the shot points and the geophone are recorded for each shot point location. The furthest distance between the geophone and the shot point should be at least as large as the depth of the deepest geologic unit of interest, and preferably three times as large as the desired depth of penetration.

Seismic surveys should always be run in opposite directions along a line so that dipping subsurface layers or breaks in bedrock topography can be detected. When using multiple-channel seismographs, the line can be reversed by moving the shot point from one end of the geophone line to the other. When using single-channel seismographs, the direction of the line can be reversed by moving the geophone to the position of the last shot point on the previous line and increasing the distance between the geophone and shot point in a direction opposite to that used in the previous line.

The geology of a site or transportation route should be considered when establishing seismic refraction survey lines. Survey lines should be oriented nearly perpendicular to the strike of major geologic structures (fault and fracture zones, folds, scarps on the bedrock surface) so that linear anamolies can be traced across parallel survey lines.

The geophones should have good contact with the ground. Thin layers of loose surficial material, organic material, and snow should be removed so that the geophones can be placed on firm ground. Likewise, the energy source should have good contact with the ground. A poor contact will result in loss of penetration depth of the seismic waves.

6.5.1.2 Characterization of Rock Type by Seismic Wave Velocity. Seismic wave velocities of materials beneath a survey line can be calculated from the time-distance plots of the field data and the materials can be tentatively identified on the basis of these velocities. Seismic wave velocities of materials encountered at a test site can be determined by running refraction surveys with short geophone lines across outcrops of the various materials. The first travel-time curve on the time-distance plots will indicate the seismic wave velocity in the outcropping material. Care must be taken to perform the calibration surveys at locations where the materials exposed at the surface have similar mechanical properties to the materials in the subsurface. The materials tested at the ground surface should be weathered to the same degree as the materials in the subsurface. The seismic wave velocities in subsurface materials can also be determined from data obtained from crosshole seismic surveys (discussed in section 6.10.3).

6.5.1.3 Limitations of Seismic Refraction Surveys. Certain conditions may result in ambiguous and/or incorrect interpretation of seismic refraction data. These conditions include:

- Insufficient density contrast at boundaries between layers
- Presence of low-density layer in the stratigraphic section
- Upper layer with a seismic wave velocity less than that of air
- Surface topography which is not level
- Shot point not at ground surface

Refraction of seismic waves will occur at the boundary of two materials only if there is a sufficient contrast in the density of the two materials at the boundary. Sufficient density contrasts will occur at the boundaries between lithologic units, at the groundwater level in granular materials, and at the top of fresh bedrock. Insufficient density contrasts may occur at the boundaries between units of similar lithology, and at the groundwater level in fine-grained materials.

In order for a subsurface layer to be detected by a seismic refraction survey, the seismic waves must be refracted upwards toward the interfaces of the layers. This will occur if the densities of successively deeper layers increase. If a low-density layer occurs in the section (a layer with a seismic wave velocity less than that of the overlying layer), the seismic waves will be refracted downward and away from the boundaries and a travel time curve for the low-density layer will not occur on the time-distance plot of the field data. Depths to individual layers, calculated from such a time-distance plot, would be incorrect since the thickness of the low-density layer would not be included in the depth calculations. The presence of low-density layers, as well as the depth and thickness of such layers, may be determined from boring logs and crosshole seismic surveys. The thickness data may be used to correct the depths calculated from time-distance plots.

Corrections must be made on the procedures presented in this chapter for calculating thickness and depth of subsurface layers if the upper soil or rock layers have seismic wave velocities which are less than that in air, if the surface topography over which a geophone line is spread is not level, and if the shot points are not at ground level. The necessary corrections are described in U.S. Army Corps of Engineer Manual EM 1110-1-1802 (1979).

6.5.2 Seismic Reflection Methods

Seismic waves which are reflected off material boundary interfaces can be recorded by a seismograph and used in seismic reflection analyses. On land, the reflected seismic wave arrives at the geophone after the refracted seismic wave, and the signal of the reflected wave is usually masked by the signal of the refracted wave (except if a seismograph which is specifically built for use in seismic reflection surveys is used). For this reason, reflection surveying is not useful for land sites. In aqueous environments, it may be difficult to place geophones on the firm sediment in lake, bay, or river bottoms, making seismic refraction surveys impractical. Seismic reflection surveys can be used successfully in water-covered environments and have been used to determine depth to bedrock, as well as to locate subbottom faulting, buried channels, buried sand and gravel deposits, and buried manmade objects.

Mechanical waves are generated at the water surface, pass through the water column, and are reflected off boundaries of density and/or velocity contrasts with subbottom materials. The density and velocity contrasts may be due to changes in the sediment type; or changes in the physical properties of a single layer due to changes in grain size, porosity, or density. The recorded display obtained during a seismic reflection survey will resemble a geologic cross section, showing bedrock (if present) and layers of sediment lying on top of bedrock. The recorded display indicates the travel time of seismic waves from the energy source to the receiver, and the apparent depths and thickness of layers on the display are not true representations of the actual depths and thicknesses of layers because of the differences in the seismic wave velocities in the subbottom materials. The actual thicknesses and depths of the layers can be calculated, knowing the seismic wave velocity in the materials, using equations given by Dix (1952).

Seismic reflection surveys are usually made from a boat equipped with an energy source and a receiver or *hydrophone.* Both are towed behind the point and the energy source is activated at intervals along the survey line. Energy sources include pingers, mechanical boomers, electrical sparkers, or pneumatic air guns, all of which are towed behind the survey boat; or dynamite, which is thrown from the survey boat.

Good control must be kept on the location of the survey boat and shot points. The direction of the boat may be controlled by visual, radio, or radar sighting of land-based range poles; and location of the shot points may be determined from the records of boat speed and times of energy source activation.

6.6 ELECTRICAL RESISTIVITY METHODS

Electrical resistivity methods utilize the differences in the electrical resistivities of different earth materials. Since the resistivity of earth materials are affected by mineralogy, porosity, degree of saturation, moisture content, and chemistry of the pore fluids, electrical resistivity surveys can be used to define subsurface layering, locate cavities and gravel pockets, and locate the groundwater table. For example, clays tend to have low resistivities because of the presence of exchangeable cations in the pore fluids while sands containing fresh water have high resistivities. The resistivity of an earth material usually decreases as the moisture content of the material increases. The resistivity of certain earth materials are given in Table 6-4.

An electrical resistivity array consists of four electrodes which are pushed into the ground. Two of the electrodes transmit an electrical current to the ground and the other two electrodes measure the voltage drop in the earth materials between the current electrodes (Figure 6-5). The resistivity of the earth materials can be calculated using a form of Ohm's Law. The resistivities which are calculated are apparent resistivities and not true resistivities. The apparent resistivities are average resistivities of all of the earth materials through which the electrical current flows. As the electrode spacing is increased, the electrical current flows through more material, and the apparent resistivities calculated from the field arrays are averages of the resistivities of more materials. Subsurface materials with unusually high or low electrical resistivities will result in anomalously high or low apparent resistivities. High resistivities will become detectable as the electrode spacing is increased and electrical current flows through the material.

Table 6-4.
Electrical Resistivity of Various Earth Materials Usually Encountered in Electrical Resistivity Surveys

Material	Resistivity, ohm-meters
Clay	1–20
Sand, wet to moist	20–200
Shale	1–500
Porous limestone	100–1,000
Dense limestone	1,000–1,000,000
Metamorphic rocks	50–1,000,000
Igneous rocks	100–1,000,000

US Army Corps of Engineers (1979)

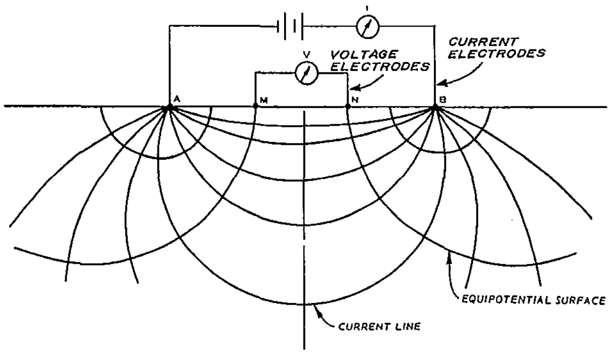

Figure 6-5. Schematic diagram of an electrical resistivity array indicating locations of current electrodes, voltage electrodes, and equipotential and current lines in the subsurface. US Army Corps Engineers, 1979

Three electrode configurations can be used in electrical resistivity surveys (Figure 6-6). The equations which can be used for calculating the apparent resistivity of earth materials are:

Schlumberger Array $\rho_a = \pi s^2 V/(aI)$

Wenner Array $\rho_a = 2\pi a V/I$

Lee Array $\rho_{aL} = 4\pi a V_L/I$
$\rho_{aR} = 4\pi a V_R/I$

where (ρ_a) is the apparent resistivity, (I) is the current flowing through the current electrodes, (V) is the voltage across the voltage electrodes, and (S) and (a) are the electrode spacings indicated in Figure 6-6.

Two types of resistivity surveys can be performed. *Depth Sounding* involves increasing the spacing between electrodes so that the apparent resistivities of earth materials at increasing depths are measured. The center part of the array is kept at the same location while the electrodes are moved further away from the center point for each sounding. The maximum electrode spacing should be 1 to 3 times the depth of the geologic unit which is being investigated. *Profiling* involves running a resistivity survey line while maintaining constant electrode spacings. This can be done with the Wenner array by taking one current electrode out of the ground and replacing it a distance beyond the other current electrode in the direction of the advancing survey line. The wiring to the electrodes must be changed so that the proper configuration of current and voltage electrodes exist. A profile survey will measure the apparent resistivities of earth

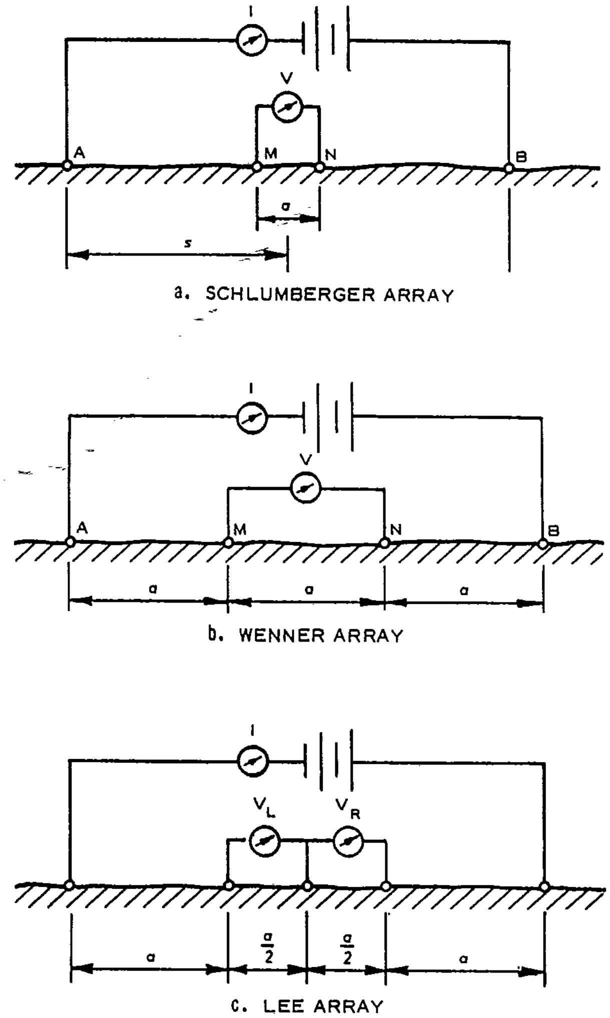

Figure 6-6. Three electrode configurations used in electrical resistivity surveying. US Army Corps of Engineers, 1979

materials along the survey line to a relatively constant depth. The most efficient way to identify a subsurface feature using electrical resistivity methods is to perform a depth sounding to locate the anomaly and then to perform a profile survey to delineate the anomaly, using the same electrode spacing at which the anomaly was discovered in the depth sounding.

Qualitative interpretations of resistivity data are made on the basis of discovering and delineating an anomaly in the apparent resistivity of the subsurface materials. Figure 6-7 illustrates different types of apparent resistivity anomalies. The earth model consists of three layers with resistivities of P_1, P_i, and P_n. For very small electrode spacings, all of the electrical current passes through layer 1 and the calculated apparent resistivities will be approximately equal to P_1. For very large electrode spacings, most of the electrical current will be passing through layer n and

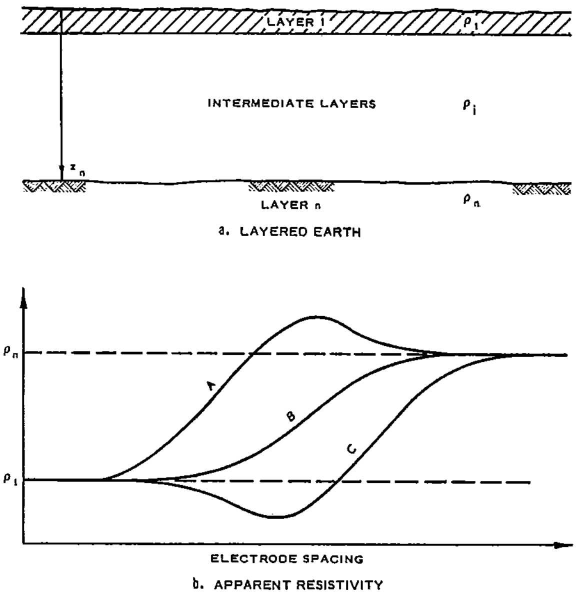

Figure 6-7. Apparent resistivity anomalies due to varying electrical resistivity of the intermediate layer of a three-layer system. US Army Corps of Engineers, 1979

the apparent resistivities will be approximately equal to P_n.

Curves A, B and C represent the effects of different resistivity conditions in the intermediate layers. Curve A represents the case where the intermediate layer has a higher resistivity than layer n. This may occur when an air-filled solution cavity, pocket of organic material, lense of clean sand or gravel, or igneous intrusion occurs in the intermediate layer. Curve C represents the case where the intermediate layer has a lower resistivity than layer n. This may occur if the groundwater level exists in the intermediate layer, or if a lense of saturated clay or zone of saturated fault gouge exists in the intermediate layer. Any anomaly in the apparent resistivity of subsurface material should be defined by borings made at the location of the anomaly.

Quantitative interpretation of electrical resistivity data involves curve matching or generation of theoretical resistivity curves by a computer. Quantitative interpretation is discussed in Orellana and Mooney (1966), Van Nostrand and Cook (1966), Dobrin (1976), and U.S. Army Corps of Engineers (1979).

6.7 GRAVITY METHODS

Gravity methods involve the measurement of anomalies in the gravitational field of the earth which are due to differences in the densities of materials in the subsurface. For example, an air-filled solution cavity would have a different density than surrounding rock units, which would result in a gravity anomaly; a trough or pinnacle in the bedrock surface would appear as an anomaly when compared to the gravity readings in surrounding terrain.

The gravitational field of the earth is measured in gals, where one gal = 1 cm/sec^2. The average gravitational field at sea level is 980 gals. Gravity anomalies are measured in terms of milligals, or approximately one-millionth of the earth's gravitational field. The resolution of a gravity survey depends on the sensitivity of the *gravimeter* and on the density contrast between different units in the subsurface. Modern gravimeters are capable of detecting differences in the gravitational field of 0.01 to 0.05 milligals. For example, a spherical cavity with a 1.2 m (4 ft.) radius with its center 1.5 m (5 ft.) below the ground surface in limestone with a density of 2.12 g/cm^3 would result in a gravity anomaly of (−)0.046 milligal (Fig. 6-8). This is barely within the range of precision of some modern gravimeters. The gravity anomaly produced by a salt dome with a diameter of 800 m (2600 ft.) buried at a depth of 600 m (1970 ft.) would be approximately (−)0.04 milligal.

The equation for the *Simple Bouguer Anomaly* is:

$$G_{OBS} \pm G_{FA} \pm G_B - G_{Theor} = C_{Anomaly}$$

where G_{OBS} is the value of the gravity field measured with the gravimeter, G_{FA} is the *free-air correction,* G_B is the *Bouger correction,* and $G_{Theor.}$ is the theoretical gravity calculated for the international reference earth ellipsoid using the Geodetic Reference System of 1967 formula:

$$G_{Theor.} = 978.0318\,(1 + 0.0053024 \sin^2\phi - 0.0000059 \sin^2\phi) \text{ gals.}$$

where (ϕ) is latitude in degrees. The free-air correction is made to account for the difference in elevation between the datum (mean sea level) and the data station, and is 0.3084 milligal/m (0.094 milligal/ft.) The free-air correction is added when the data station is above the datum and subtracted when the data station is below the datum. The Bouger correction is to correct for the gravitational attraction of the mass between the data station and the datum, and is 0.115 milligal/m (0.034 milligal/ft.) (assuming an average density of the mass of 2.67 g/cm^3). The Bouguer anomaly is subtracted when the data station is above the datum and added when the data station is below the datum (Figure 6-9). The free-air and Bouguer corrections may be added together for a total correction of 0.197 milligal/m (0.060 milligal/ft.) which is added when the data station is above the datum and subtracted when the data station is below the datum. Further corrections can be applied to the observed gravity to account for the gravitational effects of any surface or subsurface geologic features (mountain ranges, fault scarps, buried river valleys, solution cavities) which are near but not within the study site. Calculations for determining such corrections are given by Dobrin (1976). These types of terrain correc-

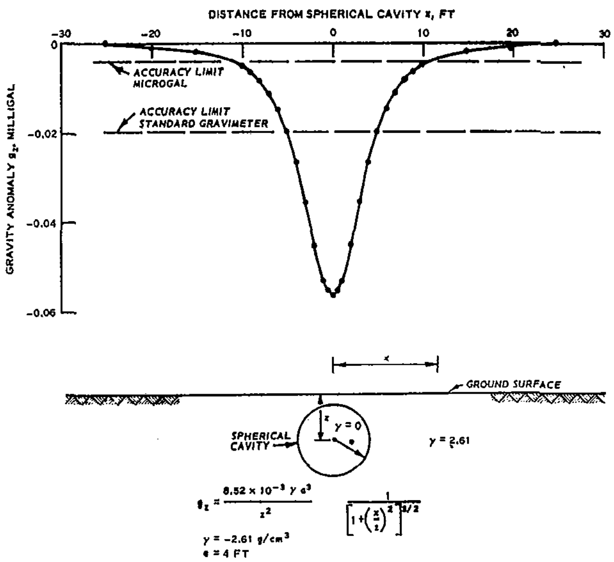

Figure 6-8. Gravity anomaly caused by a spherical cavity, with a 4-foot radius located five feet below the ground surface, in limestone with a density of 2.12 g/cm^3. US Army Corps of Engineers, 1979

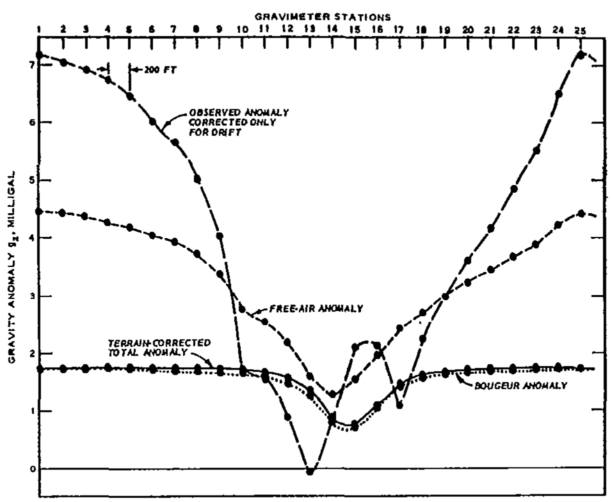

Figure 6-9. Theoretical gravity values, observed gravity values, free-air corrections, and Bouguer corrections used in calculating the Simple Bouguer Anomaly. US Army Corps of Engineers, 1979

tions do not have to be made if the topography of the study site and surrounding area is relatively flat.

6.7.1 Field Methods

Gravity readings are taken at data stations at which the elevation is known. Elevations may be determined from topographic maps or by surveying the elevations of the data stations. It is important to accurately determine the elevation of data stations since the combined free-air and Bouguer anomaly correction of 0.197 milligal/m (0.060 milligal/ft.) is approximately the same as the precision of some modern gravimeters.

Gravimeters give readings of gravity differences rather than absolute gravity, and therefore must be calibrated by making gravity readings at base stations where the absolute values of the earth's gravitational field are known. A system of calibrated data stations is maintained by the U.S. Army Corps of Engineers. Readings should be made at the base stations at least twice a day (at the beginning and end of the survey) and preferably three times a day. The observed base station gravity readings are plotted against time and a curve is fitted to provide time rates of instrument drift caused by short-term variations in the elastic properties of the deformation components of the gravimeter. The amount of instrument drift is a function of time, and for this reason the times at which readings are made at the data stations must be recorded so that the proper instrument drift corrections can be applied. Instrument drift corrections are subtracted from the calculated gravity anomalies if the slope of the correction curve is positive and are added if the slope of the correction curve is negative (Figure 6-10).

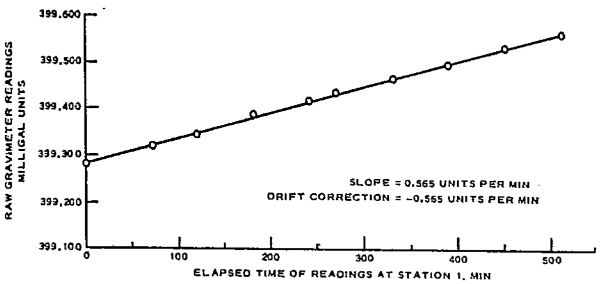

Figure 6-10. Instrument drift corrections. Data is obtained by taking readings at a calibrated base station two or more times in one day. Instrument drift corrections are subtracted from the calculated gravity anomalies if the slope of the correction curve is positive and are added if the slope of the correction curve is negative. US Army Corps of Engineers, 1979

Data stations should be spaced sufficiently close enough to each other to adequately define the subsurface conditions of interest. Surveys which require high resolution require closer spacing of data stations than surveys which require lower resolution. Spacing of data stations should be such that several gravity readings are made within the boundaries of the subsurface features of interest.

6.7.2 Interpretation of Gravity Data

Corrected values of the gravity anomalies can be plotted at the location of the data stations and contour to create a map of the Simple Bouguer anomaly. Traverse-type gravity surveys can be conducted across subsurface features of interest and the corrected values of the gravity anomalies can be plotted against distance to create cross-sections indicating the location of gravity anomalies (Figure 6-8). Subsurface structures can be determined from surface gravity data using methods given by Dobrin (1976).

6.8 MAGNETIC METHODS

Magnetic surveys measure changes in the magnitude in the total magnetic field of the earth which are due to the presence of earth materials which contain significant amounts of magnetite or hematite and therefore have high magnetic susceptibilities. Rocks which have unusually high magnetic susceptibilities include gabbro and basalt, as well as sedimentary or metamorphic rocks which contain significant amounts of magnetic minerals (magnetite, ilmenite, or hematite). The primary use of magnetic surveying is locating potential iron ore deposits, but magnetic surveying can also be used to locate basaltic igneous intrusions which may occur within or near fault and fracture zones, or which may bound structural basins in the bedrock topography.

Magnetic surveys may be performed with ground-based or airborne magnetometers. For engineering purposes, the most useful piece of equipment is the proton magnetometer. The proton magnetometer is a portable unit which can be operated by a single person. Readings are made at data stations along a survey line and the time of the readings are recorded so that the field data can be corrected for diurnal variations, in the earth's magnetic field. The initial data station should be reoccupied at the end of the work day so that closing error of the instrument can be determined. The closing error should be distributed evenly among the measured stations occupied during the work day. Data stations should not be located near any man-made object which can change the magnitude of the earth's magnetic field (power transmission lines, automobiles, metal pipelines and fences, structural steel in roads and buildings). The amount of magnetic material on the operator (belt buckles, keys, knives, compasses) should be minimized.

The data obtained during magnetic surveys can be presented as contour maps or as cross sections. There is no need to calculate the absolute magnitude of the earth's magnetic field to interpret magnetic data so that the data collected in the field (corrected for diurnal variation and instrument closing error) may be plotted directly on maps and cross sections. Linear magnetic anomalies on maps and cross-sections may be due to basaltic igneous intrusion, the presence of iron-rich rock units, or the juxtaposition by faulting of rock units with high and low magnetic susceptibility.

6.9 BOREHOLE LOGGING

Borehole logging can involve electric, radioactive, mechanical, and thermometric measurements which can be made in a borehole. Continuous borehole logs can be made by raising or lowering an investigative tool in the borehole, and measurements made by the tools are plotted as a function of depth in the borehole. Except for nuclear logging in boreholes with steel casings, it is impossible to obtain a meaningful log from a cased section of borehole.

Borehole logs can be calibrated by comparing them with cutting logs or description of cores taken from the borehole. Calibrated borehole logs can minimize the need for obtaining cores for detailed subsurface stratigraphic analysis. Distinctive "signatures" of subsurface units can be identified on the logs and correlated between holes to generate detailed stratigraphic cross sections of a site.

Borehole logs have been widely used in the petroleum industry for obtaining information from the deep subsurface. Borehole logging of shallow holes can be used to identify stratified sedimentary deposits such as sands, clays, and organic material; to identify rock units containing radioactive material; and to distinguish permeable sands from impermeable sands.

6.9.1 Electrical Methods

Electrical methods utilize measurements of the resistivity of subsurface material and the measurement of the spontaneous electrical potential established between drilling fluids and pore fluids.

6.9.1.1 Borehole Resistivity. Borehole resistivity methods work similarly to surface electrical surveys in that electrical current is imparted to the subsurface formations and the voltage drops across the formations are measured. The resistivities of the formation are calculated from the voltage drops. Two electrode configurations can be used (Figure 6-11). A normal

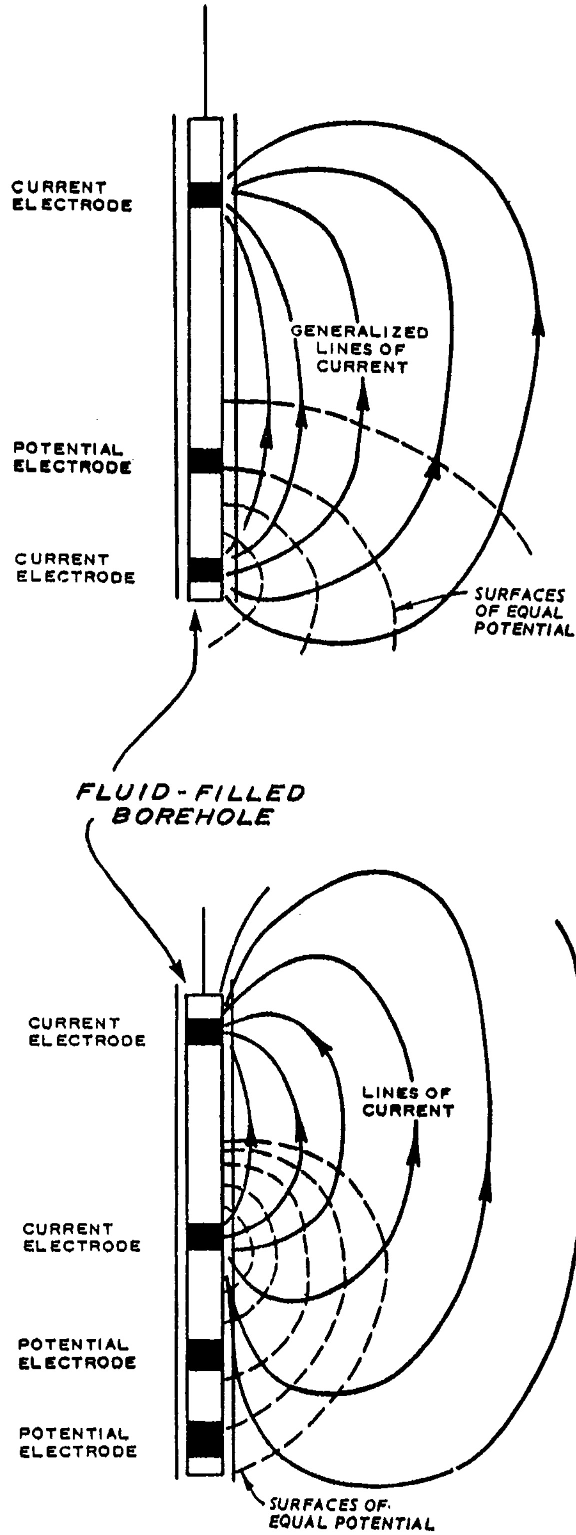

Figure 6-11. Electrode arrays for down-hole electrical resistivity logging. US Army Engineers, 1979

resistivity probe measures the magnitude of the electric field while a lateral resistivity probe measures the gradient of the electric field. Data from both probe configurations may be used to calculate electrical resistivities of subsurface materials.

6.9.1.2 Single-point Borehole Resistivity. Single-point borehole resistivity arrays consist of a single electrode in the borehole and a single electrode on the ground surface (Figure 6-12), both of which serve as current and potential electrodes. An electrical current is established between the electrodes and the voltage drop measured between the electrodes is used to calculate the electrical resistivities of the formations. The resistivities measured in this manner are average resistivities of all of the formations between the borehole and ground electrodes. As the borehole probe is raised or lowered past formations with high or low resistivities, the resistivity log will reflect the appropriate increase or decrease in apparent resistivity.

Geologic units which demonstrate high electrical resistivity include sand and sandstone, limestone and dolomite, and crystalline igneous and metamorphic rocks. Clay and shale units demonstrate low electrical resistivities as a result of the high concentration of mobile ions in such formations. Figure 6-13 illustrates the response of an electrical resistivity log to various lithologic units. The porosity, moisture content, and salinity of the pore fluids will also affect the electrical resistivity of a formation. Porous formations which are saturated will have lower resistivities than the same formations which are partly saturated. For this reason, the location of the groundwater table in a porous formation can be identified on an electrical resistivity log. However, the log interpreter must be careful not to interpret the log signature of the groundwater table as that of a lithologic contact. The resistivity of a formation will decrease as the salinity of the pore fluids increases so that electrical resistivity logs can be used to distinguish aquifers which contain fresh and saline water.

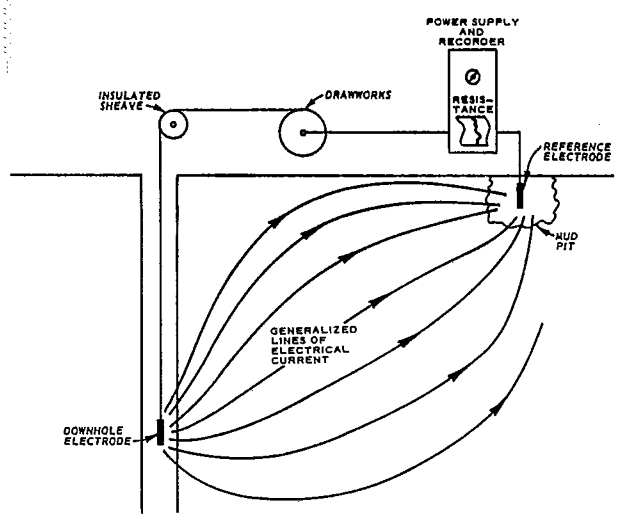

Figure 6-12. Single-point borehole resistivity array. US Army Engineers, 1979

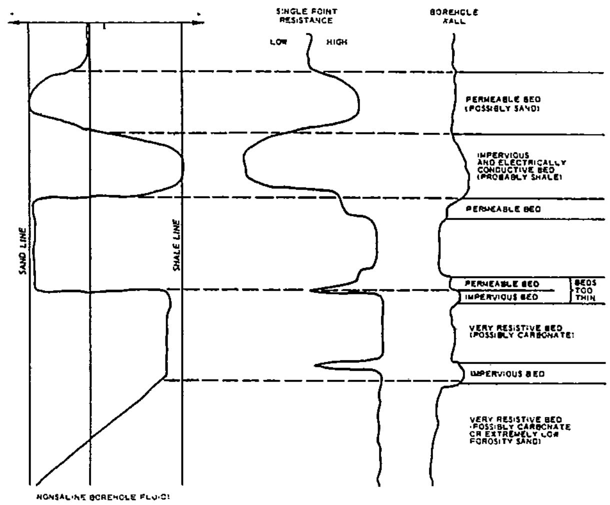

Figure 6-13. Response of borehole electrical resistivity log to subsurface units with various lithologies. US Army Engineers, 1979

6.9.1.3 Spontaneous Potential. Spontaneous potential methods measure the electrical potentials established between formation fluids and the drilling fluid, and the electrical potentials established at the boundaries of permeable subsurface layers. In both cases the electrical potential is due to differences in the concentrations of pore-fluid ions across formational boundaries or between the formation fluids and drilling fluids. If the drilling fluid is less saline than the formation fluid, spontaneous potential is usually negative when the probe is adjacent to permeable formations such as sand and positive when the probe is adjacent to impermeable formations such as clay and shale. However, in many boreholes drilled for engineering purposes, natural formation waters are used as the drilling fluids and concentration gradients between drilling fluids and formation fluids will not exist. The spontaneous potential curve in such a hole would be essentially flat and would yield little information on the properties of subsurface formations.

Electrical resistivity and spontaneous potential data are usually obtained by the same down-hole probe, and the logs are recorded simultaneously, as in Figure 6-13. The data on the two logs complement each other so that simultaneous interpretation of the logs will yield more precise data on the nature of the subsurface materials.

6.9.2 Nuclear Methods

Nuclear methods involve the measurement of natural gamma radiation in a formation, or the backscatter of radiation as the result of bombardment of the formation by gamma radiation or neutrons. The first method, *natural gamma,* provides data on the clay or uranium content of a material. The second method is *gamma-gamma* density and this provides data on the bulk density of subsurface materials. The third method, *neutron water detection,* measures the moisture content of a material. The natural gamma method can provide data on the lithology of a formation while the other two methods can provide information on physical properties of a formation.

Natural gamma radiation is emitted by radioactive potassium and uranium and a relatively high number of counts on a natural gamma log is an indication of an abundance of either element. Certain clay minerals, especially illite, vermiculite, and montmorillonite contain significant amounts of radioactive potassium in the interlayer positions of the crystal lattice so that clay and shale strata which contain these minerals will exhibit a large number of counts on a natural gamma log. Clean sandstones, limestones, and organic deposits do not contain as much radioactive potassium as clay and shale formations, and therefore will not register as many counts on a natural gamma log as clay and shale units. Igneous and metamorphic rocks may or may not contain significant amounts of radioactive potassium, depending upon the mineralogy of the rock.

Uranium minerals emit large amounts of gamma radiation, and rock units containing uranium minerals will, therefore, exhibit a large number of counts on a natural gamma log. This makes the gamma radiation log the primary tool used in uranium exploration. Significant deposits of uranium in sedimentary environments occur in sandstones where uranium minerals have precipitated in the pore spaces. For engineering purposes, the log intepreter must be able to distinguish between a clay formation and uranium-rich sandstone on a natural gamma log. This may be done by examining the log signatures of the formation on electrical resistivity and spontaneous potential logs and determining if the formation is a shale or sand unit.

The principles of the downhole gamma-gamma density measurement are illustrated in Figure 6-14. Gamma radiation emitted from the source enters the formation and passes through the electron shells of the atoms comprising the formation. As the gamma radiation passes through the electron shells, it is scattered. A certain amount of the backscattered radiation reaches the detector where it can be measured.

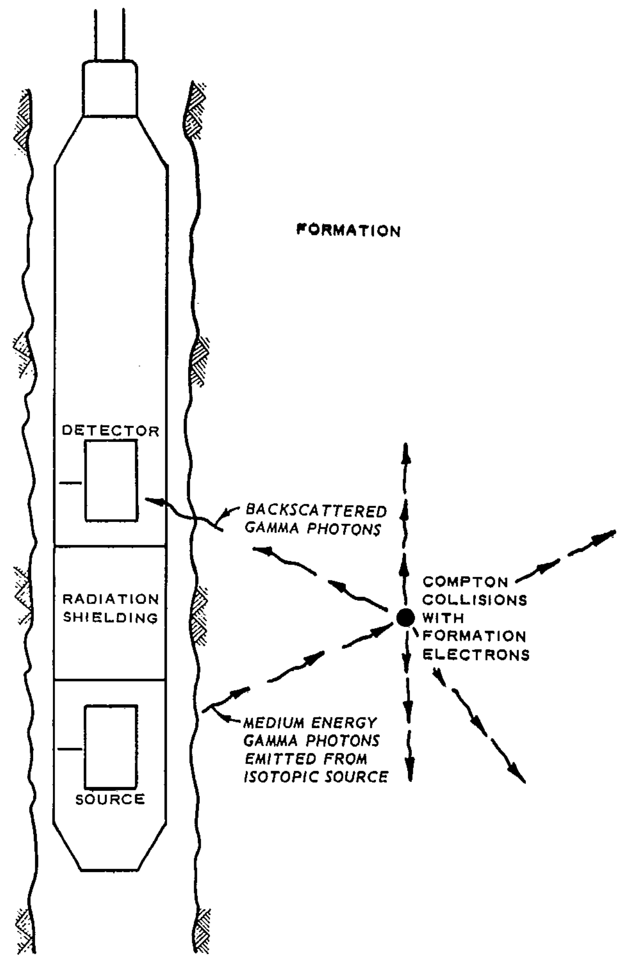

Figure 6-14. Schematic diagram of downhole gamma-gamma density probe and of the response of subsurface materials to bombardment by gamma radiation. US Army Engineers, 1979

The degree of gamma radiation backscatter is directly proportional to the density of electron shells in a material, which is in turn directly proportional to the density of neutrons and bulk density of a material. A large number of counts on a gamma radiation backscatter log is indicative of a material with high bulk density (Figure 6-15). This is not true for salt, gypsum, anhydrite, or other minerals where the ratio of electrons to atomic weight is different from that of silicate minerals.

The neutron water detection method works similarly to the gamma-gamma density method. Neutrons from the radioactive source in the probe enter the formation and collide with hydrogen nuclei. The neutrons are slowed down to a low kinetic energy range or are captured by hydrogen atoms, producing secondary neutron emissions and secondary gamma radiation. A detector which can measure the slowed-down neutrons, secondary neutrons, or gamma radia-

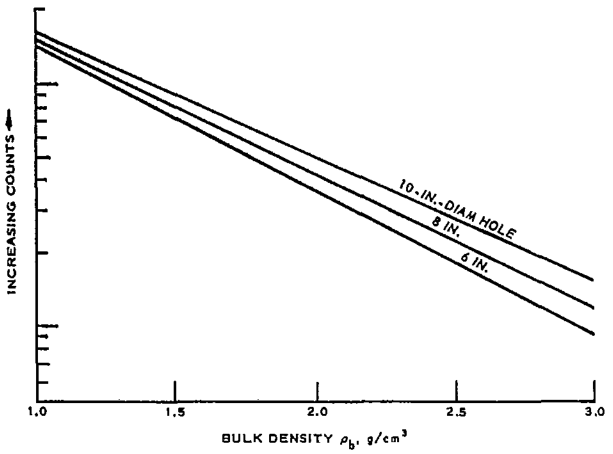

Figure 6-15. Amount of backscattered gamma radiation as a function of bulk density of the irradiated material. US Army Engineers, 1979

tion will provide data which can be used to determine the hydrogen content of a subsurface material. In the geologic environment, hydrogen exists most commonly in water and in hydrocarbons. If it can be assumed that hydrocarbons do not exist in the formations being investigated, an increasing number of counts on the neutron water detection log indicates increasing water contents of the materials (Figure 6-16).

The downhole probes used in both of the radiation backscatter detection methods must be calibrated. The gamma-gamma density device must be calibrated and standardized using materials of known density. Some materials, such as gypsum, require a special calibration different from that of silicates. The neutron water detection device must be calibrated in terms of its response to saturated geologic formations

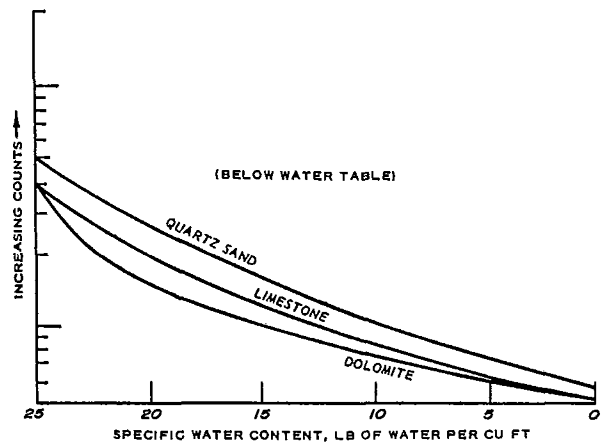

Figure 6-16. Amount of backscattered gamma radiation as a function of water content of the irradiated material. US Army Engineers, 1979

of various porosities. The neutron water detection device will detect all hydrogen in its sphere of investigation. Formations containing hydrocarbons will respond in the same way as formations containing water. The water existing in the crystal lattices of clay minerals or in hydrated minerals such as gypsum will be detected, and the moisture contents of formations containing these minerals determined from neutron logs will be different from moisture contents determined for laboratory samples. Details of calibration procedures are given in U.S. Army Corps of Engineers (1979).

6.9.3 Sonic Methods

Two methods exist for the downhole sonic investigation of boreholes; the *sonic borehole imagery* method and the *continuous sonic velocity* method. In the sonic borehole imagery method, pulses of high-frequency sound are emitted from a transducer and are reflected from the high-impedence surfaces of the borehole wall. The transducer is rotated about the central axis of the borehole and the detected reflected waves are transformed into electrical pulses which are used to present an image of the borehole wall on a cathode ray tube (Figure 6-17). The sound waves are emitted in pulses and the reflected waves can be detected in

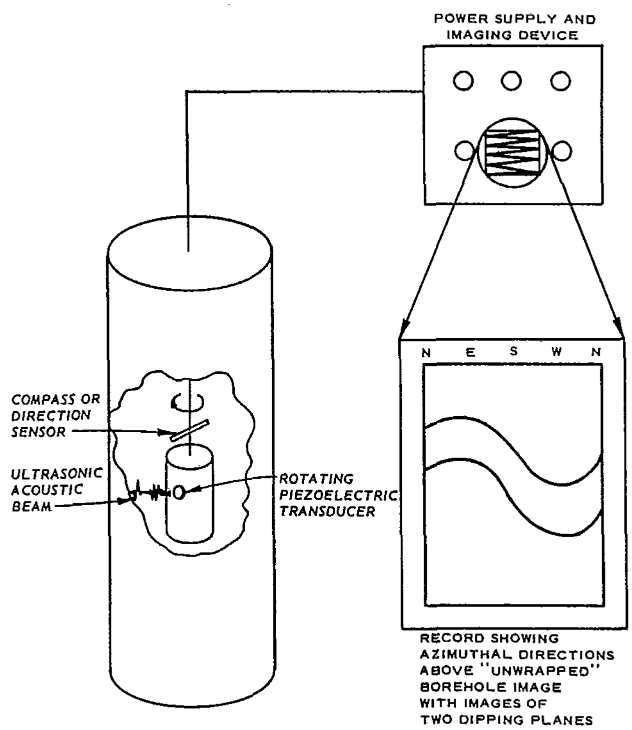

Figure 6-17. Schematic diagram illustrating mechanics of the sonic borehole imagery method. US Army Engineers, 1979

the time between the pulses. If the reflected waves do not arrive at the transducer at the appropriate time, or if the energy has been diminished, the electrical signal arriving at the cathode ray tube will be small or nonexistent. Several conditions can lead to the weakening of the reflected waves. A recess in the borehole wall at a joint intersection or a prominence in the borehole wall at a resistant strata, will cause the reflected wave to arrive at the transducer too late or too early. The presence of soft material such as fault gorge or joint infilling will diminish the energy of the reflected waves. Two restrictions on this method may make it impractical for shallow geophysical exploration. First, the borehole must be filled with fluid to allow for the efficient transmission and reflection of acoustic waves to and from the borehole wall. Second, the method will not work well in soil borings since soil does not form a high-impedence reflection boundary. The method can be used to examine the condition of borehole casing since cracks and buckles in the casing will show up as dark or dim spots on the cathode ray tube. Details of the sonic borehole imagery method are provided by U.S. Army Corps of Engineers (1979).

The continuous sonic velocity method measures the travel times of seismic waves along the borehole wall between two transducers on a tool that is lowered or raised in the hole. The travel times can be used to calculate the seismic wave velocities of the rock and soil units parallel to the axis of the borehole. Seismic wave energy emitted from a transducer enters the borehole fluid and reaches the borehole wall where it is refracted towards the wall, much as the waves in a seismic refraction survey are refracted towards the layer boundaries. If the seismic velocity in the borehole wall rock is sufficiently higher than in the borehole fluid, and if the path of the refracted wave will be travelled in less time than the path of the direct wave in the fluid, the refracted wave will be the first arrival at the detector. Differences in the seismic wave velocities of different formations can be distinguished on logs of the first arrival delay times. Formations with low seismic wave velocities will have longer delay times than formations with higher seismic wave velocities. The seismic wave velocity in a material will indicate something about its density.

6.9.4 Mechanical Methods

Caliper or *borehole diameter logs* are logs of the mechanically or acoustically measured diameter of the borehole and represent one of the most useful and simplest techniques used in borehole geophysics. Mechanical calipers consist of a downhole probe with one to six or more feeler arms which come into contact with the borehole walls and can detect irregularities on the walls as the probe is pulled up the hole. Changes in the orientation of the feelers are translated into electrical signals which can be transmitted through a cable to a strip chart recorder. Mechanical calipers may be used in holes filled with water, mud, or air. Acoustic calipers consist of a probe usually containing four transducers which emit acoustic waves and measure the travel times of the waves reflected from the borehole walls. Travel times are recorded on a strip chart recorder and borehole diameters can be calculated if the seismic wave velocity in the borehole fluid is known. Acoustic calipers must be used in boreholes which are filled with water or mud. Both the mechanical and acoustic calipers must be calibrated. Scaled templates, hollow tubes, or cylinder rings of various diameters can be used in the field.

Caliper logs can provide data on rock quality by identifying layers of different hardness, fracture frequency, and cementation, all of which affect borehole diameter. They can also be used to identify zones of swelling or washout; the latter application is useful since it is hard to obtain cores or samples from geologic units which are easily washed out during drilling. Caliper logs can be used to identify porous zones in a boring by locating intervals in which excessive mud filter cake has built up on the borehole walls. One of the major uses of caliper logs is to provide information by which other geophysically derived logs can be corrected for borehole diameter effects.

6.9.5 Thermometric Methods

Temperature logs are continuous records of the temperature of the fluid at successive elevations in a borehole. The temperature of the borehole fluid is measured by a probe containing one or more thermistors. Borehole temperature measurements can be used to locate aquifers in the borehole and to determine the direction of water movement in the borehole. Drilling and testing of a borehole can disturb the thermal environment in a borehole and therefore, it is desirable to leave the boring to be logged undisturbed for several days after completion to allow the borehole fluids to return to ambient temperatures.

6.9.6 General Field Methods

In all borehole geophysical methods the resolution of the measurements are inversely proportional to the spacings between transducers and detectors on the probes and to the rate at which the probes are raised or lowered in the borehole. The rate at which a probe is raised or lowered must be tailored in each application to the desired resolution and to the time which can be spent in logging the boreholes. It is necessary to reference the depths on the borehole logs to the elevation of the top or bottom of the borehole so that features indicated on geophysical logs, cutting logs, cores, and samples can be correlated.

6.9.7 Interpretation of Borehole Logs

Interpretation based on only one type of borehole log can often lead to ambiguous or incorrect interpretations of subsurface conditions. Borehole logs are most useful when two or more logs are run simultaneously so that interpretations can be based on several sets of data. For example, electrical resistivity, natural gamma, and gamma-gamma density logs can be interpreted simultaneously to identify sand and shale layers in the subsurface.

6.10 DYNAMIC PROPERTY MEASUREMENTS

Dynamic properties of earth material can be calculated if the velocities of compressional and shear waves in the materials are known, as well as the saturated bulk density. Saturated bulk density can be determined on laboratory samples or can be estimated from calibrated gamma-gamma density logs. Compressional and shear wave velocities can be determined from uphole, downhole, or crosshole seismic surveys. All three methods involve imparting mechanical wave energy into earth materials surrounding the borehole and measuring the travel times of the waves from the energy source to detectors in the same borehole or adjacent boreholes.

6.10.1 Uphole Survey

In an uphole survey the energy source is in the borehole and the detector is on the ground surface. Seismic wave energy is imparted to the subsurface materials at different intervals as the energy source is raised in the borehole. The data collected during an uphole survey are the same as those collected during a seismic refraction survey: distance between energy source and detector and wave travel times between the two. The slopes of the curves on the plots of travel time versus depth indicate the average seismic wave velocities in the various intervals (Figure 6-18).

The velocities of both shear and compressional waves need to be determined for dynamic property calculations. Shear or S-waves travel slower than compressional or P-waves so that S-waves will arrive at the detector after the P-waves (Figure 6-19). The P-wave signal often masks the S-wave signal so that it is difficult to determine the arrival time of the S-waves. In order to facilitate the determination of the S-wave

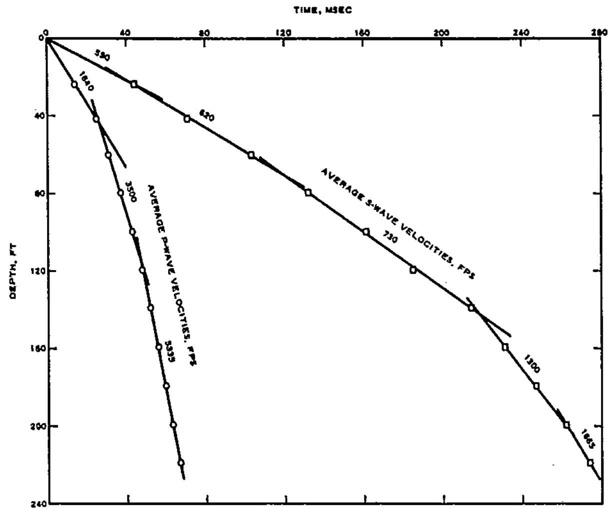

Figure 6-18. Time-depth plots of data obtained during uphole and downhole seismic surveys. The slopes of the curves indicate the average shear and compressional wave velocities in the subsurface materials. US Army Engineers, 1979

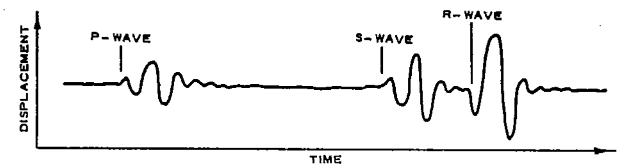

Figure 6-19. Idealized plot of P- and S-wave arrival times at a geophone. US Army Engineers, 1979

arrivals, energy sources rich in shear wave energy should be used in conjunction with seismographs which are capable of detecting and enhancing the S-wave signal.

A Meissner wave-front survey is a modification of the uphole survey in that the geophones are placed on the ground surface in a line extending away from the borehole (Figure 6-20). The method is not well suited for measuring S-wave arrivals. Energy is released at regular intervals in the borehole starting from the bottom and working upward. A geophone recording is made for each energy release and a P-wave arrival time is determined for each energy release at each geophone location. The P-wave arrival times are plotted on a grid at points whose coordinates represent the horizontal distance of the geophone from the borehole and the depth of the shot. The arrival times can then be contoured to show areas of similar arrival times. Figure 6-21 is such a contoured plot showing the interface of strata and indicating the seismic wave velocities in the strata. Limitations on interpreting the

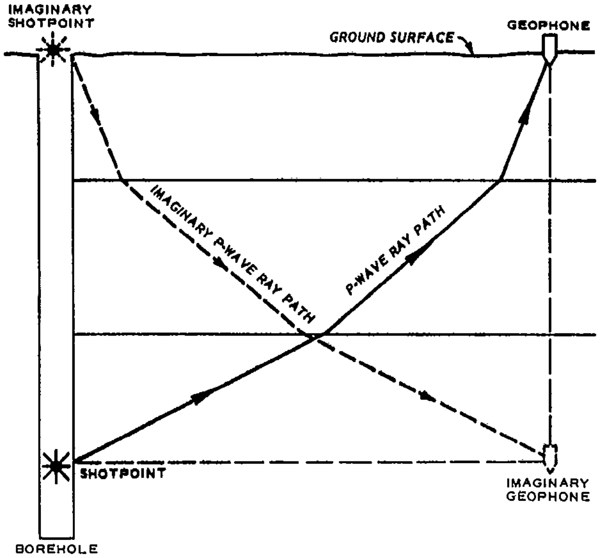

Figure 6-20. Schematic diagram of geophone array used in a Meissner wave-front survey. US Army Engineers, 1979

contoured plot of arrival times are that the wave-front analogy breaks down in areas of dipping geologic units or extreme topography, and that the contoured arrival time points represent the behavior of the waves only along the ray paths and not in the materials directly below the geophones.

6.10.2 Downhole Survey

The downhole survey is similar to the uphole survey except that the energy source is placed at or near the ground surface close to the borehole and detectors are located at one or more depths in the borehole. Wave arrival times can be plotted against depth and seismic wave velocities can be calculated just as in the uphole survey. Energy sources can be either explosive or mechanical and seismographs should be capable of discerning S-wave arrivals. Additional details of the uphole and downhole survey methods are given in Viskne (1976) and U.S. Army Corps of Engineers (1979).

6.10.3 Crosshole Survey

Crosshole surveys are conducted to determine the velocities of P- and S-waves in layers of earth material by measuring the travel times of seismic waves that have traveled from an energy source in one borehole to a detector in another borehole (Figure 6-22). As in uphole and downhole surveying, equipment which is capable of distinguishing S-waves from P-waves must be used. The seismic wave velocities determined in a crosshole survey will be apparent velocities if the boreholes are sufficiently far apart to allow a seismic wave refracted at the boundary of two layers to arrive at the detector before the direct wave. Crosshole sur-

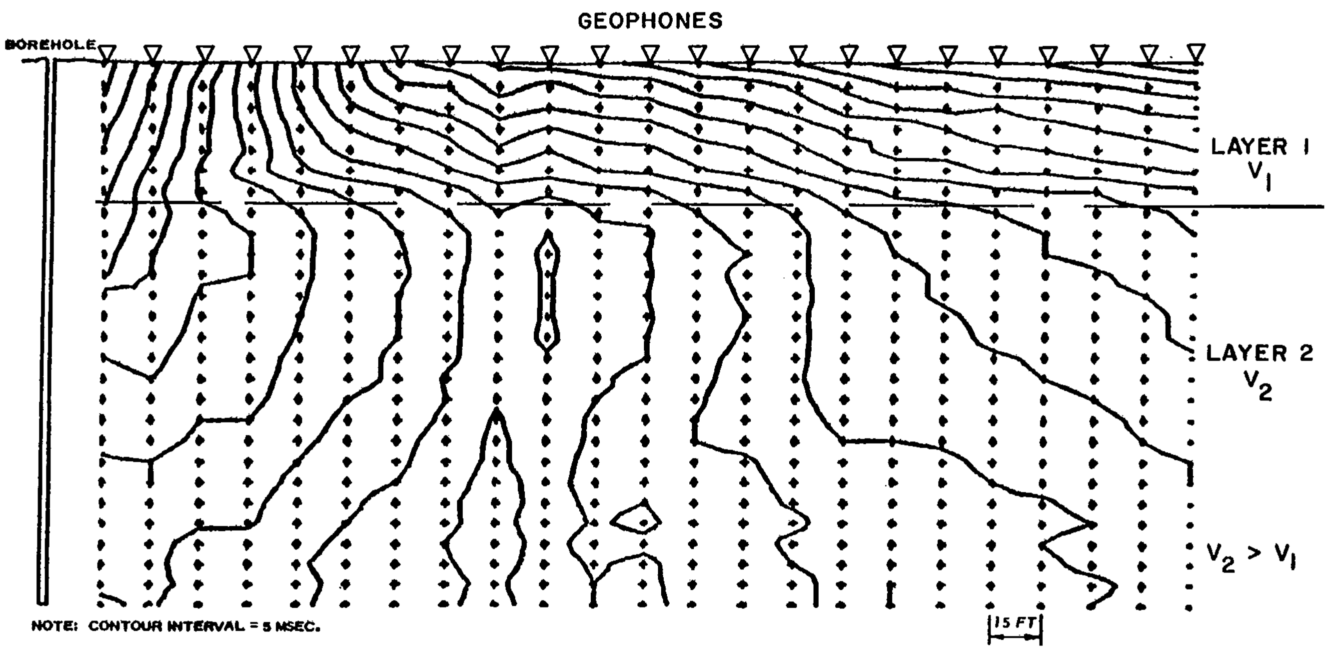

Figure 6-21. Contoured plot of seismic wave arrival times obtained from a Meissner wave-front survey. Two strata can be identified; the first one where the contour lines are close together, and the second where the contour lines are further apart. The boundary between the two strata is horizontal. US Army Engineers, 1979

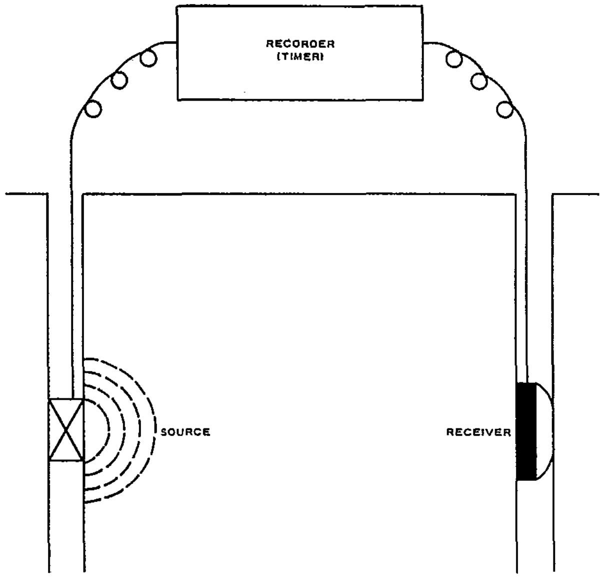

Figure 6-22. Schematic diagram illustrating the relationship of seismic wave source to receiver in a crosshole seismic survey. US Army Engineers, 1979

veys can be used to investigate the changes in seismic wave velocity with depth in a borehole and to identify any subsurface layers which cannot be detected by surface seismic refraction methods because of low seismic wave velocities in the layer. The elevations of the energy source and detectors must be known and well controlled so that the depths and thicknesses of subsurface layers, as well as distances between energy source and detector, can be accurately determined. Additional details of crosshole survey methods can be found in Viskne (1976) and U.S. Army Corps of Engineers (1979).

6.11 SUBAUDIBLE ROCK NOISE

Subaudible rock noise can be detected using digital event microrecorders. Subaudible rock noise is indicative of rock fracturing and increases in rock noise may precede slope failures and slabbing or rock bursts in tunnels as the results of changing stress conditions due to rock creep or during construction. The level of rock noise can be monitored hourly or daily and can be graphically displayed by plotting noise level as a function of time. An increase in the level of rock noise may indicate an impending failure, and at that time personnel and equipment may be moved away from the potential failure area or remedial measures may be taken to prevent the failure. The level of rock noise prior to failure of a particular material may be determined in the laboratory by measuring noise levels in a sample which is tested to failure in uniaxial or triaxial compression.

6.12 BOREHOLE T.V. CAMERAS

Television cameras lowered into boreholes can be used to visually inspect the conditions of the borehole walls and to make videotape records of observations for later analysis. Visual inspections of the boreholes can be used to distinguish gross changes in lithology and to identify fracture zones, shear zones, or zones of joint intersections. The depth and extent of lithologic units and fracture zones can be determined as the camera is raised in the borehole. The use of borehole television cameras requires that the borehole be dry or filled with clear water. Boreholes filled with drilling mud must be flushed with clear water before a borehole T.V. camera can be used.

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7.0 SUBSURFACE EXPLORATION (SOIL AND ROCK SAMPLING)

The geotechnical parameters which will affect design and construction of the transportation facility must be investigated and evaluated. A specific, well-integrated and flexible subsurface exploration program, generally conducted in several phases, is necessary to develop the maximum amount of geotechnical data for reasonable costs.

The increasing demand for more detailed geotechnical information has initiated extensive and costly subsurface exploration programs. Although the additional information will generally decrease the potential unknowns and construction risks, a balance must be maintained between the cost of the exploration program and the level of information which will be produced.

A review of subsurface exploration programs in the United States, associated with highway construction, shows that no standard approach of methodology has been adopted (Ash, 1974). The differences in approach are due primarily to widely divergent geological environments, variations in the complexity of the geology, available site area information, local equipment, personal preferences and habits of the investigators, and time and budget constraints.

Subsurface exploration procedures cannot be reduced to simple guidelines to fit all existing conditions. Each project must be evaluated according to its specific geological conditions and the type of proposed facility. Of primary importance in any subsurface exploration program is the individual selected to direct the investigation, interpret the information, and present the conclusions in a concise and usable form to those responsible for design and construction.

This section of the Manual outlines various planning and contractual procedures, and describes drilling equipment and sampling and logging methods which may be utilized in subsurface exploration programs. *In situ* borehole field tests (Appendix B), geophysical methods (Section 6), geohydrologic determinations (Section 8), and instrumentation installations (Appendix G) should be incorporated, where applicable, into the overall program and be obtained, when possible in conjunction with the actual drilling of the test borings.

Selected drilling and sampling techniques and typical field data collection forms are summarized in Appendix A.

7.1 GENERAL PLANNING

The purpose of the "Office Reconnaissance", discussed under Section 4, is to obtain and evaluate as much information as possible about the project area in the early planning stages. This information is then utilized in developing a subsurface exploration program which will determine the characteristics of the materials and structures in the ground to the degree necessary for the location, design and construction of the transportation facility.

A staged approached consisting of two or more phases of field explorations will allow the engineer to develop a design in an orderly manner, based on evaluation of information that may affect the choice of alignment and structures and the identification of special problem areas.

A preliminary exploration program generally consists of widely spaced test borings which may be supplemented with geophysical surveys, to define the principal geological parameters to be evaluated during initial project planning and cost estimating. Although the level of importance of the various parameters will vary with the project type and location, preliminary information should be obtained regarding the following subsurface conditions during early stages of design:

1. Soil and Rock Stratigraphy
2. Hydrological Conditions
3. Soil Classification, Density and Consistency
4. Rock Quality and Discontinuities
5. "Mixed Face" Conditions

6. Obstructions (such as boulders)
7. Hazards (such as methane gas)

Information evaluated during the preliminary exploration phase will assist the planners in final route selection which may then be more thoroughly investigated with detailed subsurface explorations, *in situ* and laboratory testing and geophysical surveying.

After the selection of the final route location, preliminary design analysis and detailed evaluation of the project parameters, supplementary explorations and testing may be required to further define:

1. Specific engineering characteristics of problem soil or rock conditions.
2. Adverse subsurface conditions which would necessitate special design and construction considerations.
3. Final design criteria for foundations and pavements, dewatering, temporary and permanent support of excavations, shafts, earth pressures, and other items required by the designers.

It may also be necessary that additional subsurface information be obtained during actual construction to allow the engineer and the contractor to further evaluate and modify, if required, the design and construction procedures.

7.2 MANAGEMENT AND SUPERVISION

The subsurface exploration program which is formulated during the preliminary phases of the investigation serves as a basis for planning day-to-day operations. However, it should be considered as a flexible plan and subject to continuous modification as information is received from the field. The base model conditions determined from preliminary investigations, research and reconnaissance should be continually reviewed and updated as additional data are obtained during the exploration program.

The program should be supervised by qualified and experienced personnel. They should be briefed and familiarized with all project requirements, including their specific responsibilities and on-site safety. Close liaison between the design staff and the contractor conducting the work is essential in realizing the project goals. Frequent communication and site visits between the various parties will minimize misunderstandings and unnecessary costs. The regular submission of boring logs, test data and daily reports will provide the basis for continuous review and modifications, if required.

It is desirable that one individual be selected to serve as the project geologist or geotechnical engineer throughout the stages of the exploration program.

7.3 CONTRACTS AND SPECIFICATIONS

Well defined contract management procedures are necessary to insure timely and cost-efficient completion of the field exploration program.

Many state agencies have their own in-house test boring capabilities and, therefore, do not require formal contracts or technical specifications for conducting the work. However, although project requirements and geological conditions may vary considerably within a given state, consideration should be given to some form of instruction or specification that expresses the design engineer's needs and requirements.

Contractor selection based on expertise, equipment and availability will generally provide a major benefit in more reasonable unit pricing, in addition to allowing more flexibility during the execution of the work. However, major exploration contracts which are conducted for governmental agencies, at all levels, usually require public advertising and competitive bidding.

Contracts and technical specifications will vary considerably with the agency or personnel involved. Specific contractual details are beyond the scope of this manual. However, a complete set of contract documents for a subsurface exploration program should include the following:

- Invitation to bid
- Proposal
- Contract Agreement
- General Conditions
- Technical Specifications

7.3.1 Invitation to Bid

This document is issued as a letter to prospective bidders (those requesting bids, or firms identified by the project geologist or geotechnical engineer). The invitation contains a Scope of Work developed by the project geologist or geotechnical engineer in consultation with the project design team and addresses such requirements as scheduling, complexity of the exploration program, budget considerations and method of payment. The Invitation to Bid may contain a Bid Sheet, listing all items of consideration, as a basis for unit-price bidding.

7.3.2 Proposal

The Proposal is a letter response sent to the contracting agency from a prospective bidder. It stipulates in very general language what the bidder will do, how he will accomplish it and how much he will charge. In general, it states his agreement to perform the work in accordance with the contract documents and includes itemized unit prices of each item on the Bid Sheet, the extended total cost of each item, and the total or gross sum bid.

7.3.3 Contract Agreement

The Contract Agreement is the document which, when properly signed and witnessed, binds both parties to a firm contract.

7.3.4 General Conditions

The General Conditions section is that part of the contract documents in which:

- Rights and restrictions of the various parties involved in the contracts are defined and listed.
- Measures required for the protection of others are defined and listed, including provisions required by statute law.
- Requirements as to bonds, insurance, special licenses and permits are stated, along with such other legal or business matters as are pertinent to work (e.g., progress, time of completion, liquidated damages, and subcontracting.

7.3.5 Technical Specifications

The Technical Specifications describe in detail the nature of the work and the manner in which it is to be performed. Included are such items as number and type of borings, sampling procedure, instructions for the performance of special tests, preservation and transportation of samples, submission of reports, and the manner of measurement and payment for the various items.

7.3.6 Contract Award and Implementation

Upon completion of a specified amount of time, the completed contract documents are received by the contracting agency, opened and publicly read.

The contract documents are then reviewed for content and errors and the recommended contractor (usually, but not necessarily, the low bidder) will be forwarded to the reviewing authority for approval.

Upon selection of the contractor, a pre-exploration meeting is held with all interested parties. The specification requirements are reviewed with the contractor and any outstanding questions or problems are resolved before the contracting parties affix their signatures to the Agreement.

After signing the Agreement, the contractor is given a Notice-to-Proceed. This letter usually restates the acceptance of the proposal, and indicates the latest acceptable starting and completion dates for the work. The "Notice" also re-states the level of effort (number of drill rigs) required and special requirements, such as obtaining rights-of-entry, permits to be obtained from governmental agencies, and restrictions in working hours or conditions.

7.4 EXPLORATION PROGRAM

The project geologist or geotechnical engineer, when formulating the subsurface exploration program, must carefully evaluate the variety of methods and procedures which are available, in order to maximize the amount of information obtained and minimize associated costs.

Subsurface exploration methods and procedures may vary along the proposed alignment depending on the geological conditions and environmental considerations which are encountered. Even with specialized equipment, there is the potential that representative *in situ* samples cannot be obtained or are disturbed during recovery or subsequent transportation. The diameters of the boreholdes are very small relative to the inter-borehole spacing; therefore, the subsurface conditions between the explorations are inferred and not specifically known. For roadway construction, "representative" sampling is often less than one part per million of the total volume of material involved in the project (Dumbleton, 1974).

Special care must be exercised by the project geologist or geotechnical engineer at this stage of design so that assumptions are not made which will lead to additional expense and unwarranted hazards during construction.

7.4.1 Exploration Plan

An initial exploration plan is prepared from the information evaluated during the preliminary investigation phases; it includes location, spacing and depths of explorations, and sample type and interval. There are many variables involved in the formulation of this plan, and it requires considerable experience to schedule enough boreholes without excessive or inadequate coverage. The objective should be the development of the maximum amount of subsurface information through the use of the minimum number of boreholes. The initial plan should be flexible and subject to modification and revision as to the exploration program proceeds.

Exploratory boreholes should be located with consideration to site topographic and geologic conditions and the proposed project layout and design, not at some arbitrarily chosen, fixed interval. They should be obtained in order of importance, both from the design and economic aspects, so that a maximum amount of information is obtained from a given expenditure of effort and funds.

General items which should be considered in determining the positions and priorities of the explorations include the following:

- Key locations to clarify the geological interpretation of the site as a whole.
- Key locations that could lead to relocation or redesign of the alignment.
- Bridges and other structures.
- Deep cuts and high embankments.
- Areas of engineering difficulty or complicated ground surface.
- Off-line investigations may be required for developing regional geological hazards or borrow surveys.
- Points of interpolation between those selected for priority; number and interval would be a function of the complexity of the geology.
- The less expensive methods of investigation should generally be used first. They may provide sufficient information themselves, and will indicate where more detailed and expensive investigations may be required.
- In areas of intense, complex or expensive construction activity, it may be necessary to conduct very sophisticated and expensive exploration methods which include horizontal borings, vertical inspection shafts or pilot tunnels.

The proposed boring location plan should always be checked against actual field conditions, prior to commencing the explorations, for any modifications or adjustments which may be required due to access or other restraints. This examination will also assist the project manager in finalizing the exploration and sampling methods, equipment selection, determination of property owners, and logistic considerations such as storage and supply areas, all of which must be determined and evaluated for conducting an efficient and economic subsurface exploration program.

7.4.2 Types of Borings

The following terminology is suggested for boring identification during the various phases of the investigation program.

7.4.2.1 Pilot Borings. Pilot borings are conducted during the preliminary or initial investigation stages of the project. These borings will be located at scattered, selected locations to obtain only sufficient information to enable the project manager to:

- Establish the preliminary alignment, profile and structure locations.
- Estimate the preliminary quantities of soil and rock items of construction involved in the project.

7.4.2.2 Control Borings. Control borings are the designated first-phase design borings which are obtained at selected and key locations. These are continually monitored by the project manager to determine if any modifications in the design exploration program are required.

7.4.2.3 Verification Borings. Verification borings are additional design borings which are scheduled following the analysis of the control borings.

7.4.3 Exploration Spacing

The locations of the explorations are subject to many variables and depend on the uniformity of the geological units and the type of facility proposed. If the subsurface conditions in the project area are well known, and the stratification is simple, with relatively thick individual strata of consistent physical properties, relatively widely-spaced explorations may be sufficient. If, however, erratic and rapidly changing conditions exist, more closely spaced explorations will be required. Structures which are sensitive to settlement or subjected to heavy loads also require more detailed subsurface information. The following subsections give more specific information and criteria regarding general exploration spacing.

The majority of test borings which are obtained in highway investigations are vertical. However, inclined borings may be used to advantage in exploring inclined strata and various subsurface irregularities. In addition, inclined borings may be used where surface obstructions such as rivers prevent vertical holes. Inclined boreholes furnish information in both a vertical and horizontal direction.

Borings for exploration of narrow rights-of-way should be staggered left and right of centerline. They should not be located in a single, straight line, except when spaced greater than about 150 m (500 ft.) apart. This will facilitate development of the strike and dip of the subsurface strata, and irregularities in the profile at right angles to the axis of the alignment. Selected borings may also be required outside the immediate alignment area to determine more regional geological hazards such as faults or land slides. In addition, these explorations may locate more suitable foundation conditions that would warrant a change in the preliminary alignments.

It may be preferable, in some instances, to locate the boring outside the limits of a proposed structure which is to be constructed within a cofferdam or caisson that is to be dewatered. The borehole may become a source of additional water inflow unless properly backfilled and sealed.

The term "subsurface explorations" usually implies test borings; however, a variety of exploration methods such as hand or machine excavated test pits and probings conducted at selected locations may meet the project requirements and minimize investigation costs.

Whenever responsibility for conduct of the exploration effort is delegated to others outside the Agency, it is appropriate to request a presentation by the consultant or subcontractor at which time the philosophy of depth, placement and sampling is mutually reviewed.

When the surficial conditions mask the underlying subsurface geological conditions, or when available information may be lacking for preliminary evaluation purposes, the following general guidelines may be used in establishing the preliminary locations of the subsurface explorations.

7.4.3.1 Subgrade Borings. In areas where relatively uniform subsurface conditions are anticipated and deep cuts or high embankments are not being considered, an average spacing of 60–90 M (200–300 ft.) will often be adequate. In certain sections of the United States, such as some parts of the Midwest, this average spacing may be increased up to 300 M (1,000 ft.).

As the degree of geological complexity increases, the average exploration spacing may be decreased to 30–60 M (100–200 ft.). Where highly erratic and critical foundation conditions exist, it may be necessary to further decrease the spacing to 8–15 M (25–50 ft.) between explorations.

7.4.3.2 High-Embankment and Deep Cut Borings. The average maximum boring interval for roadway embankments that will be greater than 5 M (15 ft.) in height is approximately 60 M (200 ft.). If erratic foundation conditions or compressible materials are encountered, this spacing may be decreased to 30 M (100 ft.). At least one boring may be located at the point of maximum height of the embankment.

The average maximum boring interval for single roadway cuts in excess of 5 M (15 ft.) deep is approximately 30 M (100 ft.). When the proposed cut accommodates more than one roadway, the interval may be increased to 60 M (200 ft.) for each roadway, but staggered so that the overall 30 M (100 ft.) spacing is maintained. At least one boring may be located at the maximum depth of the proposed cut.

7.4.3.3 Specific Structure Borings. The number and spacing of structure borings is highly variable depending on the complexity of the surface conditions.

A median program may consist of one boring at the end of each pier and abutment, and at the outer end of each wingwall more than 6 M (20 ft.) long. When piers and abutments are more than 30 M (100 ft.) long, additional explorations may be required. Pile trestles should have a boring at the opposite ends of adjacent bents, or at both ends of each bent in radically changing conditions. Culverts, depending on the length, may have a minimum of two borings. Borehole spacing for retaining walls may be 30 M (100 ft.) or at each end of the wall if it is less than 30 M (100 ft.) long.

The number and spacing of the boreholes may be increased or decreased from the median depending upon the anticipated geological conditions within the project area.

7.4.3.4 Critical-Area Explorations. In areas where the preliminary investigations indicate critical geological conditions such as a highly irregular and shallow bedrock surface, swamp deposits or underground caverns, it may be desirable to obtain additional explorations on a 15 M (50 ft. grid) pattern in the area of concern. These additional explorations may consist of hand or machine probings to supplement the more conventional test borings (Section 7.5.3).

7.4.3.5 Tunnel Borings. Borehole spacing for tunnel alignments is variable depending upon the site topographic and geologic conditions. Anticipated soil, rock or mixed-faced tunneling conditions will impact not only the borehole spacing, but the types of exploration used. Severe topographic conditions may prevent access to areas of critical concern.

The following general borehole spacing guidelines for tunnels are subject to greater influence and control by the site conditions, rather than the project design criteria and requirements:

Soft Ground Tunneling	
Adverse Conditions	15–30 M (50–100 ft.)
Favorable Conditions	90–150 M (300–500 ft.)
Mixed-Face Tunneling	
Adverse Conditions	8–15 M (25–50 ft.)
Favorable Conditions	15–23 M (50–75 ft.)

Hard Ground Tunneling

Adverse Conditions	15–60 M (50–200 ft.)
Favorable Conditions	150–300 M (500–1,000 ft.)

Geophysical survey methods (Section 5) used in conjunction with the subsurface exploration program, can further define the geologic conditions and modify the general borehole spacing outlined above.

7.4.4 Exploration Depths

The required depths of the subsurface explorations depends on the design considerations such as the size and type of structure and the character and sequence of the subsurface conditions. The explorations should penetrate through any unsuitable or questionable foundation materials and sufficiently deep into firm stable soils such that significant settlement will not develop from compression of the stratum or deeper soils due to loads imposed by the structure. A commonly used "rule-of-thumb" is to carry the borings to such depth that the net increase in soil stress under the weight of the structure is less than ten percent of the effective stress in the soil at that depth, unless bedrock or very dense soils known to lie on rock are encountered first. The added stress may be computed from appropriate charts or tables using the Boussinesq or Westergaard solutions.

As with the criteria governing borehole spacing, when subsurface information that would permit determinations of borehole depths is lacking, the following general guidelines may be used in establishing the preliminary exploration depths.

7.4.4.1 Subgrade Borings. In areas where the preliminary alignment profile indicates that minor cuts are anticipated, the explorations should extend from 2–3 M (5–10 ft.) below the profile elevation. If soft cohesive soils are encountered, this depth is increased as required to fully evaluate the stratum. Shallow refusals encountered within the limits of the proposed cut should be cored a minimum of 3 M (10 ft.) to determine the presence of bedrock.

7.4.4.2 High Embankment and Deep Cut Borings. If embankments greater than 5 M (15 ft.) are anticipated, the test borings should penetrate to approximately two (2) to four (4) times the height of the proposed embankment, depending on the width of the proposed roadway, unless rock is encountered above that depth. The depth may be decreased to approximately the height of the embankment if very suitable bearing material is encountered, such as dense sand/gravel soils.

In areas of excavations in excess of 5 M (15 ft.), the test borings may penetrate to approximately twice the depth of the excavation. If bedrock or refusal is encountered within the proposed excavation depth, the boreholes should be cored a minimum of 3 M (10 ft.) to determine the quality of the rock or the nature of the refusal.

If there is a possibility of artesian acquifers; soft, highly cohesive soil; or loose, liquefiable granular soils, the exploration depths should be increased to penetrate these deposits.

7.4.4.3 Specific Structure Borings. The "rule-of-thumb" guidelines for depths of borings, described above in Section 7.4.4, refer specifically to structure borings, where the explorations should be advanced to the depth where the net increase in soil stress due to structure load is less than 10 percent of the existing effective stress in the soil at that depth. It is desirable that all explorations penetrate to rock with selected explorations penetrating 3 to 6 M (10 to 20 ft.) into the rock. A minimum depth of 10 M (30 ft.) of penetration below the footing elevation has been suggested unless rock or exceptional bearing materials are encountered.

Some Agencies (The Illinois Department of Transportation) have prepared recommended depths of structure borings based on Standard Penetration test Values (N) or Unconfined Compressive Strength (Qu). However, the department acknowledges that the table may not be applicable in stratified, unconsolidated deposits or in uniform deposits of silt, such as loess.

7.4.4.4 Critical-Area Explorations. In the areas where the preliminary investigations indicate critical geological conditions which would have a major impact on the design and construction of the project, the borehole depths should be extended to evaluate these conditions. Such conditions include highly compressible soils, artesian conditions and underground caverns.

It is preferable to continue the explorations deeper than necessary, when in doubt, than to terminate at depths which will not provide the design engineer with sufficient data.

7.4.4.5 Tunnel Borings. The minimum depth of tunnel borings is generally at least one (1) to one and one-half (1-1/2) tunnel diameters below the proposed invert grade. However, if the proposed vertical alignment is subject to modifications, it may be more economical to extend these depths to two (2) or three (3) times the tunnel diameter, for contingency purposes. As discussed in Section 7.4.4.4, the exploration depths should be further increased to evaluate any critical geological conditions which may exist in the vicinity of the tunnel invert, or within a depth that could impact on tunnel construction.

7.4.5 Sampling Requirements

An objective of the subsurface exploration program is to obtain samples that are representative, or nearly so, of the *in situ* soil and rock conditions. The sampling requirements, including type and interal, are subject to the same geological variables and project requirements which control the location and depth of the explorations.

Generally, representative *in situ* samples should be obtained at every change in soil strata and at an interval not to exceed 1.5 M (5 ft.) vertically. This interval may be increased in thick uniform deposits or be decreased in the more complex sediments.

It may be advantageous in areas of erratic conditions or immediately beneath foundation bearing elevations to obtain representative samples continuously. Rock core, due to the nature of the sampling devices, is usually obtained continuously.

The majority of the samples obtained in test borings are usually 35–75 mm (1.5–3 in.) in diameter; project requirements may dictate diameters up to 150 mm (6 in.) for samples for certain laboratory testing.

The recovered sample is examined and logged by qualified field personnel and a representative portion or portions are selected and preserved, usually in glass jars. Occasionally, it may be preferable to preserve the entire sample that is recovered for more detailed analysis and testing.

Sample preservation and transportation are discussed in more detail under Section 7.9.

7.4.6 Right-of-Entry, Permits and Utilities

Prior to commencing the actual field exploration program, the owners of the property where the work will be performed must be contacted and permission obtained to conduct the work.

This permission, preferably written, is usually obtained by representatives of the agency or organization requesting the work and is not a function of the drilling contractor or his personnel. This permission should cover rights of access and conduct of the work and any special provisions required by the property owner, such as working hours and cleanup.

Certain public and private property, such as navigable waters, railroad property and public streets, may require special permits for right-of-entry. These working permits may involve a fee or special insurance and are usually obtained by the drilling contractor.

The location of any underground or overhead utilities must be determined before commencing the drilling operations. This should be a function of the drilling contractor, who should contact the various utility companies for their approval of the proposed exploration locations. In the event that a state agency is initiating the work and not a private contractor, these functions may be assumed by the agency conducting the work.

Regardless of the procedures followed or who obtains the various rights-of-entry and clearances, these functions should be completed prior to moving any equipment into the field.

7.4.7 Borehole Location Tolerance

The allowable tolerance for the field locations of the explorations in relation to the plan locations is subject to major variations depending on the level of the exploration program, degree of complexity of the subsurface conditions, and anticipated use of the information obtained. The borehole location tolerances should be established by the project geologist or geotechnical engineer for the specific site investigation and may vary from one foot to tens of feet. Site access and underground utilities may dictate locations beyond the limits of any established tolerances.

If excessive costs are associated with meeting very close location tolerances, consideration should be given to supplementary explorations which may bracket the area of concern for more reasonable costs.

7.4.8 Survey of Locations

The "as drilled" locations and elevations of all explorations must be determined and plotted on the applicable project base plans. This may be accomplished by locating the explorations in the field prior to the conduct of the work or subsequent to the actual drilling.

It may be preferable, for liability purposes, that the proposed locations not be physically staked in the field. They would be located on the project base plans for the Contractor's location purposes, but subject to approval by the engineer's field personnel. Centerline alignment base line stakes would generally provide sufficient control for the Contractor to determine the proposed locations. The actual drilled location and elevations are usually determined to the nearest 25 mm (1 in.) vertically and .3 M (1 ft.) horizontally after the exploration program is complete.

In the event that a detailed survey of the completed subsurface explorations is not required, the approximate locations should be determined employing compass and field pacing procedures and be tied into physical or topographic features when possible. Ground surface elevations could be estimated from project topographic maps. A statement of the degree of accuracy of these locations should be indicated on the applicable project base plans.

Numbering systems for explorations in the program vary considerably depending on the standards of the agency or project manager. Numbers may be established prior to the field work or be assigned consecutively as the work progresses.

7.4.9 Drilling Equipment

A great variety of conventional and modified drilling rigs are available in both the private and public sectors. Test boring equipment is manufactured in a number of sizes and styles, ranging from small, hand-held portable drills and augers, to massive, off-shore mineral exploration equipment. The selection of the drilling equipment is an important aspect of any subsurface exploration program. The equipment must be capable of meeting all, or as many of the project requirements as possible, have sufficient mobility and possess the ability to convert rapidly from one drilling technique to another. Hydraulic-feed machines are usually preferable, especially when they can maintain a constant advance pressure through varying formation densities, which minimizes erosion and disturbance of the *in situ* materials.

The selection of the most economical and satisfactory method of drilling involves and evaluation of the numerous borehole advancement and stabilization techniques available with consideration being given to the nature of the formations to be penetrated and the type of sample that is required. No single method of drilling will prove satisfactory and economical for all formations and sampling requirements.

Local practices, personal preferences, and equipment availability will continue to dictate the type of drilling equipment and techniques on the majority of subsurface exploration programs.

7.4.10 Special Equipment

The project requirements or site conditions may be such that special drilling or sampling equipment is required or that supplemental logistics are necessary in realizing the project goals.

It may be more practical and economical to use selected, advanced or innovative equipment, which may not be readily available, in determining the subsurface conditions for the subject site. In addition, special barges, bulldozers, cranes or compressors may be required to complete the work. This specialized and supplemental equipment will not only have an impact on the cost of the program, but on environment as well. Both of these aspects should be fully considered during the preliminary phases of exploration program estimating (Fig. 7-1).

7.5 EXPLORATION METHODS

The selection of the specific drilling equipment and methods to be used for a particular site investigation are dependent upon a number of factors. These may include site accessibility, equipment availability, and geologic conditions, in addition to economic and environmental considerations.

The majority of the test borings conducted for highway investigations are usually small diameter, 50–100 mm (2–4 in.) vertical boreholes in which a variety of sampling methods and equipment are used. Although the majority of these "standard" borings are conducted on land in the vertical mode, the basic borehole advancement and stabilization techniques would generally remain the same for larger diameter boreholes, inclined borings, or explorations conducted on water.

New exploration technology which is still in various development or experimental stages shows a potential for possible future utilization in minimizing subsurface unknowns. These methods include long distance horizontal boreholes, acoustic borehole logging, and a variety of recently developed geophysical methods which are discussed in other sections of this Manual.

7.5.1 Borehole Advancement

The more commonly used borehole advancement techniques may be classified into six groups, depending on the method used is displacing or removing material during penetration of the borehole. They are:

- Displacement Boring
- Wash Boring
- Percussion Drilling
- Rotary Drilling
- Auger Boring
- Continuous Sampling

The quality of information obtained from the various methods varies with the character of the subsurface geologic conditions; therefore, careful consideration must be given when selecting the desired method. It may be necessary to employ more than one method in advancing a particular borehole.

7.5.1.1 Displacement Borings. This method is the most simple and economical test boring procedure in

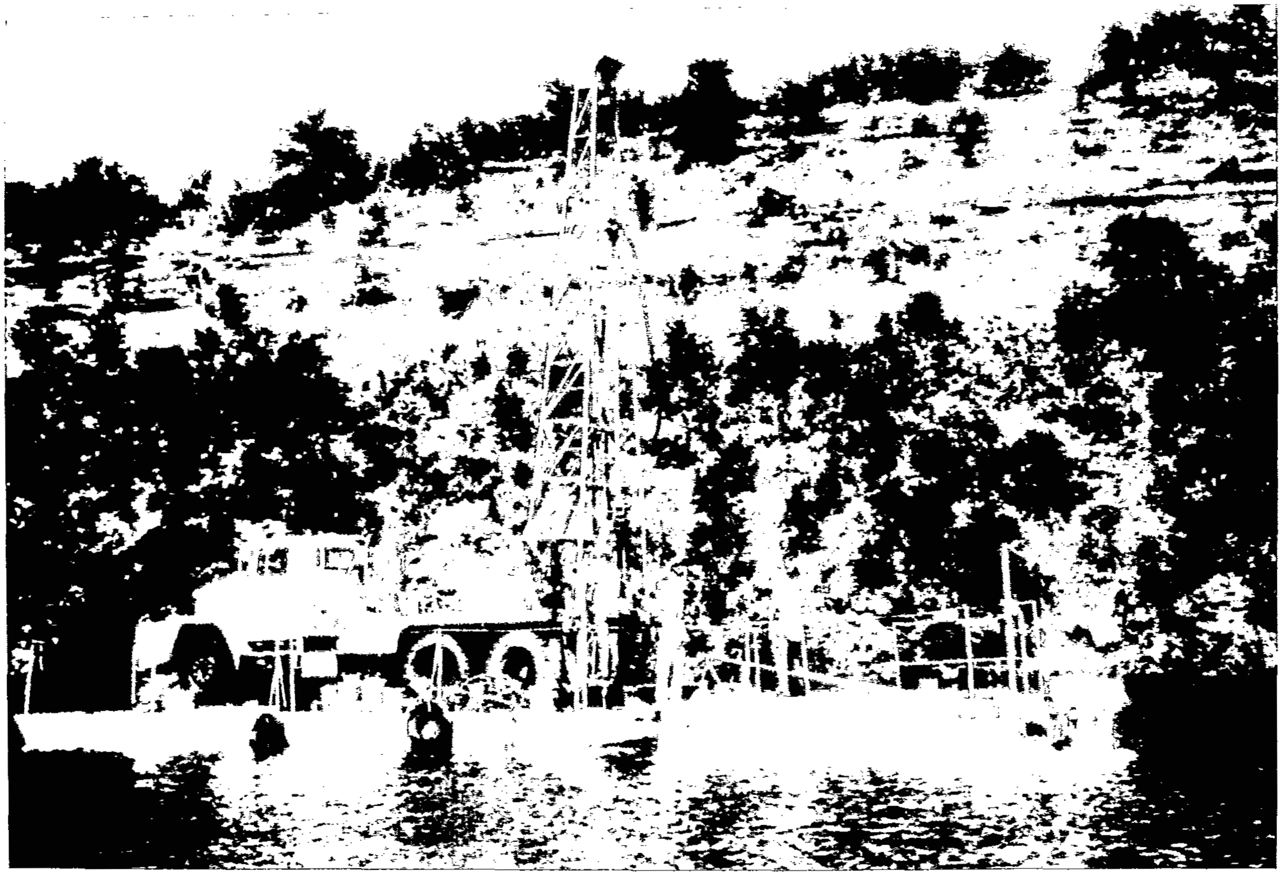


Figure 7-1. Failing drill rig and raft. (Courtesy Texas Highway Department)

non-caving ground. There is no attempt to stabilize the borehole and closed samplers such as the split tube, cup or piston sampler are forced in a closed position to the required sampling depth. This method is generally employed in preliminary reconnaissance work where only general subsurface information is required.

Exploratory probes which may be considered a type of displacement boring are discussed in more detail under Section 7.5.3.

7.5.1.2 Wash Borings. This method involves advancing steel casing, as required, and washing out the material to the bottom of the casing or desired sampling depth below the casing, with a variety of chopping bits. The drill rods and chopping bits are alternately raised and dropped, with some hand rotation, to break up the material within the casing; the loosened materials (cuttings) are then carried to the surface by the recirculating drilling fluids. The "cuttings" which are carried to the surface provide additional information about the overburden soil conditions between conventional sample locations.

The borehole may be stabilized with casing, water or drilling mud and open samplers such as the split or solid tube types, are driven into the "undisturbed" material at the bottom of the borehole.

A typical wash boring set-up is shown on Figure 7-2; however, this procedure can be conducted without the benefit of the drilling machine if the situation should arise. There are several advantages with this method in that the equipment is relatively inexpensive and very portable and may meet the project design requirements where other equipment is too large or sophisticated.

7.5.1.3 Percussion Drilling (Churn or Cable Tool Drilling). This method involves advancing the borehole by raising and dropping a heavy drill bit to form a slurry. Samples are obtained by bailing the slurry or replacing the bit with a conventional sampler after the slurry has been removed, and then driving the sampler with very heavy "down the hole" tools referred to as "jars". This is a primary method employed in the well drilling industry and is generally not as applicable for geotechnical investigations, due to potential sample disturbance.

A relatively new and innovative application of air-

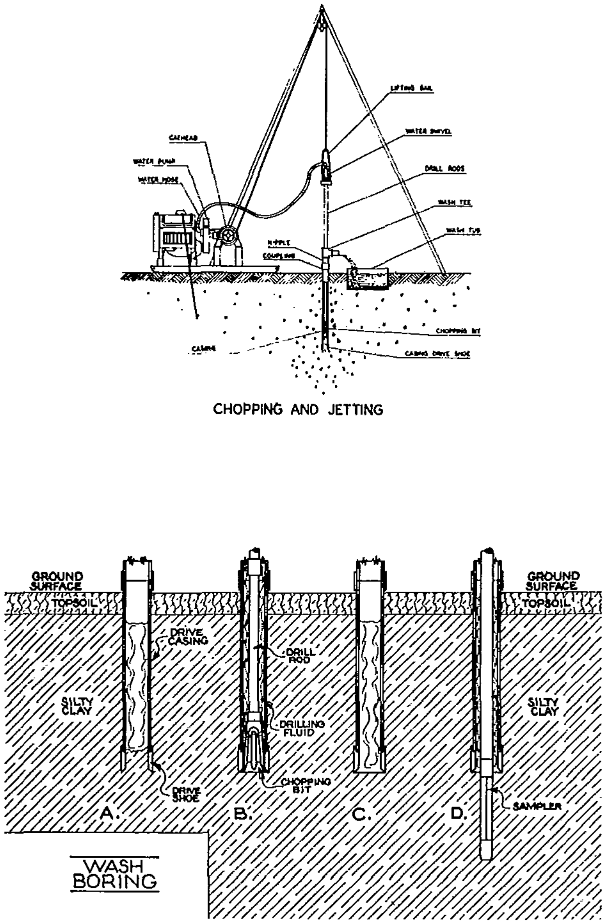


Figure 7-2. Wash Boring Technique. (Courtesy Central Mine Equipment Co.)

operated, percussion rock drilling equipment equipped with specialized eccentric drill bits are being used for rapid borehole advancement techniques. The "Odex" system is discussed in more detail under Special Drilling Techniques, Section 7.5.3.4.

7.5.1.4 Rotary Drilling. Rotary drilling is a very versatile and adaptable technique which may be used with a range of equipment models and sampling devices. Rotary drilling consists of advancing a cased or uncased borehole by rapid rotation and pressure on the drill bit which cuts and grinds the sediments at the bottom of the borehole into small particles called "cuttings". These cuttings are subsequently removed from the borehole by pumping air, water or drilling mud from a surface reservoir through the drill rods to the bottom of the borehole.

When the required sampling depth is obtained, the drill string is removed from the borehole and the desired sampling device is lowered to the bottom of the hole. Figure 7-3 shows a typical rotary drilling setup. A number of drill rods and bits are available for the various types of overburden and rock encountered and can be changed by the driller as the situation demands. Generally, the heavier duty and larger diameter drill rods will give additional stability to the drill bit, decrease whip and vibration, and help keep the borehole straight and uniform during drilling. Although the X series drill rods are often used, particularly NX size, the Diamond Core Drill Manufacturers Association (DCDMA) have standardized the various drill rods sizes which are summarized on Table 7-1.

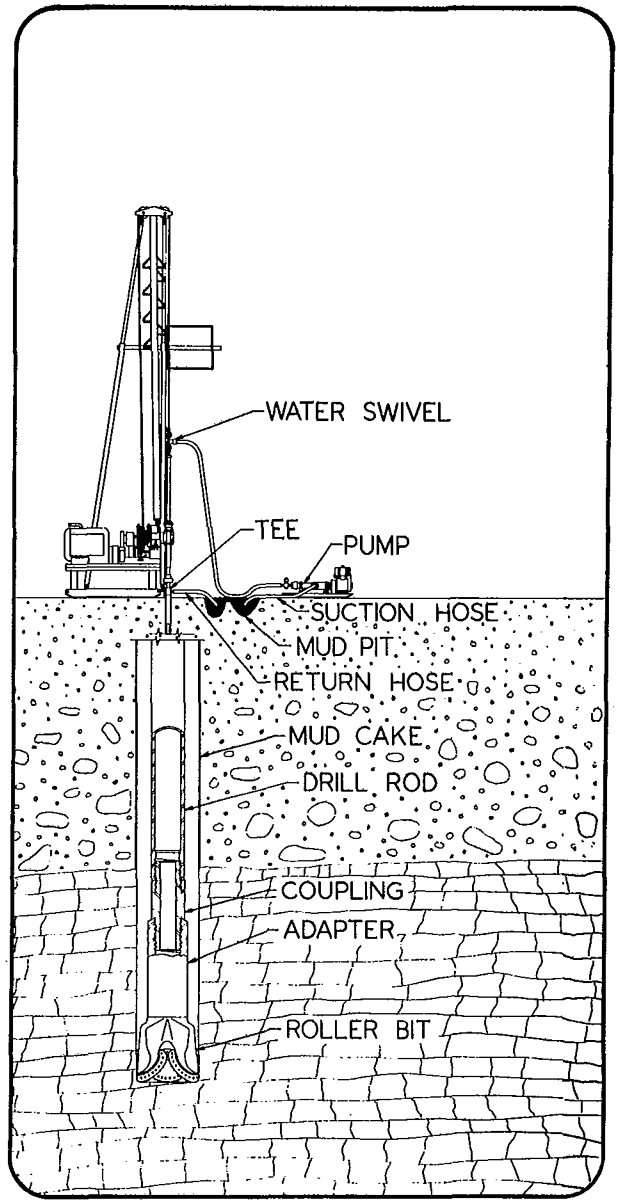


Figure 7-3. Rotary Drilling Technique. (From "Soil Sampling Methods and Equipment Catalogue," Longyear Co.)

Despite efforts being made to standardize the various sizes of drill rods and casing, many types and

Table 7-1.
Drill Rod and Coupling Standards

Symbol	Rod and Coupling (O.D.)		Rod (I.D.)		Coupling (I.D.)	
	mm	in.	mm	in.	mm	in.
RW	27	1.093	18	0.719	10	0.406
EW	35	1.375	25	1.000	11	0.437
AW	44	1.718	34	1.344	16	0.625
BW	54	2.125	44	1.750	19	0.750
NW	67	2.625	57	2.250	35	1.375
HW	89	3.500	78	3.062	60	2.375

Note: Metric measurements are given to nearest whole millimeter.

sizes remain on the market. The American Petroleum Institute (API) drill pipe is basically a heavier wall pipe than the DCDMA standard and is used more frequently in the petroleum industry. A variety of "black" pipe and specially manufactured casing is still in common usage throughout the United States.

A number of rotary drilling bits are available for the different types of soil and rock encountered and can be changed by the driller as the situation demands. Two, three and four-wing carbide insert drag bits are usually used in relatively soft or loose soils, and the heavier, tri-cone roller bits are used in the denser soil and bedrock. Figure 7-4 shows a range of sizes and types of bits which are used for rotary drilling purposes.

The rotary drilling technique is one of the most common and popular methods used in foundation investigations in the United States.

7.5.1.5 Auger Borings. The use of rotary auger drilling methods is gaining in popularity as a rapid and economical method of conducting subsurface explorations. There are certain inherent limitations when using this type of procedure, which should be carefully evaluated for the site specific exploration programs.

Generally, augers are mounted on large truck rigs for rapid mobility, but may vary from small hand-operated augers to track-mounted equipment.

Auger boring techniques may be divided into three categories depending on the type of auger equipment used:

- Construction augers
- Solid flight augers
- Hollow stem augers

Figure 7-4. Rotary drill bits. (Haley & Aldrich, Inc.)

Construction augers are generally very large diameter solid flight or bucket type augers which are used for visual inspection of very shallow, near-surface overburden conditions. Construction augers are not designed for soil sampling; however, a large amount of material may be brought to the surface for bulk sampling, if required.

Solid or continuous flight augering is generally the fastest method of obtaining a borehole in soil that is compatible to auger exploration. Samples are obtained from the auger flights. In addition, if the soils contain sufficient cohesion to enable the borehole to remain open, the augers may be removed from the borehole at any desired depth and conventional samples obtained at that depth. In loose granular soils and high water table conditions, this is rarely the case. Obstructions such as large boulders will usually stop auger penetration.

Solid flight augers are available in sizes ranging from 50–300 mm (2–12 in.) diameter. Depth penetration is generally a function of the size of the power source. Usually, auger borings are employed in preliminary exploration phases or where only near-surface information is required. Figure 7-5 shows a typical truck-mounted, solid flight auger in operation.

Hollow stem augering is an improved modification to the solid flight augers and is extensively used for geotechnical explorations. A removable center plug in the auger allows conventional sampling tools to be lowered to the bottom of the borehole without removal of the augers. The augers, in effect, function as temporary casing. In relatively stiff formations, the center plug may be eliminated during drilling or conventional wash boring techniques may be used in conjunction with the hollow stem augering technique. Figure 7-6a shows a schematic diagram of the hollow stem auger with its removable center plug. Hollow stem augers are available with inside diameter dimensions ranging from 60–150 mm (2.25–6 in.).

Although the bore wall collapse problem is eliminated with hollow stem augers, high water table conditions in loose or medium dense granular soils will tend to "blow in" or flow up inside the casing. Drilling mud can be injected during auger advancement, through a fluted kelly or with a spindle adaptor to allow penetration below the groundwater table (Figure 7-6b).

Another disadvantage when using hollow stem augers, is that the soil conditions between the samples is generally not known and some disturbance of the natural ground beneath the augers may occur which might not be acceptable for *in situ* borehole sampling and testing.

Hollow stem augers are particularly adaptable to drilling in hard or dense sediments where drilling water is difficult to obtain or where below-freezing temperatures preclude the effective use of water for drilling.

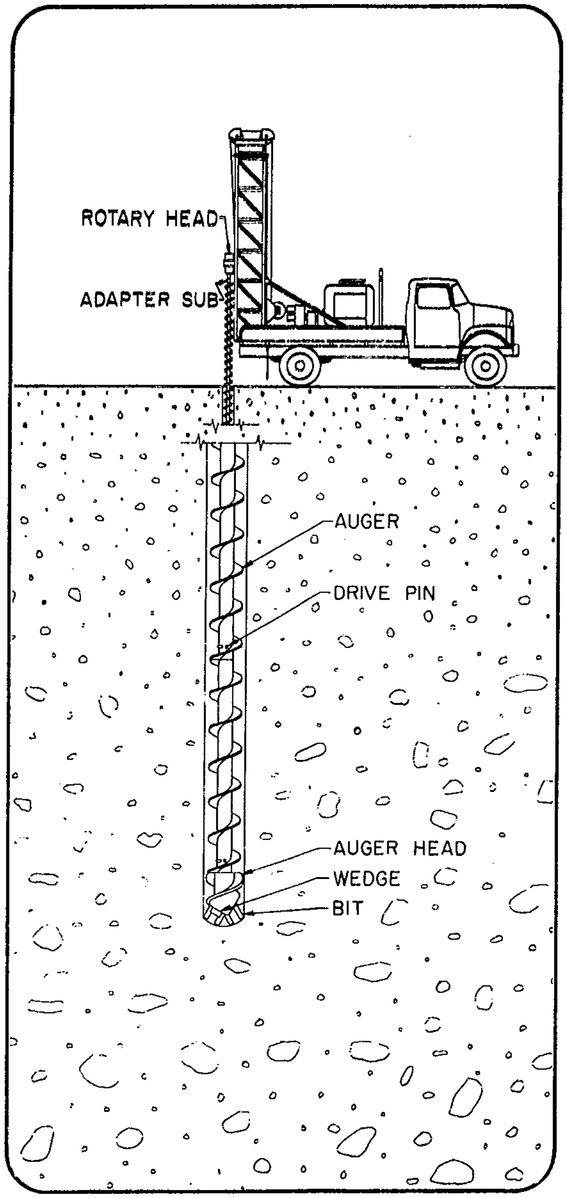


Figure 7-5. Auger drilling technique. (From "Soil Sampling Methods and Equipment Catalogue," Longyear Co.)

7.5.1.6 Continuous Sampling. A variety of sampling tools may be used to obtain continuous representative samples, with any test boring procedure. Such sampling may provide more reliable and detailed information on subsurface conditions. In some soil conditions, particularly cohesive soils with adequate shear strength, continuous sampling may be employed alone; in effect, creating an uncased borehole. However, it is usually necessary to clean out the

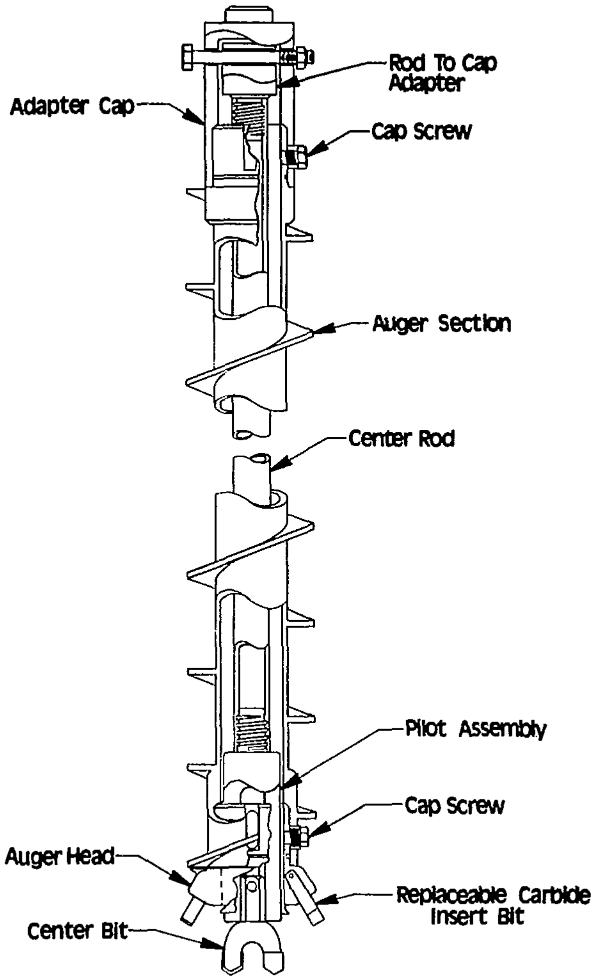


Figure 7-6a. Components of the Hollow-Stem Auger Drilling System. (Courtesy Central Mine Equipment Co.)

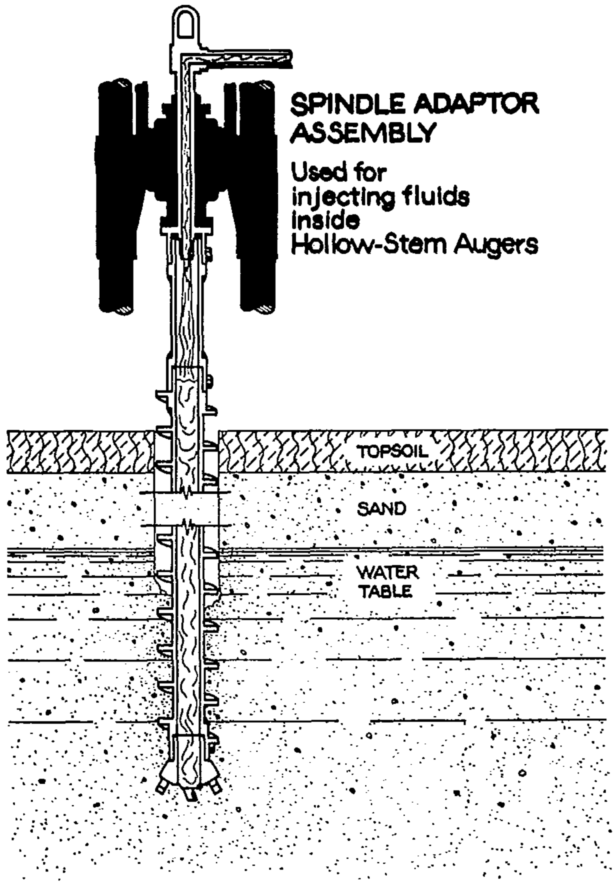


Figure 7-6b. Injecting Drilling Fluid Through a Hollow-Stem Auger with a Spindle Adaptor. (Courtesy Central Mine Equipment Co.)

borehole using conventional wash boring techniques, between samples, resulting in the need for some form of borehole stabilization.

A continuous column of soil or rock samples provides the most accurate picture of subsurface conditions.

7.5.2 Borehole Stabilization

A problem common to all test boring methods is the necessity of maintaining borewall and bottom stability in order to obtain relatively undistrubed samples of the desired stratum. The subsurface conditions encountered in a specific area will generally dictate or influence the selection of the borehole stabilization methods, which can be grouped into six general categories.

- Water Stabilization
- Mud Stabilization
- Air Stabilization
- Casing Stabilization
- Grout Stabilization
- Freezing Stabilization

As with the various techniques which may be employed in advancing the boreholes, the selected stabilization method may also affect the quality of the sample recovered. Several different methods may be employed in a single borehole, in any combination, to provide the most representative sample.

7.5.2.1 Water Stabilization. A recycling or continuous water supply system is the most common and economical method of maintaining borehole stability. Water induced into the borehole will generally counteract soil and pore-water pressures in partially or fully saturated sediments for a sufficient length of time to allow sampling at the selected stratum. Water alone will generally not prevent the caving or sloughing of the borehole in soft or cohesionless sediments, especially above the water table. An uncased borehole, utilizing water for stabilization purposes, is typically used in rock or in relatively stiff, cohesive soils.

7.5.2.2 Mud Stabilization. Drilling mud is simply a mixture of water and mineral particles in suspension which has a specific gravity and viscosity greater than water. It may be a natural or artificially pre-mixed fluid which is recycled through the uncased borehole to maintain a state of equilibrium, transport the borehole cuttings to the surface, and act as a coolant for the drill bit. It is also employed to improve sample recovery and minimize soil disturbance in cased boreholes.

Drilling mud may be prepared from any native clay or from several commercially available products, which are highly colloidal and thixotropic and contain various additives to control dispersion and viscosity. The higher specific gravity of the mud allows it to develop more positively down-the-hole pressures, in addition to forming a relatively impervious lining along the borehole walls. The mud will also tend to keep the cuttings in suspension longer, allowing more representative sampling at the bottom of the borehole. In addition, the mud will reduce abrasion and retard corrosion of the drilling and sampling tools.

The basic mud mixture which is used on many subsurface exploration programs is bentonite and fresh water (approximately six percent bentonite by weight). Attapulgite, a non-flocculating clay, will make a suitable mud when mixed with salt water. Weight additives, such as pulverized barite, hematite, galena, or other heavy mineral products, may be added to the mixture to further increase specific gravity in unstable soils or in the presence of artesian conditions. The drilling mud must be carefully mixed and monitored during the life of the test boring. Driller expertise is required to maintain the correct mixture balance for optimum performance.

U.S. Bureau of Reclamation (USBR) has established general guidelines for various drilling mud mixtures which are summarized on Table 7-2. However, these mixtures are subject to variations depending upon the local geological conditions and the most suitable mixture is the thinnest mix which will meet the specific soil conditions encountered.

The Materials Testing Section of the U.S. Soil Conservation Service (SCS) has evaluated heavy mud mixtures using bentonite with a barite weight additive for troublesome soils and Table 7-3 summarizes the various wet-weight mixtures.

The Baroid Corporation's "Quick-Gel" is a very fast acting and commonly used bentonite mud product and "Baroid" is a barite base produce used as a weighting agent.

Table 7-2
Approximate Bentonite Mud Mixtures

Purpose	Consistency	Proportion Unit of Water to Bentonite
Support of borehole	Very thick cream	27 kg (60 lb.)
Removal of cuttings	Thick to very thick cream	11–27 kg (25–60 lb.) depending on soil grain size
Retention of sample in sampler	Thin to thick cream	9–27 kg (20–60 lb.)
Assist in cutting action of sampler	Thin to thick cream	11–23 kg (25–50 lb.)

* Unit of water is 3791 (100 gal.).

To minimize environmental pollution and to facilitate the subsequent installation of observation wells and piezometers in the completed boreholes, a biodegradable organic polymer, manufactured by the Johnson Division, Universal Oil Products, under the patented trade name of "Revert" is available. "Revert" has the important characteristic of automatically changing or reverting to a fluid as thin as water after a period of three to four days.

Figure 7-7 shows "mud" being mixed in a recirculation tank prior to drilling.

Drilling fluids can be lost when lenses or pockets of highly permeable strata, such as clean gravel, are encountered, especially in the presence of a strong groundwater flow. However, these permeable zones can be sealed by the addition to the drilling fluid of mica, wood fibers, straw, or other commercial fibrous products, which will be deposited in these zones and seal off the pervious strata.

Table 7-3
Bentonite/Barite Mixture Weights

Dry Weight of Powered Mud Per 0.28 Cubic Meters (10 Cubic Feet) of Water				Wet Weight of Mud Mixture	
Bentonite		Barite			
kg	lbs.	kg	lbs.	kg/m^3	p.c.f.
27	60	0	0	1041	65
27	60	9	20	1057	66
27	60	27	60	1105	69
27	60	54	120	1153	72
27	60	108	240	1298	81

Figure 7-7. Mixing revert "mud" in a recirculation tank. (Haley & Aldrich, Inc.)

Subsurface exploration techniques combining rotary drilling methods with uncased mudded borehole stabilization is a relatively rapid and economical procedure which can facilitate the recovery of more representative subsurface information than other conventional exploration methods.

7.5.2.3 Air Stabilization. The application of compressed air which is circulated through the borehole to remove cuttings is also a method of borehole stabilization. Compressed air circulation is also used in cold climates as a non-freezing alternative to drilling mud, which is susceptable to a solidification in open-air reservoir pits at the ground surface. This method is usually applied with large rotary or percussion drilling equipment where excessively dense soils, rock or obstructions must be penetrated. The principal of reverse circulation is utilized where the borehole cuttings are carried up the center of the drill rods (Figure 7-8). The drilling method may combine both rotary and percussion techniques through the use of diesel operated "mini" pile driving hammers, although down-the-hole hammers are also available.

A continuous column of "dry" sample is brought to the surface for evaluation and sampling. A foam flushing agent may be added to assist in removing the

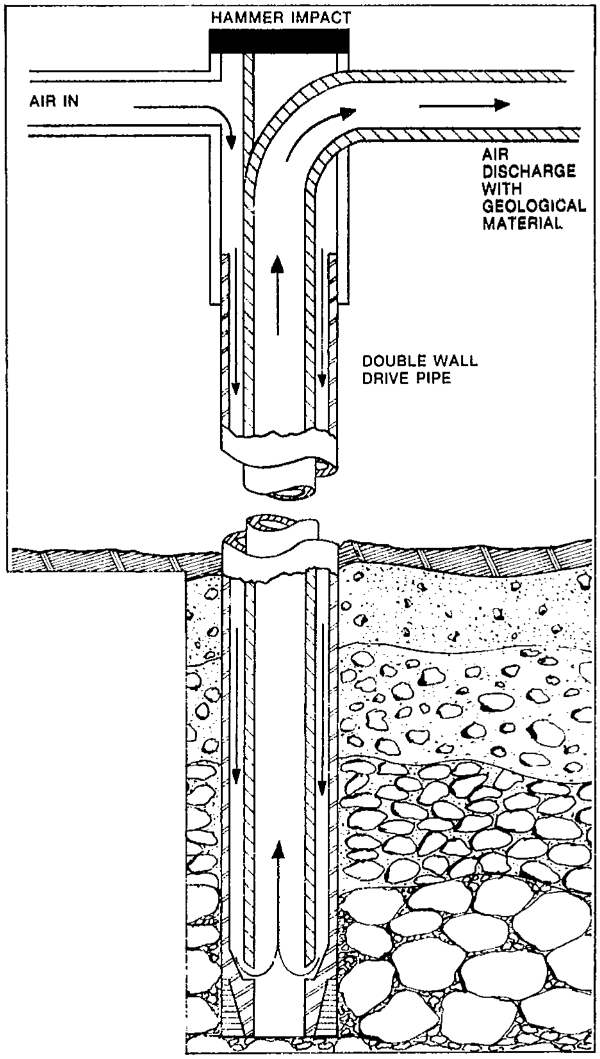


Figure 7-8. Reverse air recirculation technique. (From "The Progressive Drilling Contractor," Becker Drills, Inc.)

heavier cuttings which will also have the added benefit of lubricating and sealing the borehole walls. Although conventional sampling methods are usually not conducted with this procedure, it can be adapted for a variety of sampling techniques, but with some difficulty and at a major loss in production rates.

Borehole stabilization and drilling with air is generally not a practical technique in cohesive soils below the water table. Section 7.5.3.4 describes in additional detail one method of air stabilization referred to as the "ODEX" Drilling System.

Although this system can penetrate extremely dense and "bony" over-burden at very rapid rates, sampling limitations, environmental considerations, supplemental equipment requirements and cost should be carefully evaluated during consideration of this methodology.

7.5.2.4 Casing Stabilization. Driving heavy duty steel pipe or casing provides the most reliable, and commonly used, although relatively expensive, method of advancing a borehole to its required depth and maintaining stability of the borehole walls. The casing is usually advanced by constant blows of a drive hammer (typically 136 kg; 300 lb.; falling .6 M or (2 ft.) upon a drive head which is attached to the casing (Figure 7-9). As the blows to drive the casing supply constant energy, supplementary information may be obtained on the soil resistance by counting the

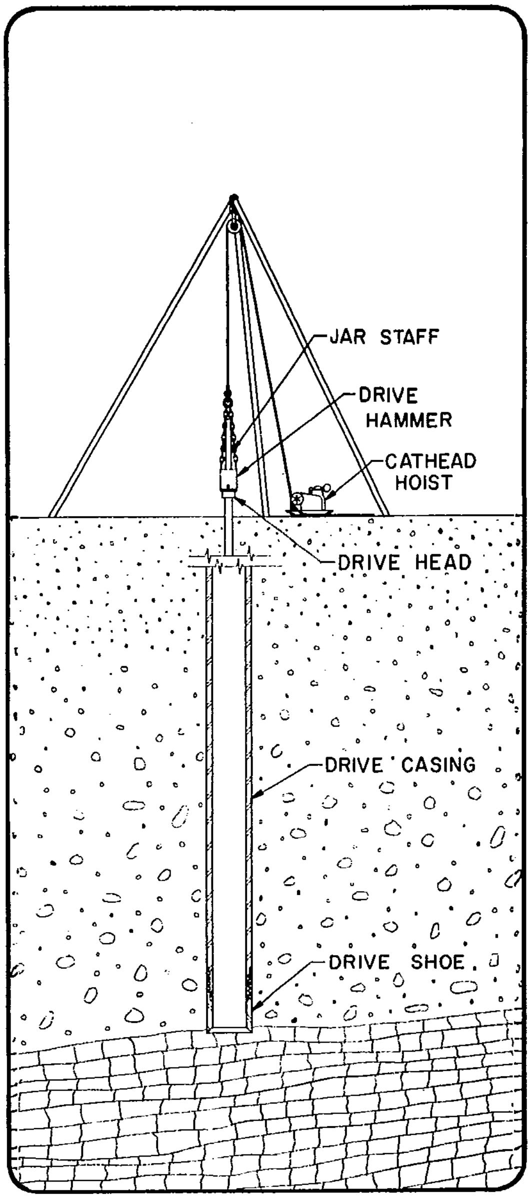


Figure 7-9. Borehole casing stabilization. (From "Soil Sampling Methods and Equipment Catalogue," Longyear Co.)

number of blows per foot and the resulting penetration.

The Diamond Core Drill Manufacturers Association has established standards of nomenclature for the various casing diameters which are summarized in Table 7-4.

The casing is usually driven in increments of 1.5 M (5 ft.) with representative samples being obtained at the completion of each drive. The increments may be varied to meet specific sampling requirements. Various sizes of casing may be "telescoped" within each other to facilitate the coring of obstructions or to reduce the desired size of the borehole at depth.

The heavy duty steel casing may also be equipped with a diamond bit "shoe" and drilled, rather than driven, to the required sampling depth. This procedure will allow the penetration of obstructions without a decrease in borehole size.

After the casing is seated at the required depth, the hole must be thoroughly cleaned out before obtaining a sample. In soft or loose materials, stability of the borehole bottom is increased by keeping the casing filled with water or drilling fluids.

The use of hydraulically operated hollow stem auger drilling equipment, with a removable center plug to allow passage of the sampling tools through the auger is also considered a type of temporary casing.

7.5.2.5 Grout Stabilization. A borehole may encounter local zones of instability in the bedrock due to shear zones, faults, weathering or fractured rock which prevents deeper penetration of the drilling tools. This zone may be stabilized by pumping a cement or chemical grout into that portion of the hole and redrilling the borehole through the hardened plug. Although additional time may be required for setting of the grout, it may be preferable to advancing casing to this depth, which would also reduce the borehole size.

This method may also be applied in extremely unstable areas in the overburden. However, depending on the depth of the zone, advancing casing may be a more practical solution.

7.5.2.6 Freezing Stabilization. A borehole may be stabilized by freezing the soil through which it passes by replacing the drilling fluid with alcohol, diesel fuel or a brine solution which is chilled with "dry ice". This method is generally not applicable in unsaturated ground or where there is a strong groundwater flow, and it also represents an environmental concern from the standpoint of pollution.

In some instances, it may be advantageous to use this procedure to recover "undisturbed" samples of naturally frozen granular soils to determine the presence of ice lensing or segregation.

A more costly method involved circulating the cooling liquid through a series of pipes which have been driven or drilled in a circle around the primary borehole. This procedure could facilitate the recovery of large-diameter samples of unstable granular soil or fractured rock.

Table 7-4
Standard Casing Sizes

Type	Designation		I.D.		O.D.	
	mm	in.	mm	in.	mm	in.
XH "Black"	64	2-1/2 in.	64	2.50	73	2.88
	76	3 in.	76	3.00	89	3.50
	89	3-1.2 in.	89	3.50	102	4.00
	102	4 in.	102	4.00	114	4.50
Flush-Joint	RX or RW		30	1.19	37	1.44
	EX or EW		38	1.50	46	1.81
	AX or AW		49	1.91	57	2.25
	BX or BW		60	2.38	73	2.88
	NX or NW		76	3.00	89	3.50
	HX or HW		102	4.00	114	4.50
	PX or PW		127	5.00	140	5.50
	SX or SW		152	6.00	168	6.63
	UX or UW		178	7.00	194	7.63
	ZX or ZW		203	8.00	219	8.63
Flush-Coupled	RX or RW		30	1.19	37	1.44
	EX or EW		41	1.63	46	1.81
	AX or AW		51	2.00	57	2.25
	BX or BW		65	2.56	73	2.88
	NX or NW		81	3.19	89	3.50
	HX or HW		105	4.13	114	4.50

7.5.3 Special Exploration Techniques

A variety of subsurface exploration techniques, which range from simple to very sophisticated, may be used to determine or supplement information concerning the geological conditions in the project area. Any exploration technique which can properly evaluate the geotechnical parameters which will effect the design of the project is usually acceptable. Frequently, the most simple and economic methods, which are often overlooked, may be the most suitable. Occasionally, the very sophisticated and expensive methods are the only techniques which will provide the necessary information. Several special exploration techniques are summarized below to indicate the range and level of these techniques.

7.5.3.1 Exploratory Probes. Exploratory probing techniques are employed as preliminary or supplementary measures to determine the gross characteristics and depths of relatively thin surficial soil deposits.

- *Hand Probes.* Hand probes are made to obtain reconnaissance information in wetland areas, concerning the thickness and lateral extent of soft, compressible organic soils. Small diameter, flush coupled, steel rods are pushed by hand to refusal in the underlying inorganic soil.
- *Rod Probes.* Rod probes can be conducted with any conventional drilling equipment to provide general information on soil penetration resistance and depth to bedrock or refusal. Standard drill rods equipped with a point are driven or rotary-drilled to the required depth or refusal. Rod probes may be employed to supplement conventional subsurface exploration methods.
- *Auger Probes.* Auger probes can be conducted with rotary drill rigs equipped with solid flight augers to provide general information on soil types, penetration resistance, groundwater conditions and depth to bedrock or refusal. An advantage of auger probes over other probing methods is that soil is returned to the surface for general analysis and a borehole is created which, if remaining open, may be utilized for groundwater observation purposes.
- *Percussion Probes.* Air operated percussion drilling equipment may be used to obtain additional information on the depth of relatively shallow bedrock, especially in areas where very dense or "bouldery" overburden overlies bedrock. This equipment may be employed in conjunction with an acoustical listening device to more accurately define the subsoil and rock conditions. Refer to Appendix A for more details regarding acoustical monitoring.

7.5.3.2 Hand Explorations. The most basic of all exploration tools is the shovel and its use for examining in detail the near surface soil conditions should not be overlooked. Other hand explorations include:

- *Hand augers.* A variety of hand augers or post hole diggers may be used in obtaining representative samples of the near surface soil conditions. A variety of sizes and styles of cutter heads are available and extensions may be added for greater penetration depths (Figure 7-10). Small gasoline engine powered hand augers will increase depth penetration and decrease the difficulty of performing the work.
- *Retractable Piston Samplers.* Small diameter hand operated piston samplers are used primarily in reconnaissance survey to obtain represen-

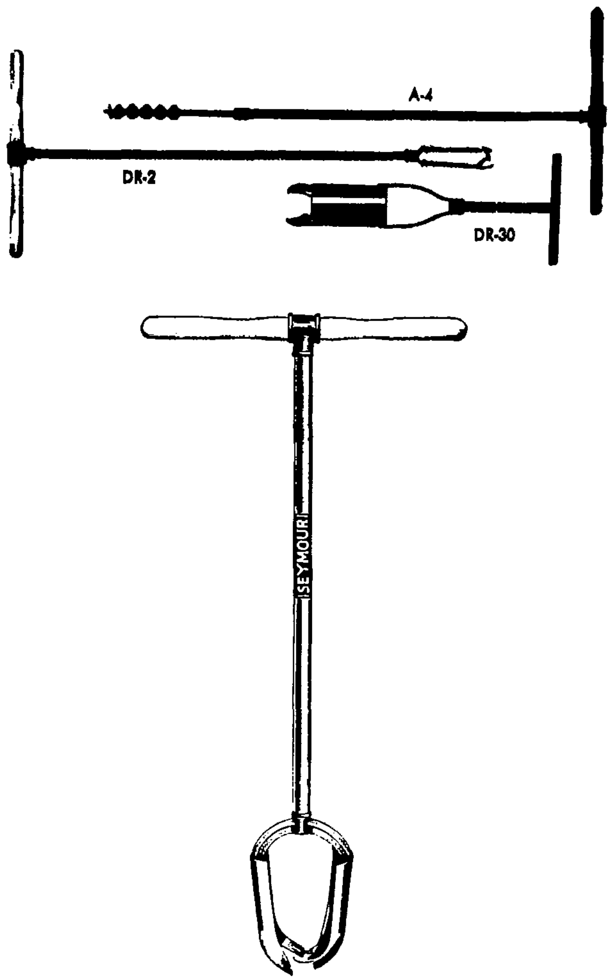


Figure 7-10. Hand auger equipment. (From "Soil Testing Equipment Catalogue," Soiltest Inc.)

tative samples of deeper, soft, cohesive and usually organic sediments for evaluation and testing. The sampler is forced by hand in the closed position to the desired sampling depth. The rods are then lifted approximately .3 M (1 ft.) which retracts and locks the piston. The sampler is then pushed approximately .3 M (1 ft.) into the soil, retaining the sample (Figure 7-11). A variety of these retractable piston samplers are available including the Davis, Stockstad, Hankinson, and Veihmeyer types.

Portable geotechnical field equipment has been recently developed by the Swedish Geotechnical Institute which can be utilized during preliminary site investigation phases. This very light and portable equipment can be easily carried by one technician and may be adapted to obtain various types of soil samples and *in situ* tests (Adestam, 1981).

7.5.3.3 Test Pits. Test pits and trenches may be excavated by hand or by conventional earth-excavating equipment to provide detailed examination of near-surface geological conditions (Figure 7-12). The technique is utilized for such purposes as determining geologic contacts, presence of faulting, preliminary slope stability estimates, and the recovery of bulk samples for laboratory testing. In addition, the test pit may serve as a basis for conducting *in situ* tests such as in-place density or water percolation determinations.

The U.S. Occupational Safety and Health Administration (OSHA) prohibits personnel entry into a test pit extending more than 1.5m (5.0 ft.) below ground surface or any pit displaying evidence of instability, without proper sheeting and bracing.

7.5.3.4 "ODEX" Drilling System. A drilling system currently used in the construction industry for installing earth anchors and tiebacks, has the potential to advance unsampled, cased holes in overburden soils at remarkable rates under favorable conditions. The procedure employs conventional air-operated, percussion drilling equipment utilized in the industry for decades. The standard percussion drilling equipment has been modified by the Swedish firms of Atlas Copco and Sandvik Coromant to facilitate the installation of heavy duty, removable casing for borehole stabilization in conjunction with the drilling operation. The modified system is referred to as the ODEX system (Figure 7-13).

Many indirect subsurface exploration techniques, such as downhole geophysical logging, borehole seismic surveys, and numerous other exploratory procedures, require a small diameter hole or holes within which to work. In addition, instrumentation, such as

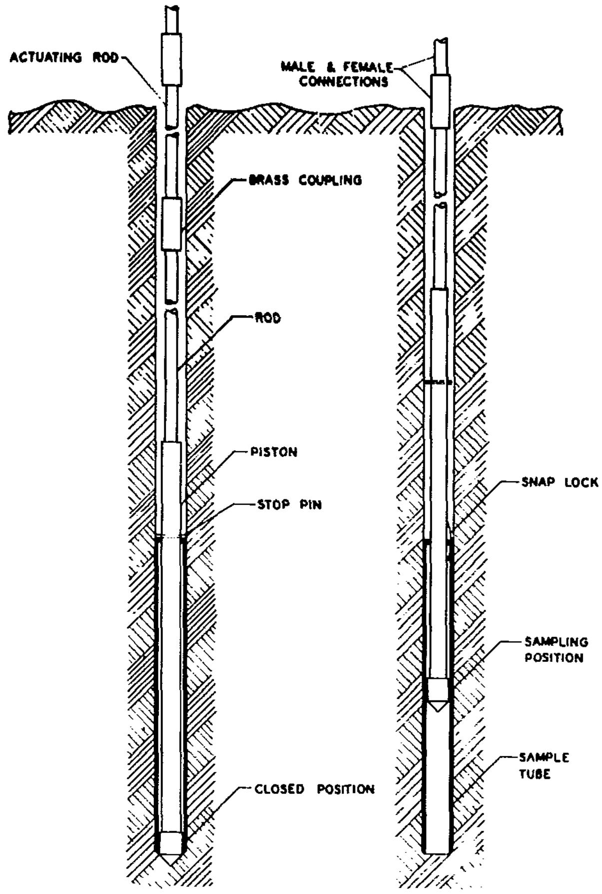


Figure 7-11. Retractable piston sampler (Davis type). (From "Acker Soils Sampling Tools Catalogue," Acker Drill Co.)

inclinometers, extensometers, observation wells and piezometers, require small diameter holes into which the instruments are installed. For many of these methods to be cost effective, it is essential to develop a procedure for drilling small diameter, uniform holes rapidly and at minimum cost. Through the use of such holes and indirect logging techniques, in combination with conventional sampled test borings, a thorough definition of subsurface conditions can be developed at minimal cost.

The ODEX drilling equipment consists of a rotary percussion drill rig equipped with a specially designed Sandvik drill bit (Figure 7-14).

The drill bit consists of three movable parts and is available in three different sizes:

	ODEX 76		ODEX 115		ODEX 127	
	mm	in.	mm	in.	mm	in.
Pilot bit	70	2.75 in.	109	4.31 in.	109	4.31 in.
Reamer bit	20	3.78 in.	152	6 in.	162	6.38 in.

Figure 7-12. Machine-excavated exploration pit, as shored in accordance with OSHA requirements. (A.W. Hatheway)

As the pilot bit drills the overburden materials at the bottom of the hole, drill rod rotation automatically swings out the eccentric reamer which enlarges the hole so the casing can advance behind the ODEX bit (Figure 7-15). A portion of the impact energy is transferred from the rock drill by way of a shank adapter to a driving cap above the casing which is advanced without rotation.

When the drilling is completed, the drill bit is rotated in the opposite direction, aligning the eccentric reamer with the drill bit. This allows the drill tools to be withdrawn into the casing (Figure 7-16). If the ODEX hole has penetrated into solid rock, drilling can continue with conventional equipment through the casing tube (Figure 7-17). Inexpensive, smaller diameter plastic casing may be lowered through the temporary heavy duty steel casing, which is removed after completion of the drilling, for future monitoring and instrumentation installation purposes.

As with any drilling system, there are inherent limitations with the technique and driller expertise is essential. Excessive equipment noise and the ten-

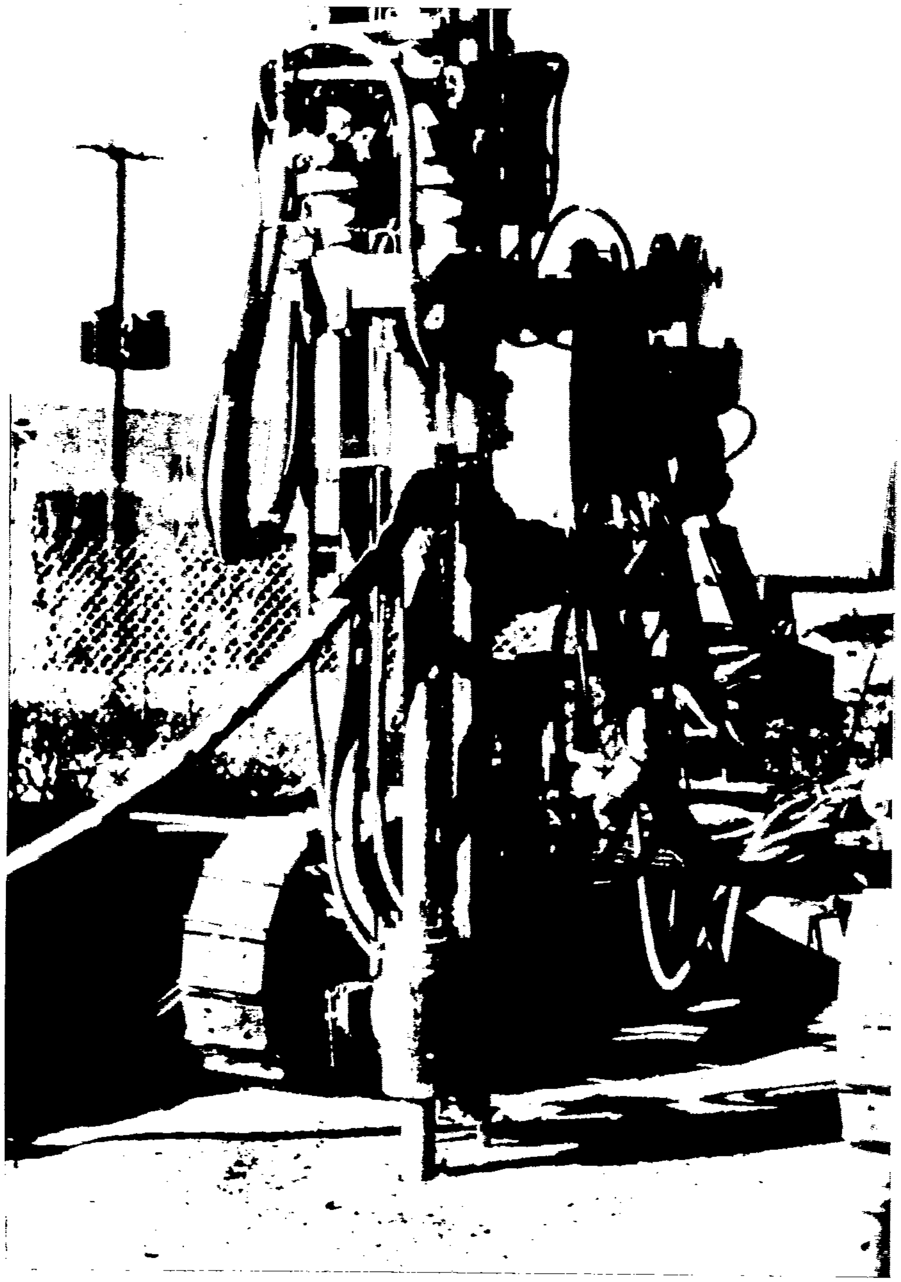


Figure 7-13. ODEX equipped Chicago pneumatic G900 "Air Track" with an Atlas COPCO double acting hammer. (Haley & Aldrich, Inc.)

dency of the ODEX bit to plug in cohesive materials or granular materials with an excessive amount of fines must be considered when applied to specific site conditions.

7.5.3.5 Horizontal Drilling Systems. Drilling equipment which is specially designed for installing horizontal drains and tiebacks may be used for determining general soil conditions in embankments or vertical faces where accessibility might prevent vertical borings. Conventional sampling is very difficult or impractical using this technique, but various rotary sampling devices could be adapted to obtain samples, if required. Although the depth penetration is a function of the material being penetrated, the Acker "Holegator" is rated for 58M (190 ft.) using 150mm (6-in.) diameter hollow stem auger of 122M (400 ft.) using 75mm (3-in.) diameter BW flush-joint casing (Figure 7-18).

Long distance horizontal borehole drilling tech-

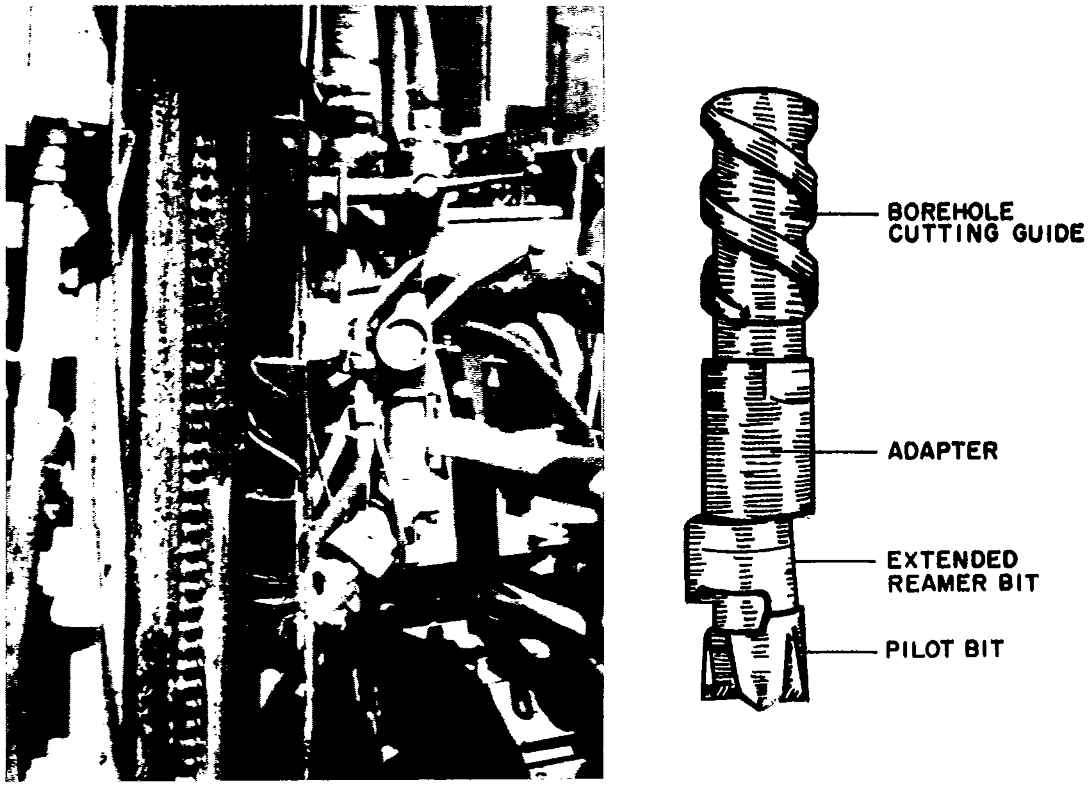


Figure 7-14. ODEX 76 bit with extended reamer. (Haley & Aldrich, Inc.)

niques are in various stages of development as potential alternatives to vertical borings for underground structures.

Horizontal alignment drilling could provide valuable information at locations where the geological structure is primarily vertical and the proposed excavation extends very deep underground, or in heavily developed urban areas where surface access and disruption would be a major consideration.

As with any newly developing technique, difficulties remain; these include direction control, penetration rates, lack of experienced drillers, and lack of

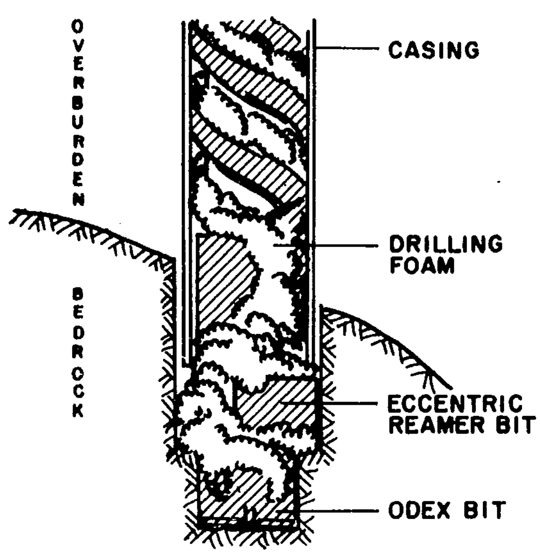


Figure 7-15. ODEX bit advancing with eccentric reamer extended and casing following. (Haley & Aldrich, Inc.)

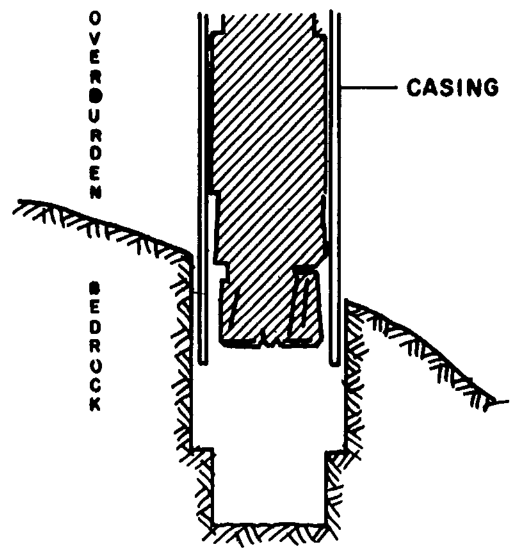


Figure 7-16. ODEX bit withdrawn with eccentric reamer retracted. (Haley & Aldrich, Inc.)

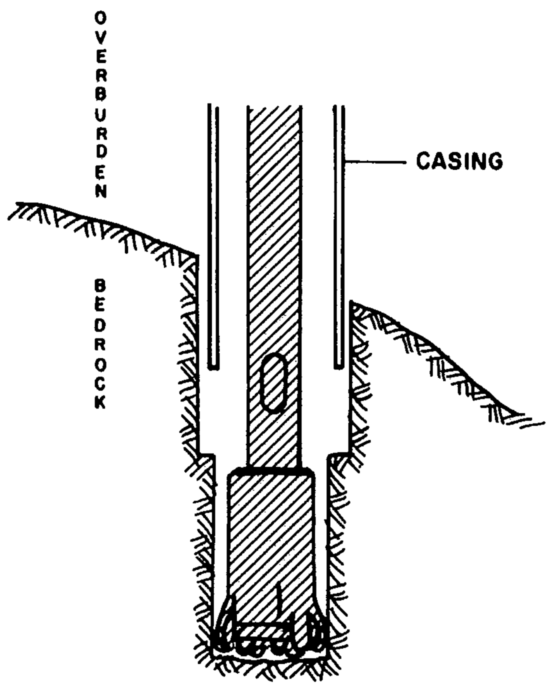


Figure 7-17. Conventional rock drilling in ODEX borehole. (Haley & Aldrich, Inc.)

equipment designed to perform the work. At present, the major disadvantage of horizontal drilling is the excessive cost, which, in part, is due to its low demand as a viable exploration tool.

7.5.3.6 Underwater Drilling Equipment. Subsurface explorations which are located within bodies of water are usually conducted from the surface, employing floating rafts or barges supporting conventional drilling equipment, which are anchored into position. Special "jack-up" drilling platforms are also available if the surface conditions are such that stability of the floating equipment cannot be maintained for drilling purposes.

Consideration may also be given to specially designed underwater hydraulic drill rigs which are set on the bottom from support vessels and operated by driller-divers, (Figure 7-19).

Samples may be obtained by rotary, vibratory or gravity coring procedures and *in situ* testing such as penetrometer and vane shear may be conducted. Although these underwater drill rigs are usually associated with deep ocean area investigations, their use may be advantageous for a specific site situation.

7.6 SOIL SAMPLING

Once the selection of the test boring methodology has been determined based on the anticipated subsurface geological conditions, the types of soil samples required for engineering analysis and their method of recovery is selected.

The numerous sampling devices based on the type

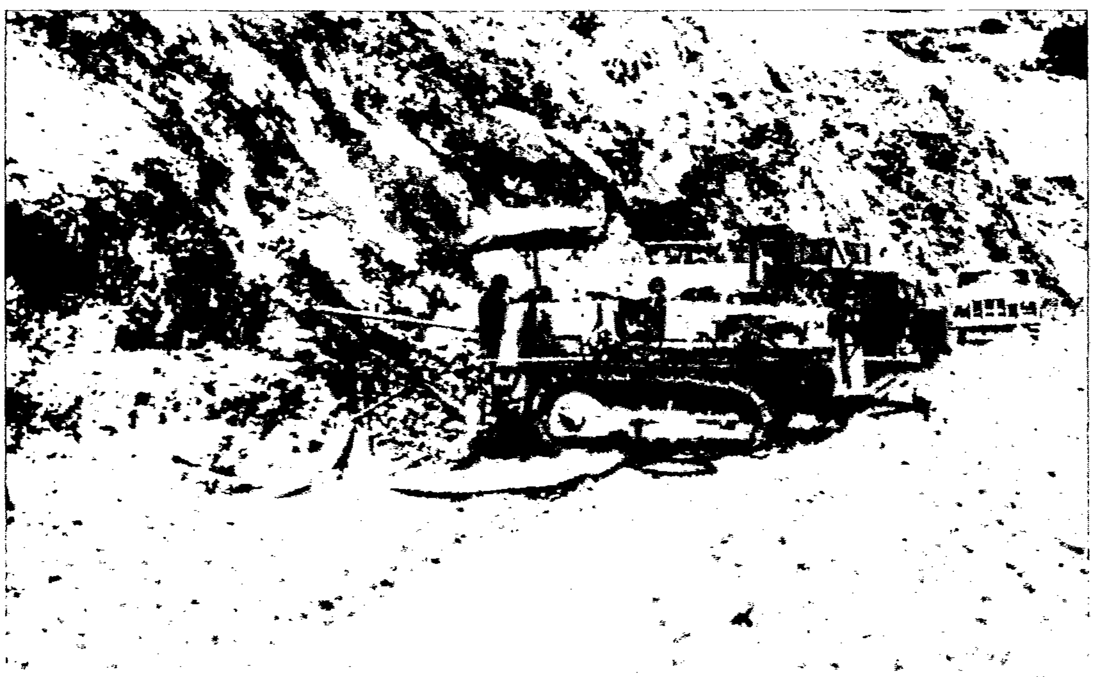


Figure 7-18. Horizontal drilling machine employed for installation of slope-stabilizing drain system. (A.W. Hatheway)

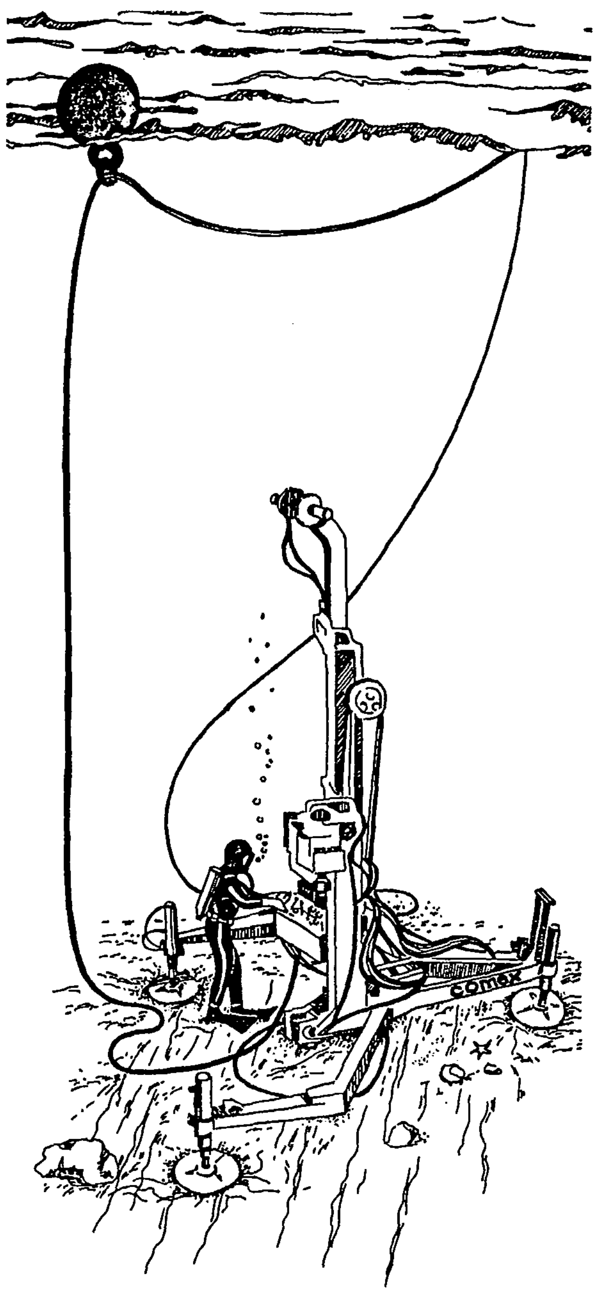


Figure 7-19. COMEX data underwater drill rig. (From "Etude de Sols en Mer," COMEX Data)

of sample they are capable of obtaining, are divided into two broad categories:

- Disturbed samples
- Undisturbed samples

A disturbed sample is a representative sample of a selected geological unit which has undergone structural alteration or contamination by the sampling operation. These types of samples are used for classification purposes and are obtained primarily by "open drive" samplers. Borehole cuttings and other displacement type samples would also be classified as "disturbed," but only semi-representative.

Undisturbed samples are those which have been obtained by methods which minimize disturbance and are suitable for laboratory performance tests. A completely undisturbed sample cannot be obtained with present technology, and any undisturbed sample may become "disturbed" during subsequent handling or transportation.

As with any subsurface exploration program, the specific subsurface geological conditions in the project area will dictate the most applicable method of recovering representative samples for engineering analysis.

7.6.1 "Wash" Sampling

The most basic and generally not representative sampling method, consists of recovering borehole cuttings from a variety of drilling procedures for examination and classification. The borehole cuttings will give only a general picture of the subsurface conditions, but intermixing of the various strata may lead to erroneous interpretations. Borehole cuttings will furnish supplemental information between conventional sample locations and should be closely monitored during penetration of the borehole.

7.6.2 Split-Barrel or Split-Spoon Open Drive Sampling

The open drive "split-spoon" sampler and its associated Standard Penetration Test (SPT) is the primary method of obtaining representative samples for foundation analysis. The split-spoon, which is cut into two longitudinal sections is driven into the soil at the bottom of the borehole. The recovered sample is removed for classification and preservation in the event that additional reference or laboratory testing is required. The split-spoon sampler is available in a variety of sizes and lengths. Various baskets, sleeves, or "trap doors" can be added to the sampler to assist in the retention of the sample during the recovery process. Figure 7-20 summarizes the more popular sizes of the basic split-spoon sampler.

Split-spoon samplers should be equipped at the top with a reliable check valve and should have a minimum inside sampling length of .5M (1.5 ft.). A recovery of less than 3M (1 ft.) is generally not considered as an acceptable sample in fine grained or cohesive soils. A second sample may be necessary immediately below the unsuccessful recovery.

Split-spoon drive samples should be obtained at or

STANDARD SAMPLER*

SIZE	SHOE I.D.	CON-NEC-TION	SPLIT SECTION LENGTH
2" O.D. x 1-1/2" I.D.	1-3/8"	AW	12", 18", 24"
2-1/2" O.D. x 2" I.D.	1-7/8"	AW	12", 18", 24"
3" O.D. x 2-1/2" I.D.	2-3/8"	NW	12", 18", 24"
3 1/2" O.D. x 3" I.D.	2-7/8"	NW	12", 18", 24"
4 1/2" O.D. x 4" I.D.	3-7/8"	NW	18", 24"

*ALSO AVAILABLE IN SOLID BARRELS UP TO 60 INCHES IN LENGTH AND HINGED TYPE SPLIT TUBES

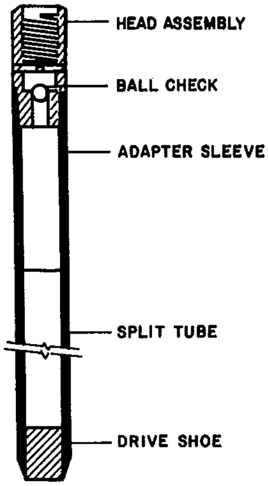


Figure 7-20. Standard split-spoon sampler. (From "Field Evaluation of Advanced Methods of Subsurface Exploration for Transit Tunneling," U.S. Department of Transportation.)

near the ground surface, at the beginning of every change of stratum, and at those intervals required by organization policy. At the sampling points, advancement of the bore hole should be stopped. Casing, if used, should be advanced as required, but only to a depth no greater than the maximum depth of jetting and chopping or drilling, and all of the material should be removed from inside the casing or borehole. When the driven casing method is used, water is generally employed to clean out the casing before sampling, and side discharge bits are used for such cleaning.

A variety of methods and equipment for obtaining the measure of penetration resistance have been standardized (ASTM D-1586; AASHTO T-206). The Standard Penetration Test (SPT) consists of counting the number of blows required to drive a 51 mm (2-in.) O.D. × 35 mm (1.38 in.) I.D. split-spoon sampler a distance of .3 M (1 ft.) with a 64 kg (140-lb.) hammer free falling .7 M (2.5 ft.). The sampler is usually driven a total of .5 M (1.5 ft.) and the blows are recorded per 150 mm (6 in.) of penetration. The penetration resistance (N) is determined by adding the second and third 150 mm (6 in.) penetration resistance blow counts. A driving rate of 100 blows per .3 M (1 ft.) penetration is normally considered "refusal," however, this criterion may be varied depending on the desired information. In excessively dense soils where the Standard Penetration Test is not applicable, or when larger diameter samples are required, heavier drive hammers and solid sample spoons may be utilized to obtain representative samples. As in the drilling techniques discussed previously in this Manual, heavy-duty 67 mm (2.63 in.) N-size drill rods should be employed in obtaining drive samples in the deeper and larger diameter borings for stability during the driving operations.

The relationship between the consistency or relative density of the soils and the dynamic penetration resistance is summarized in Table 7-5.

Table 7-5
Soil Density/Consistency Descriptors

Non-Cohesive Soils		Cohesive Soils	
Relative Density	Number of blows per 0.3 M (1 ft.) (N)	Consistency	Number of blows per 3 M (1 ft.) (N)
Very loose	0– 4	Very soft	0– 1
Loose	5–10	Soft	2– 4
Medium dense	11–24	Medium stiff	5– 8
Dense	25–50	Stiff	9–15
Very dense	51–	Very stiff	16–30
		Hard	31–60
		Very hard	61–

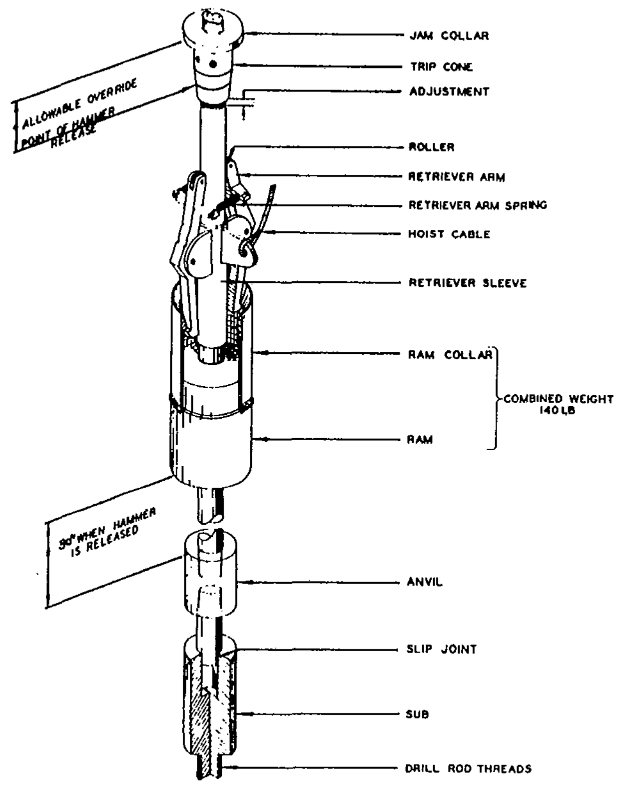
When the relative density of the soil is critical (such as for liquefaction studies) automatic trip hammers weighing 64 kg (140 lb.) are commercially available which ensure a .7 M (2.5 ft.) free-fall drop (Figure 7-21).

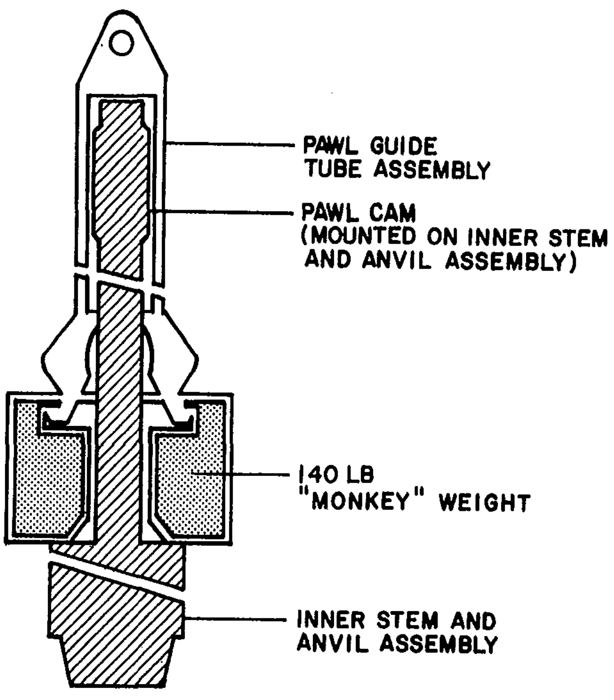
However, recent studies of the SPT (Kovacs and Salomone, 1982; Schmertmann, 1977) simply advocate the increased standardization of the SPT for liquefaction studies. The major recommendations include the use of a rope and drum system to lift the 140 lb. weight, with two wraps of the rope around the drum, and the use of drilling mud to support the sides of the borehole.

A factor of particular importance in using borehole tests like the SPT, especially in cased boreholes, for liquefaction studies is maintaining the fluid level in the borehole near the top of the hole. Failure to do so may result in the soil at the bottom of the hole becoming unstable because of upward seepage of groundwater into the hole, and soil may actually "blow" into the casing. Measured parameters (e.g., blowcounts) may be much lower than expected because of the disturbed soil being sampled.

Obtaining representative samples for engineering analysis, using open split-spoon samplers, is standard procedure in geotechnical engineering. The principal advantage of the open drive samplers is their simplicity in construction and operation and their relative economy for evaluating *in situ* soil parameters through widely accepted empirical correlations. In addition, they recover representative specimens suitable for classification and for certain laboratory testing. However, there are limitations in the use of open drive sampling, and reasonable and careful evaluation of the data must be exercised.

Several studies have been conducted (deMello, 1971), (Schmertmann, 1974), which summarize the advantages and disadvantages of the Standard Penetration Test. Some of the disadvantages are that the sample may become highly disturbed or contaminated during penetration and may not be truly representative of the stratum sampled. Also, excessive hy-





Army Corps of Engineers type.

(From EM1110-2-1907)

British Pilcon type.

(Pilcon Engineering, Great Britain)

Figure 7-21. "Automatic" Trip Hammers.

drostatic pressures or penetration friction may indicate erroneous *in situ* relative densities. These acknowledged inconsistencies do not decrease the practical importance of open drive sampling. Precise geotechnical data from penetration resistance is not within the capability of the procedure. Its purpose is to obtain an approximate comparison of the *in situ* geological conditions and to provide samples for soil classification. Refer to Appendix B for specific details on conducting the Standard Penetration Test.

7.6.3 Thin-Wall Tube Sampling

A method of obtaining larger and less disturbed samples of soil was introduced by H.A. Mohr in 1937. This method consists of pressing thin, seamless tubing into cohesive soils of soft consistency for preservation and supplemental laboratory testing. Although loose, fine grained granular soils may be sampled with this method, sample retention may be a problem, unless the sampler device is equipped with a piston which creates a vacuum and helps retain the sample in the tube.

The thin-walled tubing, more commonly referred to as Shelby tubing from the manufacturer's tradename, may be any thin-wall tubing that is beveled to form a tapered cutting edge and drawn in slightly to reduce sample friction against the wall of tube during penetration. The tubes are usually cut in .6–.9 M (2–3 ft.) lengths and coated with a lacquer or other rust preventative solution.

The DCDMA has established standard dimensions for thin wall tubing which are summarized in Table 7-6.

The thin-wall tubing may be used with a variety of sampling devices to obtain representative and relatively undisturbed samples. As with any sampling device or method, variations in design, operation and ability to recover the sample is dependent upon the character of the materials being sampled. Standard guidelines for thin-wall sampling have been established in AASHTO T207 and ASTM 1587. Detailed procedures are also included in the USBR "Earth Manual."

7.6.3.1 Thin-Wall, Open-Drive Sampler. The most common and simplest method of obtaining relatively undisturbed samples consists of pressing an open, thin-walled tube into the desired stratum at the bottom of the borehole. This method does not utilize any sample retention devices, although the sampler head is equipped with a ball check valve and vents to relieve air and water pressure buildup within the tube.

The thin-wall open-drive sampler, which has the advantage in its simplicity of construction and operation, also has several major disadvantages as follows:

1. Disturbed and intermixed soil materials from the bottom and sides of the borehole may enter the tube as it is lowered into position.
2. Penetration of the sampler under the weight of the drill rods may occur in very soft or loose materials, preventing accurate measurements for controlled sampler penetration.
3. Total or partial sample recovery is difficult without a supplemental retention system.
4. Hydrostatic pressures may disturb the sample during penetration or totally prevent the sample from entering the tube.

The majority of these disadvantages may be eliminated by using one of the several varieties of the stationary piston sampler and/or the use of borehole casing to eliminate sampler contamination.

Table 7-6
DCDMA Standards for Thin Wall Tubing

Size		Dimensions			
		I.D.		O.D.	
mm	in.	mm	in.	mm	in.
51	2.0	48	1.878	51	2.000
54	2-1/8	51	2.004	54	2.125
76	3.0	72	2.838	76	3.000
89	3-1/2	85	3.333	89	3.500
127	5	106	4.170	127	5.000

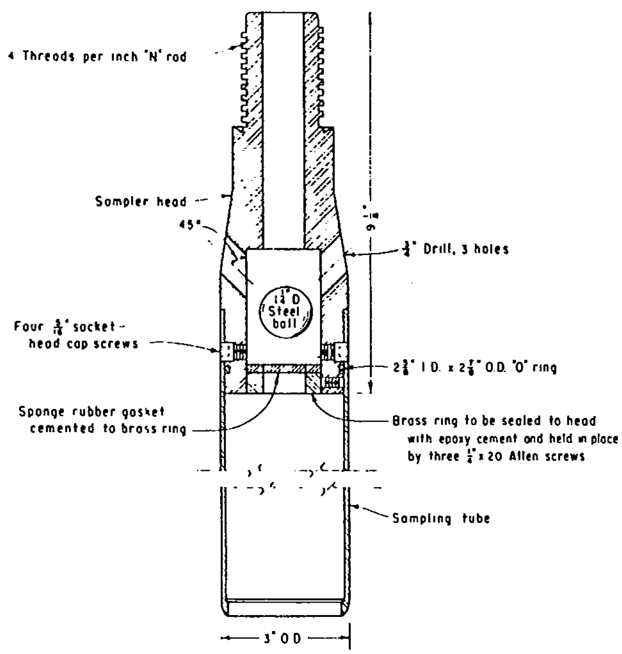


Figure 7-22. Thin-wall open drive sampler.

7.6.3.2 Mechanical Stationary Piston Sampler. The Mechanical Stationary Piston Sampler, first developed by John Olsson of Sweden in 1922 and modified by Dr. M. Juhl Hvorslev, of the U.S. Army, Corps of Engineers, is similar in construction to the thin-wall, open-drive sampler discussed in Section 7.6.3.1. Several major improvements in the design of the sampler include the addition of a sealed piston and locking cone in the head assembly to prevent the piston from moving downward (Figure 7-23). The piston can be locked and fully sealed at the bottom of the thin-wall tube so that it can be lowered into the borehole without contamination.

Once the sampler is in position, the piston, through a series of small diameter inner actuating rods, is locked to the drill rig or the casing and pressure is applied to the outer drill rods which forces the thin-wall tube down from the "Stationary" piston. When the full press is completed (.6 m; 24 in.) any pressure buildup is released through a small hole in the actuating rods. The tight seal of the piston also creates a vacuum on the sample which aids in sample retention.

The sampler is rotated two full turns to shear off the soil at the bottom of the tube and withdrawn very carefully from the borehole. A short waiting period before and after shearing allows additional skin friction to develop between the sample and the tube, which will further minimize sample loss during recovery.

Generally, Stationary Piston Samples are obtained with large hydraulic operated drill rigs. However, a series of pully arrangements, referred to as a "Christmas tree" can be attached to the borehole casing for reaction purposes, and be pressed with any type of power take-off unit (Figure 7-24).

The Mechanical Stationary Piston Sampler is a significant improvement over the thin-wall open-drive sampler, in that it decreases sample disturbance and improves recovery; the apparatus is, however, more complicated and time consuming to operate than open-drive samplers. However, it can be operated in uncased boreholes, which can be a major time saving factor.

7.6.3.3 Floating Piston Sampler. The Floating Piston Sampler is generally similar in appearance to the

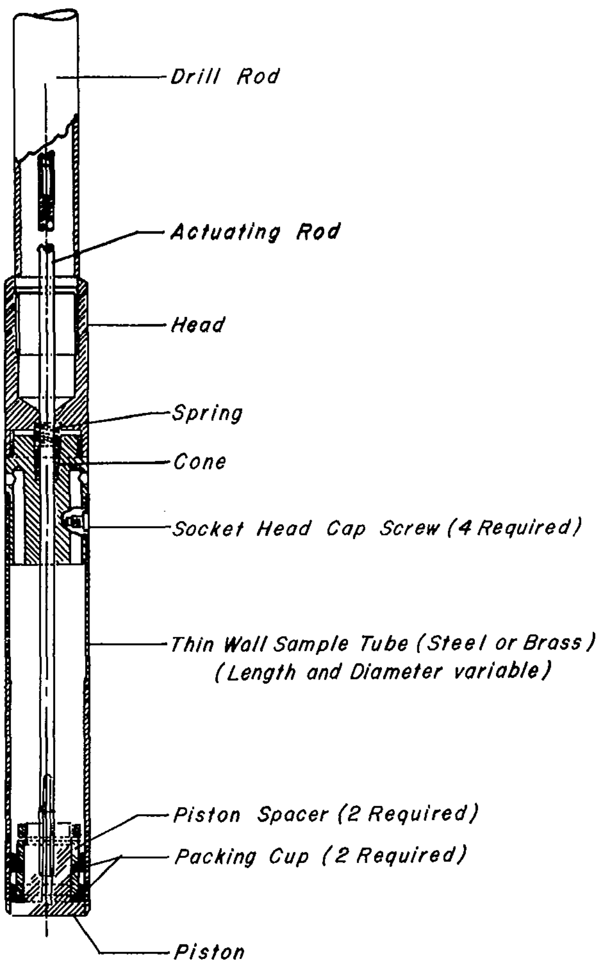


Figure 7-23. Mechanical stationary piston sampler (Acker type). (Haley & Aldrich, Inc.)

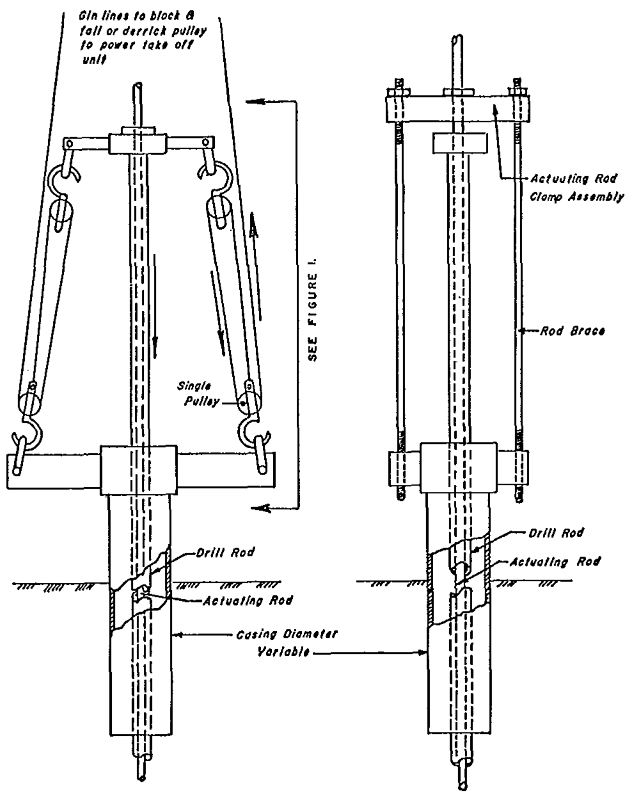


Figure 7-24. Mechanical stationary piston sampler "Christmas Tree" assembly. (Haley & Aldrich, Inc.)

Stationary Piston Sampler, except that the actuating rods which connect to the piston are eliminated, thus allowing the piston to "float" within the assembly. The ability of the piston to "float" is a disadvantage in soft soils and the proper use of this equipment is limited to sampling stiff or hard cohesive soils.

In operation, the piston is manually set flush with the bottom of the thin-wall tube. Providing that the packing between the piston and the tube is tight, the piston will remain in this position while it is lowered to sampling elevation in the borehole, providing performance similar to stationary piston sampling. As the sampling tube is pressed into the soil to be sampled, the piston moves upward relative to the tube.

In soft cohesive soils, the force required to make the piston move along the tube may be excessive, relative to the shear strength of the soft material being sampled. Thus, the soil to be sampled may be compressed or otherwise disturbed by the sampling process, as the tube with piston in the flush position is pressed into the soil. If sufficient resistance is not provided by the material being sampled, the piston will remain in the flush position, soft soil will simply be displaced, and no sample will be obtained.

7.6.3.4 Retractable Piston Sampler. The Retractable Piston Sampler is similar to the Stationary Piston Sampler in that it retains the inner rod which operates the piston. However, even though it is much simpler to operate, it loses many of the advantages of the Stationary Piston Sampler. The sampler, with the piston at the bottom of the tube to prevent materials from entering the sampler, is lowered to the bottom of the borehole. The piston is then partially retracted into the tube and locked in place. This is accomplished by a series of extension rods connected to the surface (Figure 7-25).

The sampler assembly is pressed into the soil and the piston is then fully retracted which closes the vents in the head assembly preventing hydrostatic pressures from forcing the sample out of the tube during subsequent recovery. Although the assembly and sampling operations are similar to the Stationary Piston Sampler, the tight seals obtained with the packer system of the stationary sampler cannot be duplicated with the Retractable system. There is a tendency for soil and fluids to pass around the piston into the sample chamber creating excess pressures which jam the piston and allows only the recovery of the disturbed materials. In addition, the vacuum created during penetration with the Stationary Piston Sampler which improves recovery, is not possible with this procedure. The Stationary Piston Sampler can be operated as a retractable piston sampler, but the value of this procedure modification is questonable. A small hand

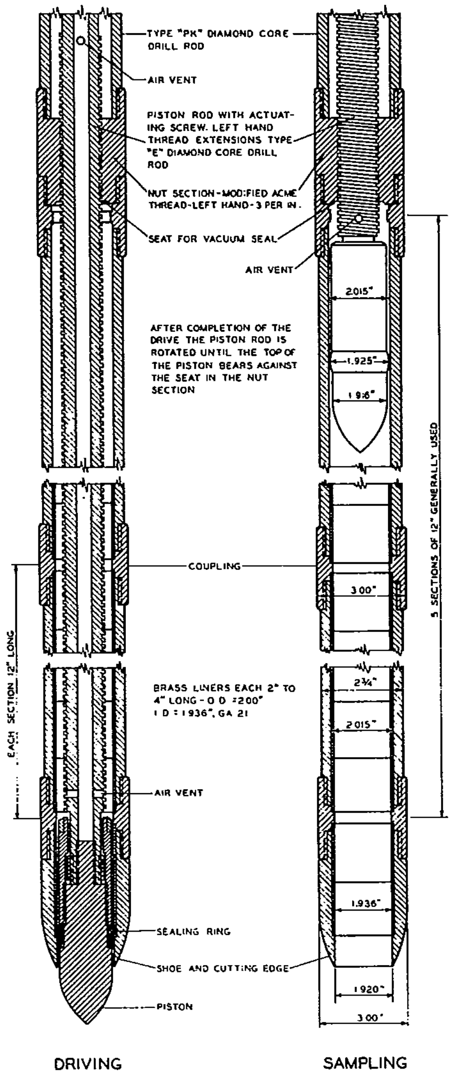


Figure 7-25. Porter or "California" retractable piston sampler. (From "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes," M.J. Hvorslev)

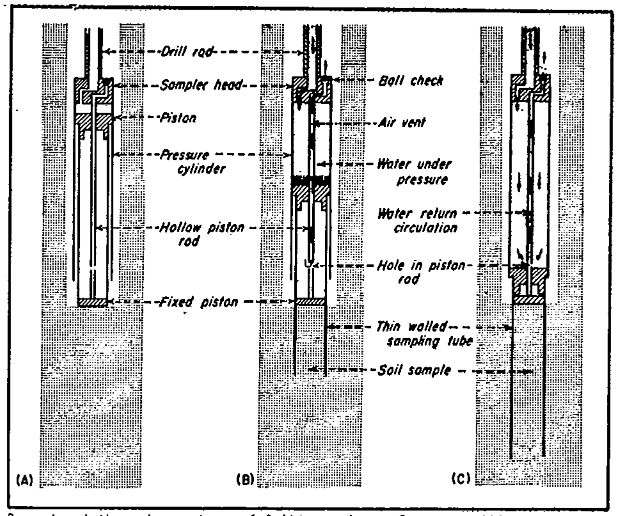
operated, portable model of the retractable piston sampler is discussed in Section 7.5.3.2.

7.6.3.5 Hydraulic/Pneumatic (Osterberg) Piston Sampler. The hydraulic Piston Sampler is designed to obtain undisturbed samples of soft and potentially sensitive soils in uncased boreholes. The design of the sampler varies considerably from the Stationary Piston Sampler, in that it consists of an inner thin-wall sampler tube and outer pressure cylinder. In the sampling position, a movable piston is attached to the top of the sampling tube and a stationary piston rests on the soil to be sampled. The sampler is activated by pumping fluids or gas through the pressure cylinder, which drives the upper piston and sampling tube down over the lower piston into the soil a fixed distance (Figure 7-26). Then the piston is withdrawn with the sample from the borehole. The Osterberg is adaptable to both 80 mm (.3 ft.) and 130 mm (.5 ft.) diameter, thin wall sampling tubes.

The self-contained and very portable aspects of the Hydraulic/Pneumatic Piston Sampler make it an ideal sampling device in swamps and areas of difficult access for large, conventional drilling equipment.

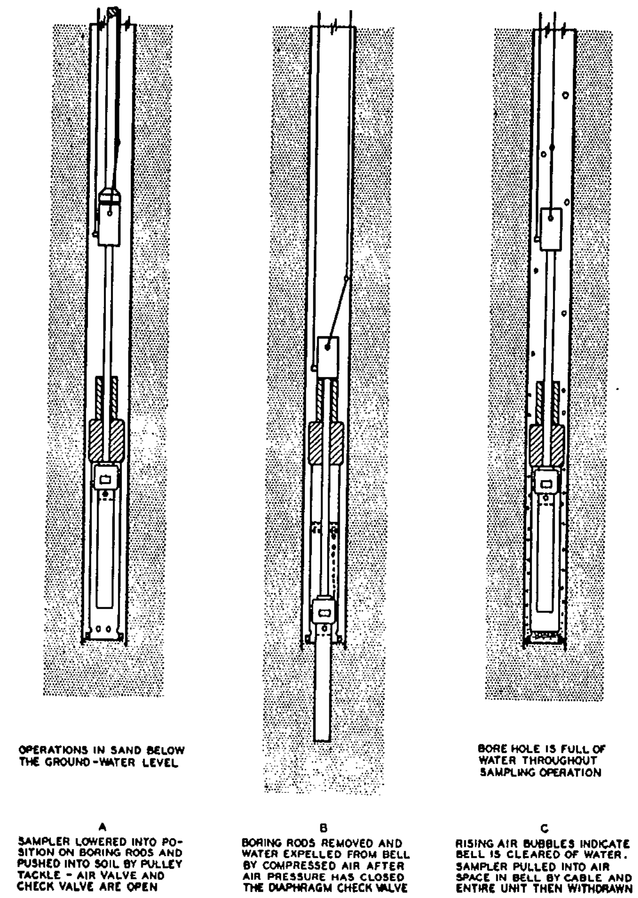
7.6.3.6 Bishop Sand Sampler. The Bishop Sand Sampler developed by A.W. Bishop, in the United Kingdom, in 1948, utilizes both mechanical and pneumatic methods for recovering loose, saturated sands below the water table.

The Bishop sampler consists of an inner thin-wall sampler tube and outer pressure cylinder. The sampler is pressed into the soil by conventional mechanical methods. Compressed air is then pumped through a specially designed head assembly which forces water from the outer cylinder and closes the pressure relief valves in the sampler. The sampler tube is then



Reproduced through courtesy of Soiltest, Inc., Evanston, Ill.

Figure 7-26. Hydraulic Osterberg Piston Sampler Operation. (From "Geologic Site Exploration" U.S.D.A. Soil Conservation Service)



A. W. BISHOP: GEOTECHNIQUE, DEC. 1948, P.129

Figure 7-27. Bishop sand sampler operation. (From "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes," M.J. Hvorslev)

retracted into the outer cylinder and the entire unit is removed from the borehole (Figure 7-27).

A high degree of success has been reported in obtaining undisturbed samples of sand using this method (Bishop 1948).

7.6.3.7 Swedish Foil Sampler. A method to obtain long continuous undisturbed samples and minimize sample skin friction disturbance during sampler penetration was developed by the Royal Swedish Geotechnical Institute. Although this procedure has not met with wide acceptance in the United States its potential advantages in obtaining undisturbed samples should be considered. Continuous undisturbed samples up to 18 M (60 ft.) in length have been obtained using this method.

The foil sampler utilizes a lockable piston technique and the steel tube assembly has been modified to accommodate a chamber which contains up to 21 M (70 ft.) of steel foil coiled in strips. Each steel foil strip is approximately 11 mm (0.43 in.) wide and may vary in thickness from .06 to 0.2 mm (.0025 to .008 in.) (Figure 7-28).

As the sampler is pressed into the soil, the steel coils unroll and axially encase the sample as it enters the tube so that there is no relative movement between the sample and the foil. The inner sample tube sections, which are usually 3 m (10 ft.) in length, are added to the string for the desired penetration depth. The sampler is removed from the borehole by uncoupling the sections and cutting the liner at the desired lengths and sealing the ends, or the sample can be easily removed in the field for observation by pulling on the steel foil.

As with any sampling system, there are limitations to the procedure; however, recent refinements in the equipment and technique have overcome many of the objections. These include automatic sample retainers, advancing the sampler by jetting with water or mud; and the application of rotary core barrel techniques to facilitate deeper and easier penetration (Kjellman, et al, 1950).

7.6.4 Rotary Core Barrel Sampling

A variety of core barrels, which were originally developed for drilling and sampling bedrock, have been modified or adapted to obtain "undisturbed" overburden samples in very dense or partially cemented soils. These core barrels are used when the more conventional thin-wall samplers (Section 7.6.3) cannot penetrate the selected geological unit.

There are many local variations in the type and mechanics of these core barrels which are commercially available under a variety of trade names.

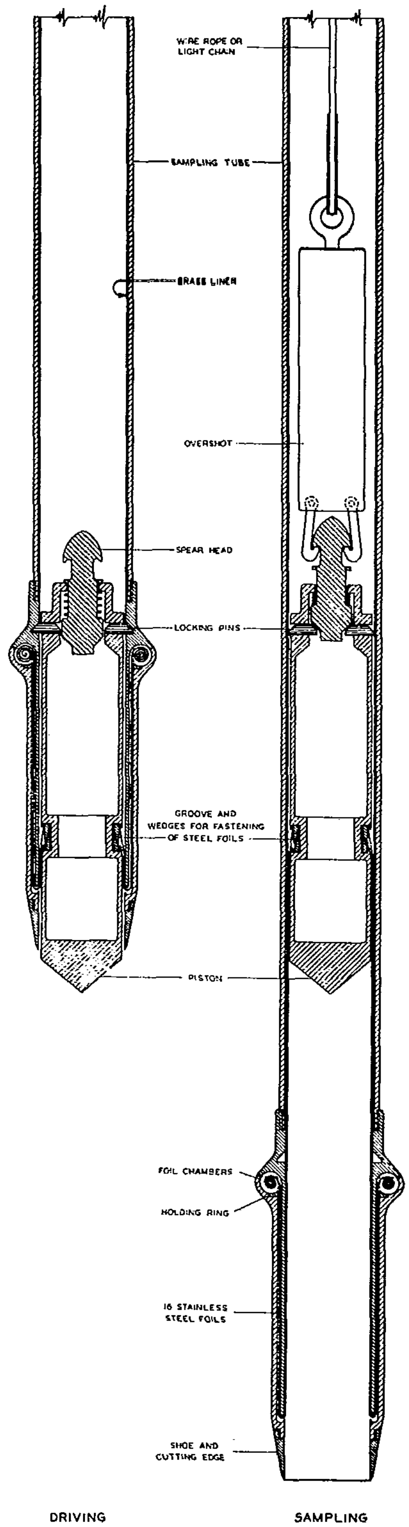


Figure 7-28. Swedish foil sampler. (From "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes," M.J. Hvorslev)

Single wall or single tube core barrels equipped with saw-tooth cutter bits have been used to some extent in sampling soils. However, the samples are usually disturbed by intermixing, swelling or contamination with drilling fluid. Core barrels equipped with non-rotating innerliners are more suitable for overburden sampling and several varieties are discussed in the following sections.

7.6.4.1 Denison Sampler. The Denison core barrel was developed in 1939 by the Denison District, U.S. Army Corps of Engineers and is presently manufactured under further patent-right developments held by the Acker Drill Company, Inc., Scranton, Pennsylvania.

The Denison Sampler is designed to recover undisturbed, thin-wall samples in dense sand/gravel soils, hard clays, partially cemented soils or soft and weathered rock. The sampler consists of a double-tube, swivel-type core barrel with a non-rotating inner thin-wall steel or brass liner designed to retain the sample during penetration and subsequent transportation to the laboratory (Figure 7-29).

The inner liner tube of the Denison has a sharp cutting edge which can be varied to extend from zero to about 76 mm (3 in.) beyond the outer rotating cutter bit. The amount of extension can be varied by means of interchangeable saw tooth cutter bits which are preselected depending on the anticipated formation which is to be sampled. The maximum extension is used in relatively soft or loose soils and a cutting edge flush with the coring bit is used in hard or cemented formations. An important feature of the Denison Sampler is a system of check valves and release vents which by-pass the hydrostatic pressure buildup within the inner sampling tube, improving sample recovery and minimizing pressure disturbance of the sample.

The Denison Sampler is rotated into the formation in the same manner as conventional rock coring procedures, in either a cased or mudded borehole. The Sampler is designed for use with water, mud or air and is available in five sizes, ranging from 75 mm (2.94 in.) to 197 mm (7.75-in.) O.D. A schematic drawing of the Acker-type Denison Rotary Core Barrel Sampler is shown on Figure 7-30.

Figure 7-29. Denison core barrel sampler. (Haley & Aldrich, Inc.) (Courtesy Texas Department of Highways)

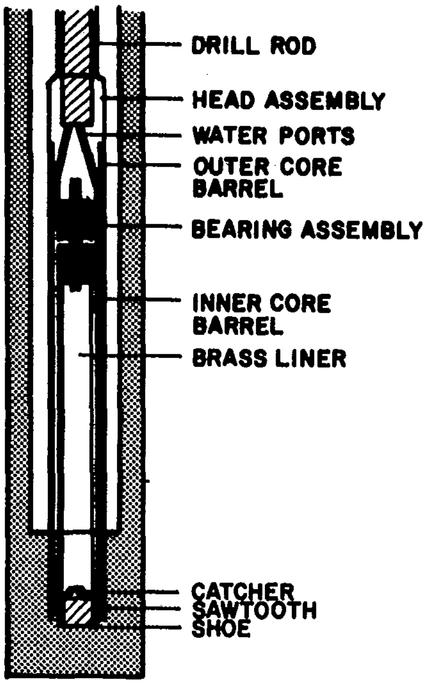


Figure 7-30. Denison core barrel sampler operation. (From "Field Evaluation of Advanced Methods of Subsurface Exploration for Transit Tunneling," U.S. Department of Transportation)

The Denison Sampler is not a practical tool for sampling loose sands or soft clays, as the sample retention devices are usually inadequate for these materials. The presence of cobbles and boulders will present major difficulties for penetration and recovery. The saw-tooth bit, with which the Denison is usually equipped, is not capable of coring hard boulders which may cause collapse of the inner sampler tube if it is in an extended position.

7.6.4.2 Pitcher Sampler. The Pitcher Rotary Core Barrel Sampler is a modification of the Denison sampler which was developed by the Pitcher Drilling Company, Inc., Daly City, California in 1960. It is presently manufactured and distributed by Mobile Drilling Incorporated, Indianapolis, Indiana.

The Pitcher Sampler was also developed to recover undisturbed thin-wall samples in formations which are too dense for conventional thin-wall sampler penetration. The Pitcher Sampler consists of a single-tube, swivel-type core barrel with a self-adjusting, spring-loaded inner thin-wall sample tube which telescopes in and out of the cutter bit as the hardness of the material varies. This telescoping aspect eliminates the need to pre-select a fixed inner barrel shoelength as with the Denison Sampler.

The inner steel or brass thin-wall liner tube has a sharp cutting edge which projects a maximum of 150 mm (.5 ft.) beyond the saw-tooth cutter bit in its normal assembled position. As the sampler enters the borehole, a sliding valve directs the drilling fluid through the thin-wall sample tube for a thorough pre-flushing of the borehole. When the sample tube comes in contact with the bottom of the borehole, it telescopes into the cutter barrel and closes the sliding valve which diverts the drilling fluid to an annular space between the sample tube and the cutter barrel. This sliding valve arrangement allows the circulation of the drilling fluid to remove the borehole cuttings during sampling and prevents disturbance of the recovered sample by the drilling fluid.

The spring-loaded inner sample tube automatically adjusts to the density of the formation being penetrated. In very soft materials, it will extend as much as 150 mm (.5 ft.) beyond the cutter bit and as the formation density increases, the sample tube telescopes into the outer core barrel and compresses the control spring, which, in turn, exerts a greater force on the tube to insure adequate penetration. In extremely dense formations or obstructions, the sample tube will retract completely into the outer core barrel to allow the cutter bit to penetrate the obstruction.

The Pitcher Sampler is also rotated into the formation in the same manner as conventional rock coring procedures in either a cased or mudded borehole. The sampler is designed for use with either water or mud and is available in four sizes, ranging from 64 mm (2.5 in.) to 149 mm (5.875-in.) O.D. A schematic drawing of the Pitcher Sampler operation is shown on Figure 7-31.

The telescoping liner aspect of the Pitcher Sampler is a major advantage in highly variable formations, which prevents collapse of the sample tube. However, the Pitcher Sampler, like the Denison, is not capable of coring very competent cobbles and boulders.

7.6.4.3 Triple Tube Conversion Core Barrel Sampler. Recent modifications and improvements in conventional rock drilling core barrels allow interchangeable conversions from rock coring barrels to soil coring units. These core barrels, utilizing basic rock coring barrel design, combine the principles of the Denison or the Pitcher sampler. In addition, a third inner liner which retains the sample, further minimizes sample disturbance and improves recovery.

The Sprague & Henwood "MD" type soil sampling core barrel and the Acker Swelling Soil core barrel are two examples of improved core barrel design for sampling in overburden materials.

The various types of core barrels are discussed in more detail in Section 7.7.1.

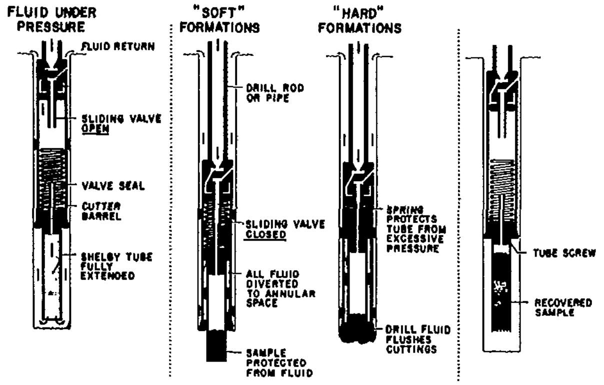


Figure 7-31. Pitcher sampler operation. (From "Soil Sampling and Equipment Catalogue," Mobile Drilling Inc.)

7.6.5. Block Sampling

One of the oldest, and considered by many as the most reliable, methods of obtaining undisturbed samples for laboratory testing, consists of cutting large blocks of soil from the natural, *in situ* formations. Although tests samples are usually obtained from sediments which display cohesion, either real or apparent, there are instances where granular soils have been satisfactorily obtained by lowering the water table in the sample area (Salomone 1978).

A test pit or shaft is excavated to the desired sample location and the soil is cut by hand in the shape of a projecting cube. The cube should be approximately 50 mm (2 in.) smaller in all dimensions than a wooden box in which the sample is to be encased for transport. The box should be constructed so that the top and bottom panels may be easily removed or replaced in the field.

Several layers of cheesecloth are carefully placed to avoid damage to the cube corners and edges. Melted microcrystalline wax is poured first into the bottom of the box. The sample is then placed, in a centered position into the box and wax is poured between the sample and the box and allowed to harden. Wax is then added to the top of the sample and the cover is attached.

The bottom of the sample is then cut away from the ground and the box containing the sample, is reversed. Cheesecloth wax and cover are added to the bottom of the box, completing the sampling and preservation of the block sample.

7.7 ROCK CORE SAMPLING

The primary objective of rock core sampling is to obtain continuous, undisturbed cores in the intact rock mass for evaluation of characteristics which may affect its performance as in excavations or as a construction material.

These characteristics include the following:

- Elevation
- Lithology
- Weathering
- Hardness
- Structure
- Permeability
- Discontinuities
- Mineralogy

The rock core samples which are recovered can be further evaluated in the laboratory for such additional engineering properties as compressive strength, elastic modulus and abrasion resistance. The completed rock core borehole may be tested and monitored to determine permeability, groundwater conditions, the presence of gas and squeezing or expansive properties of the rock. The borehole may be further utilized for *in situ* testing purposes, geophysical surveys and the installation of various types of monitoring equipment or instrumentation. Rock core sampling can provide substantial geotechnical information in the immediate vicinity of the borehole. However, rock core sampling usually provides only a limited amount of information about the overall rock mass, and this information must be extrapolated into engineering decisions for the entire formation.

Careful observation and evaluation during drilling and logging of the recovered core is essential to any site investigation program.

The rock coring procedures and equipment which were developed in 1863 by Leschot, a Swiss engineer, remain basically the same; a hollow steel tube equipped with a diamond bit is rotated into the rock surface. However, major improvements in the core barrels, diamond bits and associated equipment have created very sophisticated rock core sampling devices. Diamond rock drilling methods have been generally standardized by the American Society for Testing and Materials (ASTM D-2113). To facilitate standardization of equipment the Diamond Core Drill Manufacturers Association (DCDMA) has established standard sizes for bits, shells and casings. The various DCDMA size standards for core barrels and bits are summarized in Table 7-7.

The primary purpose of any type of core barrel is to recover the total amount of rock which is physically cored, in a relatively undisturbed state. When drilling in competent rock total recovery is rarely a problem; however, when the formation is highly weathered, fractured or soft, core recovery becomes poor. The strength and behavior of the rock mass is primarily

Table 7-7
DCDMA Core Barrel and Bit Diameter Standards
(inches)

Description	RX or RW	EX or EW	AX or AW	BX or BW	NX or NW	HX or HW	PX or PW	SX or SW	UX or UW	SX or ZW
Bit Set Normal I.D.	0.750	0.845	1.185	1.655	2.155	3.000	—	—	—	—
Bit Set Normal and Thinwall O.D.	1.160	1.470	1.875	2.345	2.965	3.890	—	—	—	—
Bit Set Thinwall I.D.	0.735	0.905	1.281	1.750	2.313	3.187	—	—	—	—
Shell Set Normal and Thinwall O.D.	1.175	1.485	1.890	2.360	2.980	3.907	—	—	—	—
Casing Bit Set I.D.	1.000	1.405	1.780	2.215	2.840	3.777	4.632	5.632	6.755	7.755
Casing Bit Set and Shoe O.D.	1.485	1.875	2.345	2.965	3.615	4.625	5.650	6.780	7.800	8.810

dependent upon the various inherent discontinuities; core which is not recovered may represent significant engineering implications.

The selection of the most practical core barrel for the anticipated bedrock conditions is important. The selection of the correct drill bit is also essential to good recovery and drilling production. Although the final responsibility of bit selection is usually the drilling contractor's, there is a tendency in the trade to use "whatever happens to be at hand." The selection of the diamond size, bit crown contour and number of water ports is dependent upon the characteristics of the rock mass and the use of an incorrect bit can be detrimental to the overall core recovery. Generally, fewer and larger diamonds are used to core soft formations and more numerous, smaller diamonds which are mounted on the more commonly used, semi-round bit crown are used in hard formations. Special impregnated diamond core bits have been recently developed for use in severely weathered and fractured formations where bit abrasion can be very high.

Typical rock core sampling involves the use of the open diamond bit. However, numerous types of coring and non-coring diamond bits are also available for use. An excellent summary of drilling equipment and bits is presented by W.L. Acker III in Chapters 10 and 11, "Basic Procedures for Soil Sampling and Core Drilling" (Acker, 1974).

There are a number of rotary core barrels which have been developed for a variety of formation conditions by several manufacturers. Rotary core barrels are manufactured in different types and sizes and have reached a high level of sophistication for improving the quality and quantity of sample recovery. By combining other types of modifications within the core barrel, the determination of formation structure and defects is accomplished.

7.7.1 Rotary Core Barrel Types

The Rotary Core Barrel is manufactured in three basic types: single tube, double tube, and triple tube. These basic units all operate on the same principle of pumping drilling fluid through the drill rods and core barrel. This is done to cool the diamond bit during drilling and to carry the borehole cuttings to the surface. A variety of coring bits, core retainers, and liners are used in various combinations to maximize the recovery and penetration rate of the selected core barrel.

The simplest type of rotary core barrel is the single tube, which consists of a case hardened, hollow steel tube with a diamond drilling bit attached at the bottom. The diamond bit cuts an annular groove or kerf in the formation to allow passage of the drilling fluid and cuttings up the outside of the core barrel. However, the drilling fluid must pass over the recovered sample during drilling and the single tube core barrel cannot be employed in formations that are subject to erosion, slaking or excessive swelling. Although the single tube is a very rugged core barrel and easy to operate, its limitations during sampling of both soil and rock are contributing to its declining application on geotechnical engineering projects (Figure 7-32).

The most popular and widely used rotary core barrel is the double tube, which is basically a single tube barrel with a separate and additional inner liner and is available in either a rigid or swivel type of inner liner construction (Figure 7-33). In the rigid types, the inner liner is fixed to the outer core barrel so that it rotates with the outer tube. In contrast, the swivel type of inner liner is supported on a ball bearing carrier which allows the inner tube to remain stationary, or nearly so, during rotation of the outer barrel, a major improvement over the rigid type for sampling of overburden materials. The sample or core is cut by

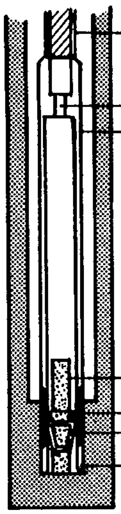


Figure 7-32. Single tube core barrel.

rotation of the diamond bit. The bit is in constant contact with the drilling fluid as it flushes out the borehole cuttings. The addition of bottom discharge bits and fluid control valves to the core barrel system minimizes the amount of drilling fluid and its contact with the sample which further decreases sample disturbance.

Additional modifications in the double tube core barrel such as the Sprague and Henwood Series M and Christensen Diamond Products Series D, are examples of continuing developments by manufacturers to improve sample quality and recovery.

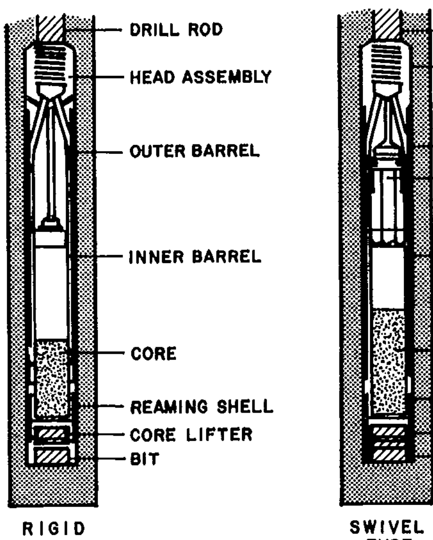


Figure 7-33. Double tube core barrels. (From "Field Evaluation of Advanced Methods of Subsurface Exploration for Transit Tunneling," U.S. Department of Transportation)

The third and most recent advancement in rotary core barrel design is the triple-tube core barrel, which adds another separate, non-rotating liner to the double tube core barrel. This liner, which retains the sample, consists of a clear plastic solid tube or a split, thin metal liner. Each type of liner has its distinct advantages and disadvantages; however, they are both capable of obtaining increased sample recovery in poor quality rock or semi-cemented soils, with the additional advantage of minimizing sample handling and disturbance during removal from the core barrel.

The rotary core barrels which are available range from one to ten inches in diameter, and the majority may be used with water, drilling mud, or air for recovering soil samples.

Of the three basic types of core barrels, the double tube core barrel is most frequently used in rock core sampling for geotechnical engineering applications. The triple tube core barrel is used in zones of highly variable hardness and consistency. The single tube, because of its sample recovery and disturbance problems, is rarely used.

Two advanced rotary core barrels from various manufacturers are discussed in more detail below. However, the selection of these core barrels is not intended to imply that other equipment would not be equally suitable to the given circumstances.

7.7.1.1 NWD4 Double Tube Core Barrel. The Christensen Diamond Products NWD4 swivel-type, double tube core barrel offers a non-rotating adjustable, chrome plated inner liner which is available in either solid or split tube versions (Figure 7-34).

There are several unusual but highly successful modifications in this barrel which include:

- Core barrel disassembly from the top or back of the tube prevents excessive wrench handling of the diamond bit and core lifter assembly.
- Adjustable inner liner annulus which controls the amount of fluid circulating through the core barrel. The amount of water which is required for drilling is a function of the quality of rock. This capability allows the core barrel to be adjusted to accommodate these changes, rather than replacing the entire core barrel.
- Rapid inner tube conversion from solid to split liner without special tools or replacement kits.

Depending on the quality of the rock being cored, the NWD4 may be alternately used in the solid or split inner liner modes. The solid liner is used primarily in very sound and competent portions of the rock while the split liner is used in the weaker and more weathered portions.

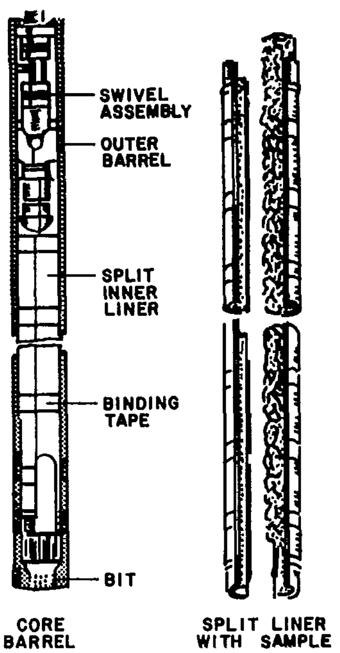


Figure 7-34. Christensen NWD4 split inner liner core barrel. (From "Field Evaluation of Advanced Methods of Subsurface Exploration for Transit Tunneling," U.S. Department of Transportation)

The design of the split inner liner allows expansion of the two liner halves during the core recovery process. This feature allows swelling clays or highly fractured material, which could normally block a conventional solid liner, to move up into the Chrome-plated liner, reducing blockage and grinding of the core and improving recovery in the lower quality rock. An additional and major advantage of the split liner is observed during subsequent surface handling of the recovered core. The inner liner is easily removed from the core barrel and the filament tape which binds the liner halves together is cut and the two sections separated, exposing the recovered core in its near *in situ* state (Figure 7-35). This design feature of the split liner eliminates the necessity of "banging out" with a hammer as is frequently done with the conventional solid liner. Such core removal could severely disturb and alter the quality of the recovered core, leading to erroneous conclusions about the overall rock mass.

The split inner-liner is not exclusive to Christensen core barrels and is used in a variety of types and sizes of double and triple tube core barrels. The capability of improving recovery in poor quality rock and the subsequent surface handling advantages, makes it a valuable equipment addition for the purpose of rock core evaluation.

7.7.1.2 NWM3 Triple Tube Core Barrel. The Acker Drill Company, Inc. NWM3, swivel-type, triple-tube core barrel is a modification of the Series M double-tube core barrel, that includes an additional inner solid clear plastic liner which retains the sample recovery (Figure 7-36).

The purpose of the third, non-rotating inner liner is to further improve sample recovery in soft or highly fractured rock and to provide a temporary storage container for the recovered rock core during transportation and storage.

The NWM3 incorporates an adjustable inner liner which can control the flow of water to the bit, an important design feature in variable formation conditions. The use of bottom discharge bits also minimizes the amount of drilling fluid in contact with the recovered sample, decreasing the erosive action in highly decomposed rock.

The NWM3 triple-tube core barrel is an important advancement in drilling technology that improves recovery in formations which are difficult to sample with conventional core barrels. A special hydraulic or pneumatic jack is required for inner tube (Figure 7-37) removal and subsequent sample extraction from the inner tube. Although the solid plastic sample liner tube has definite advantages during transportation and storage, it can impede, somewhat, field examination, photographing, and evaluation of the core immediately upon recovery.

7.7.2 Specialty Core Barrel Types

A variety of special core barrels have been developed for specific sampling problems and requirements. These core barrels may adapt conventional rotary core barrel design or utilize completely different techniques and equipment. Several of these specialty core barrels are briefly summarized below:

- Wire Line or Retractable Core Barrel
- Calyx or Shot Core Barrel
- Steel Tooth Cutter Barrel
- Percussion Core Barrel

7.7.2.1 Wireline Core Barrel. In conventional rock coring the entire drill stem and core barrel must be removed after each core run (usually 1.5 to .5 feet). This is a time-consuming operation on deep core holes, in addition to creating an inherent risk for collapse of the rock into the unsupported borehole. The Longyear Co. "Q" Series Wire Line system, is designed to recover rock core without removing the drill stem from the borehole after each core run.

When drilling is completed, a special latching mechanism is lowered through the drill rods at the end

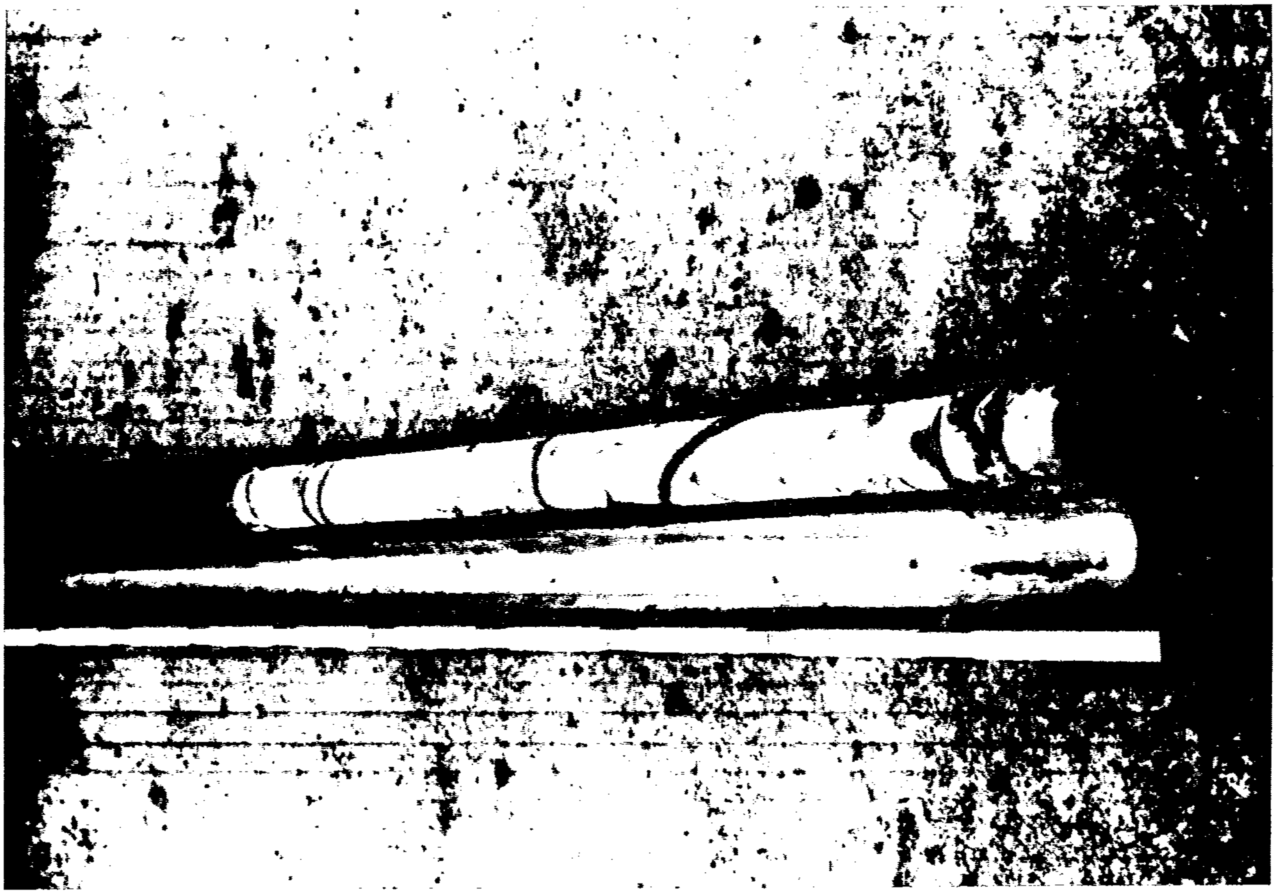


Figure 7-35. Full recovery of argillite in NWD3 split liner. (Haley & Aldrich, Inc.)

of a cable which attaches to the inner barrel of the sampler. The inner barrel, containing the rock core, is rapidly brought to the surface, leaving the outer core barrel and drill rods still in position within the borehole. The wireline can also be adapted for horizontal drilling and triple tube applications.

7.7.2.2 Calyx or Shot Core Barrel. This method of obtaining very large diameter samples of competent rock core, derives its name from the use of chilled, hard steel shot used as the cutting medium. Single tube, heavy walled, soft steel cutter barrels of varying lengths and diameters are manufactured by Ingersoll-Rand Company, especially for this purpose. The steel shot is fed into the annular space between the core and corebarrel and grind their way to the bottom of the hole where they are picked up in a special kerf cut into the bottom of the barrel. The steel shot, which is added as the drilling progresses, wears away the rock beneath the rotating barrel. A special "Calyx" at the top of the barrel causes a reduction in the rate of the returning wash water and serves to collect the borehole cuttings and worn-out shot.

The core is removed from the borehole by special large diameter core lifters or by grouting the core inside the barrel with gravel. Considerable driller expertise is required with this method. The diameter of the core that can be recovered is limited only by the capability of the equipment to turn the core barrel and subsequently recover it.

7.7.2.3 Steel Tooth Cutter Barrel. Single tube core barrels equipped with metal teeth are used for obtaining large-diameter cores in soft or seamy rock. However, any type of core barrel may be equipped with steel cutter teeth if the situation does not require the use of diamond bits. The Denison and Pitcher Samplers discussed in Section 7.6.4 are generally equipped with this type of cutter bit. The steel cutter teeth may also be equipped with hard metal alloy inserts such as tungsten-carbide, to improve drilling rates. The metal inserts may be replaced in the bit very readily, renewing a dull or damaged bit for additional drilling.

The steel tooth cutter barrels are operated in the same manner as conventional rotary core barrels except that they are rotated at much slower speeds.

As the costs associated with these types of bits are

Figure 7-36. Acker triple tube core barrel with solid clear plastic liner. (From "Field Evaluation of Advanced Methods of Subsurface Exploration for Transit Tunneling," U.S. Department of Transportation)

considerably less than costs for bits equipped with diamonds, they are used in areas of difficult drilling where bit loss may be appreciable. These areas would include the drilling of structural steel in concrete or

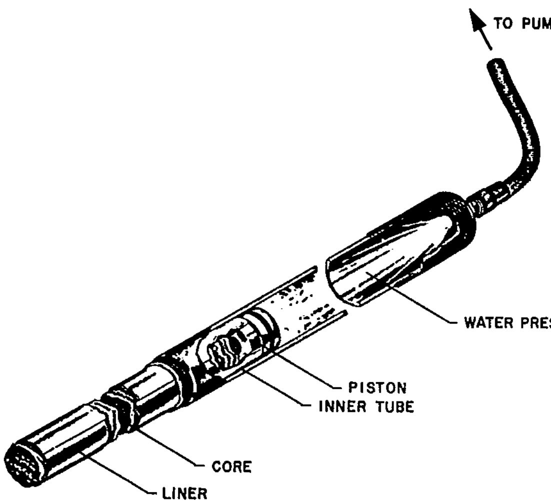


Figure 7-37. Hydraulic removal of split inner liner from core barrel.

dry hole drilling which would burn up and destroy diamond bits very rapidly.

7.7.2.4 Percussion Core Barrel. The percussion or cable tool core barrel is more widely used in the soil and water well industry and not commonly associated with foundation investigations.

This core barrel consists of an outer barrel with a hardened steel bit and an inner barrel equipped with a pressure release system and core retainer. The inner barrel remains in contact with the rock and slides down over the core as the surrounding material is cut away by raising and dropping the outer barrel.

Cores can be obtained in materials ranging from partially cemented soils to medium-hard rock. However, some disturbance and breakage of the core usually occurs during the dynamic sampling process.

7.7.3 Integral Rock Core Sampling

The determination of the various bedrock discontinuities which effect the strength and stability of a rock mass, are of critical importance in the design and construction of underground openings in rock. The structural integrity of the rock mass is affected by the presence and orientation of such features as bedding, jointing and faulting, and also by the spacing, continuity, planarity and infilling of these discontinuities.

The primary method for evaluating the geotechnical parameters relies upon measurements and observations of exposed bedrock in the area of the proposed construction. These outcrops may or may not reflect the actual *in situ* conditions of the bedrock unit at depth. In urban areas, bedrock outcrops may be very limited and far removed from the actual area of construction. Typical subsurface exploration programs, which are initiated to obtain information about the structural defects of the bedrock, may lack the detail required for a reasonable assessment of these characteristics. Grinding of the rock core, poor recovery and washing out of the gouge and infillings during the drilling operation create erroneous conclusions regarding the quality of the *in situ* rock mass. In addition, conventional exploration methods are not capable of determining the orientation of the overall bedrock structure or the discontinuities.

Several subsurface exploration methods recently developed are capable of obtaining the structural orientation of the planar features of the *in situ* bedrock. These rock core orienting methods are summarized in Section 7.7.4. However, these techniques are combined with conventional diamond core drilling and are not capable of recovering totally intact, undisturbed, continuous samples of the bedrock.

Techniques which combine total intact core recovery, and structural orientation are discussed below:

7.7.3.1 LNEC Integral Sampling Method (ISM). A method that combines structural orientation with total intact rock sample recovery is the "Integral Sampling Method" (ISM), developed by Manuel Rocha in 1970 during his tenure as Director of the National Laboratory of Civil Engineering (LNEC) in Lisbon, Portugal.

This relatively new and sophisticated technique of injecting grout into a small diameter pilot hole, orienting the grout rods to a surface feature and then overcoring the solidified mass with a larger diameter core barrel is used for detailed structural and engineering analysis of the *in situ* rock mass.

Special patented ISM orienting equipment and technical assistance may be obtained from Sprague & Henwood, Inc., Scranton, Pennsylvania. A drilling crew thoroughly familiar with all phases of the procedure and equipment is essential. There appear to be no rules available to help a newcomer cope with initial difficulties due to unfamiliarity with the equipment.

This expensive but very valuable subsurface exploration technique will provide detailed information about the *in situ* rock mass properties which cannot be obtained by other conventional methods.

A conventional cased borehole with the required inclination is drilled to the depth where structural information on the bedrock unit is desired. The ISM core can then be recovered in NX (76 mm; 3 in.) and HX (98 mm; 3.875 in.) sizes, depending on the anticipated quality of the bedrock. The recovery of the ISM core sample is achieved in three basic operational phases.

Phase I

A stabilizing guide assembly, having an outside diameter slightly less than the diametrer of the borehole, is installed at the bottom of the hole. A small diameter pilot hole, approximately 32 mm (1.25 in.) diameter, is drilled into the intact rock below the stabilizing guide assembly with an RWT size coring or non-coring diamond bit. The stabilizing assembly maintains the pilot hole in coaxial alignment with the primary borehole (Figure 7-38). When the pilot hole is completed, the pilot drill and stabilizing assembly is removed from the borehole.

Phase II

A second stabilizing guide assembly, which incorporates a detachable grout/reinforcing/orienting, or "GRO" tube. This perforated, steel reinforcing tube, is lowered into the borehole so that the GRO tube extends into the pre-drilled RWT pilot hole. The GRO tube is connected to the surface with a string of interlocking, aligned, hollow orienting rods. A special orienting device is attached to the orienting rods and visually aligned with a permanent landmark whose directional bearing from the borehole may be determined at a later date (Figure 7-39).

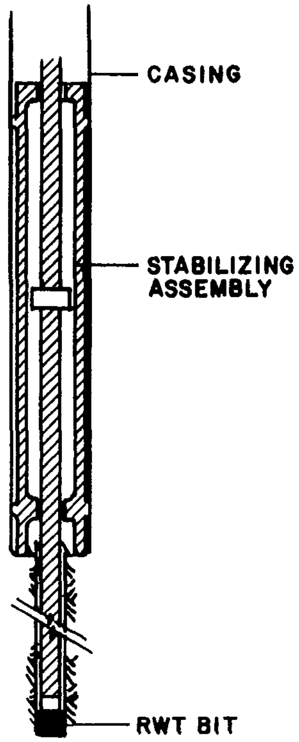


Figure 7-38. ISM Phase I, RWT pilot hole drilling and stabilizing assembly.

A predetermined amount of cement or chemical grout is then injected through the orienting rods and GRO tube into the voids and fractures around the pilot hole. After the grout is allowed to solidify, the GRO assembly is sheared off above the grout rod and recovered from the borehole (Figure 7-40).

Phase III

The solidified grout bonds the fractured rock to the oriented GRO reinforcing rod and the entire installation is overcored with a conventional core barrel, usually of the same diameter as the basic borehole (Figure 7-41). A variety of core barrels are suitable for the overcoring phase, although

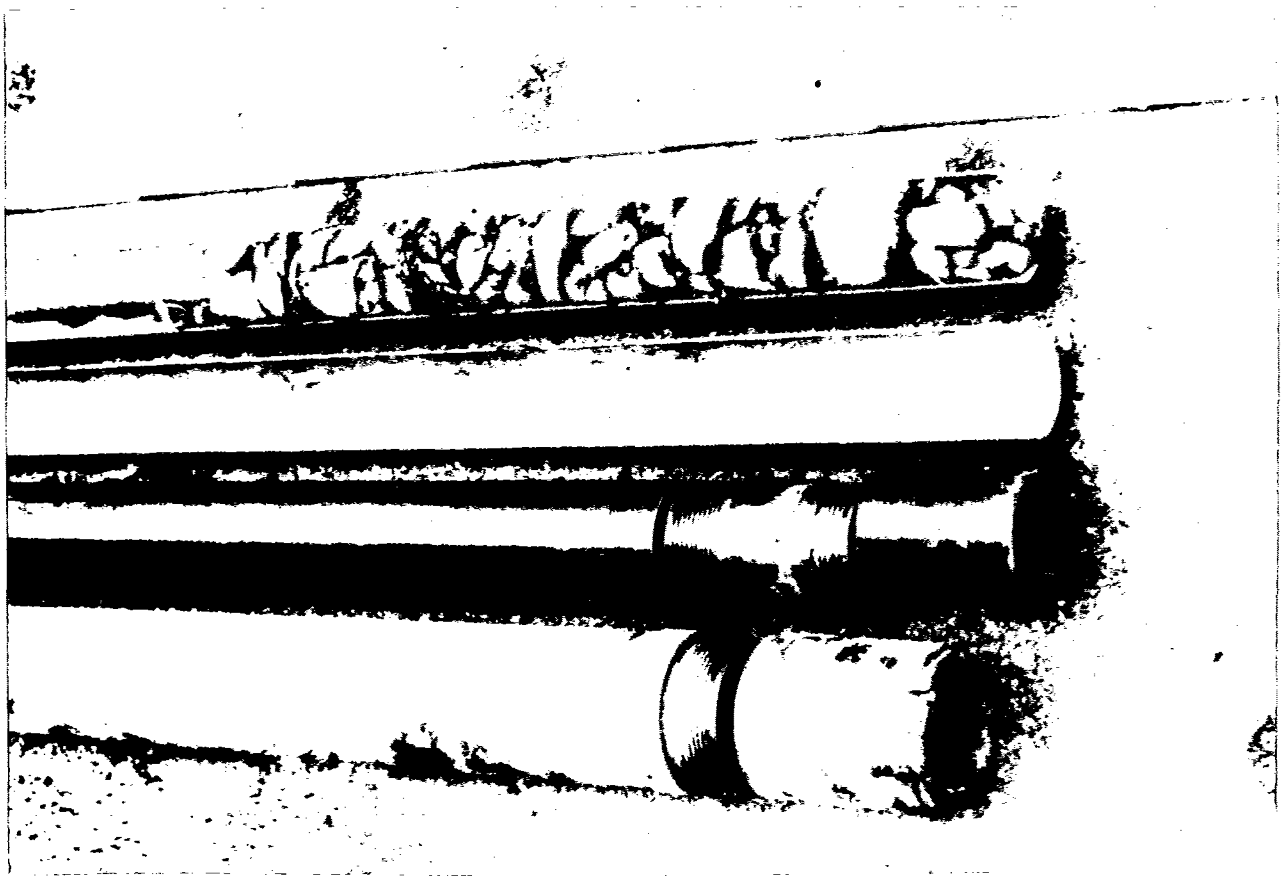


Figure 7-39. ISM orienting sight assembly. ISRM sampling device, with test-load of previously cored argillite of high joint frequency; note the centered grout tube appearing at the lower end of the core barrel. (A.W. Hatheway)

one equipped with a split inner liner is preferred. The core barrel is retrieved in the normal manner and the intact integral rock sample is evaluated for structural defects.

7.7.3.2 CSIR Integral Sampling Method. In 1975, the South African Council for Scientific and Industrial Research (CSIR) developed another prototype method for obtaining integral rock core samples (Orr, 1975). This integral sampling technique follows the same basic procedures developed by LNEC. However, there are several major differences in the equipment and specific methods relating to the insertion of the grout medium and the use of plastic (PVC) grout/reinforcing/orienting rods.

A reinforcing rod manufactured from PVC is used in order to preclude damage to the drill bit in case any eccentricity occurs during the overcoring process. A resin-filled cartridge is attached to the reinforcing rod, eliminating the need for surface grout reservoirs and pumps (Figure 7-42).

The resin cartridge is manufactured from commercially available PVC tubing and a wooden plunger. The system is manually operated at ground surface and displaces the resin through randomly drilled holes in the PVC grout rod. A saw cut groove along the long axis of the PVC is used for orienting the core in the same manner as the LNEC method.

7.7.3.3 ISM Application Considerations. As with any newly developed methodology, continued application and familiarization with the equipment and technical procedures will lead to improvements and modifications in the technique. The present high costs of ISM drilling are due primarily to lack of expertise with the procedure.

Several minor deficiencies exist in the application of the ISM technique at its current state-of-the-art, which should be evaluated prior to selecting this system for a particular project. These include:

- Required training of the drilling crew unfamiliar with the procedure by qualified ISM technicians under actual field conditions.

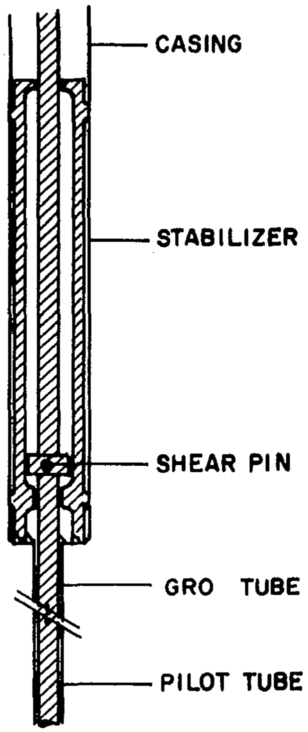


Figure 7-40. ISM Phase II, "GRO" tube and stabilizing assembly.

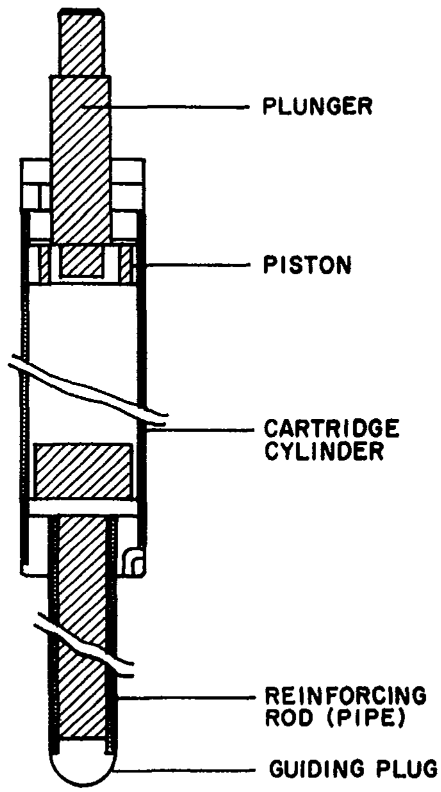


Figure 7-42. CSIR resin cartridge and reinforcing pipe assembly.

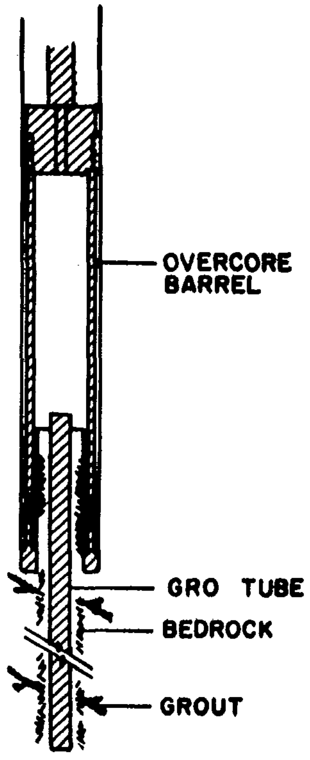


Figure 7-41. ISM Phase III, overcoring grout reinforced "GRO" tube.

- A trial and error phase by the drilling crew, even after field training.
- Limitations of the procedure in highly-fractured, open-jointed rock. Highly-fractured rock tends to collapse into the pilot borehole preventing re-entry of the grout rods, or the grout tends to flow beyond the limits of the overcoring phase.
- Modifications of the grout tube would provide total grout distribution into the desired areas.
- The selection of the proper grout for the specific situation requires considerable field experience.

7.7.4 Rock Structure Orientation Methods

The determination of the true attitudes of planar structural discontinuities of rock encountered during subsurface explorations may be accomplished in either of two ways; by measuring the azimuth and dip of the discontinuities recorded on the physical core recovered; or by determining the orientation of the structural features from their presence on the borehole wall (Barr 1976).

If rock core or impressions of the wall of a borehole can be oriented with respect to a feature of known direction, whether a scribed line on the core, a geologic feature of fixed orientation or alignment of the recording device in a known attitude, it is possible to measure the true dip and strike directions of all geological features in the borehole. These determinations consist of simple measurements; however, mathematical or graphical stereo plots are required if the borehole varies from the vertical or horizontal position (Goodman, 1968).

Various methods, ranging from simple to complex, have been developed to establish a reference point of known orientation so that all structural aspects of the borehole may be related to it and their absolute orientations determined. The Integral Sampling Method discussed in Section 7.7.3 is one of the more complex methods of achieving structural orientation of the *in situ* rock and total rock core recovery. The majority of the methods combine conventional rotary rock drilling equipment with specialized core barrels which mark the core so that it can be subsequently oriented using geological interpretive methods.

Structural orientation methods which are applied from within the completed borehole, and which include both optical and geophysical techniques, are discussed in Section 6, Engineering Geophysics, of this Manual.

The variety of rock core orientation methods and equipment are summarized below and include the following:

- Physical Core Alignment Methods

  Physical Methods
  Paint and Acid Markers
  Craelius Core Orientator

- Orienting Core Barrels

  BHP Core Barrel
  Christensen-Hugel Core Barrel

7.7.4.1 Physical Core Alignment Methods. These alignment methods all require a constant and known structural azimuth which may be used for determining the orientation relationship of other discontinuities to the known azimuth. Complex rock structure with varying features are not suitable for physical core alignment methods. In addition, these methods all retain inherent limitations in their procedure or equipment. Their major asset is the relative simplicity and low cost to conduct.

- *Physical Alignment:* the most simple and effective method, if core recovery is excellent, is to match the various core stems which are recovered so that they are in their original borehole alignment. A reference line is then drawn along the entire core and its orientation is determined from the structural feature with a known azimuth. The remaining discontinuities are then related to the reference line.
- *Paint and Acid Markers:* a relatively simple method, which can only be used in inclined boreholes, consists of breaking a container of paint against the rock surface at the bottom of the borehole (Rosengren, 1970). The paint runs down the core stub which gives a reasonably accurate indication of the underside of the core stem. Once the core is removed, it may be physically aligned and a reference line established.

  A tube of hydrofluoric acid may also be broken against the core stub at the base of the borehole, which etches the rock surface at the bottom of the core (Figure 7-43).
- *Craelius Core Orientator:* This relatively simple mechanical device used in inclined boreholes, "floats" within a conventional core barrel and consists of six, self-locking prongs which adjust to the profile of the core stub at the base of the borehole. The profiling prongs are locked into position when pressure is applied to a spring loaded plunger mounted behind the prongs (Figure 7-44). As the rock is being cored, the orientator slides back into the core barrel. When extracted, the core is placed in a cradle and rotated until the face conforms with the profiling pins. The remaining core segments are then aligned by mating opposing discontinuity surfaces (Bridges 1971).

7.7.4.2 Orienting Core Barrels. Specialized core barrels have been recently developed which scribe a reference mark on the core as it is drilled. Special recording devices within the core barrel relate known azimuth orientations to the reference mark so that when the core is subsequently removed from the core

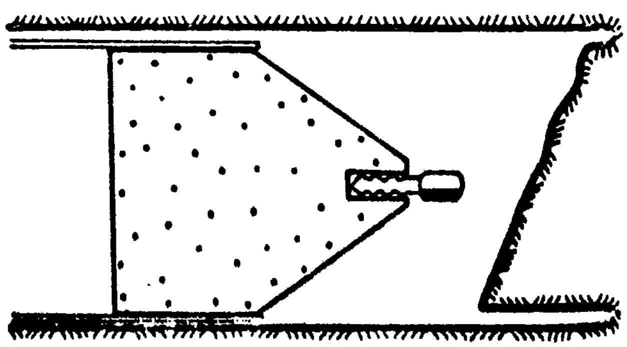


Figure 7-43. Paintor acid markers to orientate rock core (After Rosengren).

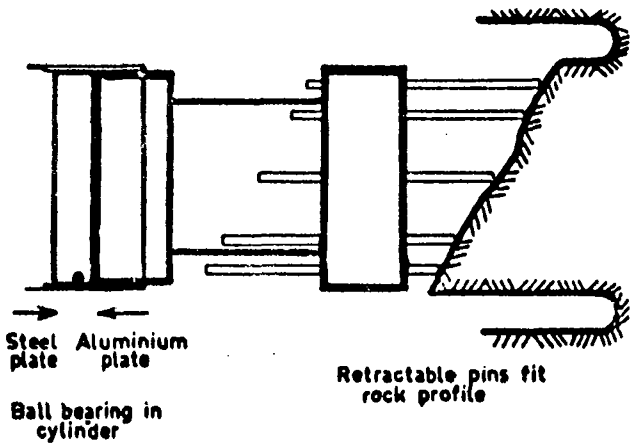


Figure 7-44. Craelius Core Orientator (After Rosengren).

barrel, it can be oriented to the exact position it occupied *in situ.*

These specialized core barrels are relatively expensive and require highly trained personnel to operate them and to interpret the results. In addition, several limitations are inherent with these devices:

- Excellent recovery is required for quality interpretation.
- Will not function in strong magnetic environments.
- Steeply inclined boreholes will interefere with the compass unit.

The individual orienting core barrels now available are as follow:

- *BHP Orienting Core Barrel:* The BHP core barrel, developed by the Broken Hill Proprietary Company, of Australia, utilizes a compass and chart recording system which aligns itself with a scribing diamond (Young, 1965) (Moelle, 1970). As the core passes the drill bit into the inner liner, a reference line of known orientation is scribed on the rock core.
- *Christensen-Hugel Orienting Core Barrel:* The C-H core barrel, developed and patented by the Christensen Diamond Products Co., operates in a similar manner to that of the BHP barrel. Incorporated within the core barrel is an Eastman Multishot directional survey instrument which photographically records the compass bearing and plunge of the borehole. In addition, it records the orientation of reference grooves which are cut into the core as it enters the barrel.

  The developed film is evaluated in a special read-out unit which synchronizes the correct photograph of the compass with depth. This information will supply the true bearing of the main scribe, the true inclination of the borehole and the true dip direction of the borehole. This information is then preset on a special readout unit referred to as a *goniometer* and the various sections of rock core can be evaluated for structure and discontinuities and their orientation determined.

7.8 EXPLORATION DIFFICULTIES

Specific advantages and disadvantages of the various subsurface exploration techniques and equipment have been discussed within the preceeding sections. However, limitations and difficulties may be encountered during the exploration program which are common to all exploration techniques. These are usually a result of site specific geological conditions and not necessarily a function of the equipment or method being used.

7.8.1 Sample Recovery

Generally, sample recovery less than .3 M (1 ft.) is considered inadequate for representative sampling. However, this criteria may be waived for the specific situation (ie: in thick, uniform deposits, recovery considerably less than .3 M (1 ft.) may be acceptable).

The use of drilling mud (Refer to Section 7.5.2.2) with its greater specific gravity and viscosity will assist in sample retention. Various sampling devices equipped with check and pressure release valves, sample retaining springs, baskets and lifters should be used or determined to be operational. Occasionally, drillers will modify equipment to meet their specific drilling technique, which may have major effects on sample recovery.

Selective overdriving of the sampling device will jam excess material into the sampler tending to retain the sample within the device during recovery. However, this procedure can only be used where disturbed samples are acceptable.

7.8.2 Sample Disturbance

Using existing soil sampling techniques, there is no way to obtain a truly "undisturbed" sample. Block sampling, discussed in Section 7.6.5, continues to be the most reliable method for minimizing sample disturbance and is used for comparison testing between other sampling techniques (Milovic, 1971; La Rochelle, 1971). However, obtaining block samples of the desired stratum may present major logistic problems, especially in urban areas.

The Swedish Foil Sampler, discussed in Section 7.6.3.7 is another attempt to modify equipment to further minimize disturbance.

M. Juul Hvorslev, in his major treatise, "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes," describes in detail the majority of exploration equipment and procedures, including sample disturbance. Investigators (Skempton, 1963; Rowe, 1971; Bozozuk, 1970; Ladd, 1974) have evaluated a variety of clay samples under controlled laboratory testing to determine the extent of disturbance on the various geotechnical parameters. Basically, the larger the diameter of the sample, the less the sample disturbance is minimized. The American Society for Testing and Materials (ASTM), STP 483 publication, "Sampling of Soil and Rock" (1970) presents several technical papers by various authors regarding sampling and associated disturbance. In addition, the Proceedings of Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, contains several articles by various investigators regarding sample disturbance (ICSMFE, 1977).

In the case of thin-wall tube sampling, disturbance was also found to vary throughout the length of the tube. The material located near the center of the tube (assuming .6 m; 2 ft. recovery) was found to be disturbed less than material at either end (Hvorslev, 1949).

The selection of the correct sampling tool, drilling technique, and borehole stabilization method will minimize sample disturbance, but additional investigations are needed to further evaluate sample disturbance and its effects.

The incorrect preservation and shipment of samples may further disturb the specimens and these procedures are discussed in detail in Section 7.9.

7.8.3 Obstructions

The termination of an exploration above the required design depth due to excessively dense materials, obstructions or "refusals" may occur during any investigation. When this occurs, it usually implies that the correct exploration method was not selected for the anticipated subsurface conditions.

Specialized tools and equipment are available to increase the capacity of conventional drilling equipment. This includes the use of diamond bit equipped casing which is drilled into the formation, rather than driven. A variety of drill bits are capable of drilling obstructions. These bits include the Servco Model 58 WCB, Underreamer (Figure 7-45) and the Christensen Diamond Products, Casing Advancer.

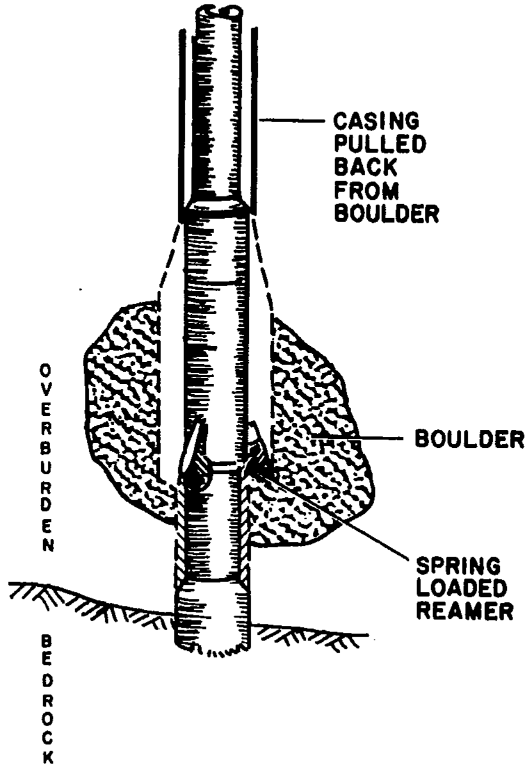


Figure 7-45. Servco model 58WCB underreamer.

7.8.4 Specific Geologic Problem Conditions

The preceeding sections have described the various methods, equipment and limitations of obtaining representative and undistributed samples for engineering analysis.

The American Society of Civil Engineers (ASCE) in its Manual No. 56, "Subsurface Investigation for Design and Construction of Foundations of Buildings," summarizes a variety of geologic problems affecting geotechnical exploration and design. Special consideration and care must be taken when selecting the proper sampling equipment, obtaining the sample and evaluating the performance of these materials (ASCE, 1972). A list of these foundation problems is summarized below:

- Organic Soils
- Normally Consolidated Clays
- Metastable Soils (loess, alluvial deposits & mud flows)
- Caliche
- Expansive Soils or Rocks
- Loose, Granular Soils
- Sensitive Clays
- Noxious or Explosive Gases
- Slope Movements
- Kettle Holes

- Meander Loops & Cutoffs
- Artificial Fill
- Karst (Sinkhole) Regions
- Weathered Shale Rocks
- Abandoned Mined Areas
- Frozen Soils

Other geologically-oriented foundation problems may be expected in different physiographic regions.

7.8.5 Groundwater Conditions

In many instances, one of the major concerns in a site investigation may be the determination of the ground water conditions and general hydrogeologic regime of the site area. The location of the groundwater table, perched or artesian conditions, surface runoff and recharge are a few of the important items that must be determined during the exploration program.

There is a tendency by the drillers who are conducting the work to obtain a borehole water reading during or immediately upon completion of the exploration; this water level is assumed to be the "groundwater table." As the effects of the drilling operation, especially if drilling fluids were used, generally have not dissipated, these short term water readings should be considered suspect. A reading made 24 hours after completion of the borehole is much preferred but in some instances, depending on the permeability of the soils, may still not be sufficient. The installation of permanent or temporary observation wells in the completed boreholes is generally an inexpensive safeguard against erroneous assumptions regarding the presence and behavior of the groundwater conditions. Groundwater and observation wells are discussed in more detail in Section 8.0.

7.8.6 Borehole Sealing

Although unsealed, completed boreholes may be satisfactory at some locations, especially in rock, they may be the source of difficulties at a later date or during construction. A borehole which is not properly backfilled or sealed may provide access for unanticipated amounts of water into areas which were considered "impermeable." In addition, breached aquicludes and artesian zones may have adverse effects on the local aquifer conditions, unless properly sealed. However, unsealed boreholes may occasionally be used to advantage, for draining local perched or trapped water to the static groundwater table.

The best method of borehole sealing is by pumping a cement grout from the bottom of the borehole to the top and removing the supporting borehole casing after the hole has been completely filled with grout. Chemical setting retardants may be added to the grout to provide sufficient time for casing withdrawal.

7.9 SAMPLE PRESERVATION AND SHIPMENT

Samples of soil and rock are obtained for classification and subsequent testing and analysis to determine their various engineering properties. Rock and soil samples (core, jar, can, tube and bag) represent essential physical information concerning the subject site, generally obtained under costly circumstances. Samples must be preserved, stored, and shipped under conditions that will minimize chances of disturbance or loss.

All soil samples and rock cores must be clearly, accurately, and permanently labeled to show all pertinent information which may be necessary in identifying the sample or core and in determining the character of the subsurface condition.

The preserving, protecting and transporting of samples may be accomplished using the methods set forth in this section, but any method which satisfactorily protects a sample intended for laboratory testing from shock, detrimental temperature changes (such as freezing), and moisture loss may be used.

- All samples should be collected from the borehole sampling sites on a daily basis and transported to the field project office or a suitable alternate location.
- Rock core and thin-wall tube soil samples should never be transported away from the field site in other than specially constructed wood, metal, plastic or fiberglass shipping containers and should be packed in excelsior or equal material in order to protect them from vibration. These containers must be secured (screws, banding, clasps, etc.) whenever they are to be transported.
- Samples should not be left unattended in vehicles and any sample which is permitted to freeze, even partially, should be replaced.
- Samples intended for laboratory testing should not be held at the site in excess of one week.
- All containers should be identified as to borehole, depth interval, box number of total sequence, and project number. These markings will be placed on the exterior and interior of lids, on both ends (to facilitate identification in storage).
- Any special actions taken on the samples should be identified on the interior of box lids, e.g., sampling thin-sections, x-ray diffraction,

paleontological analysis, paleomagnetic measurements and radiometric age determination.

- It may be advantageous, for future record purposes, to photograph the samples, particularly rock core with color film prior to packing and shipment. Section 7.10 summarizes the basic procedures for photographic recording of samples.

7.9.1 Jar Samples

Representative specimens of each sample should be preserved. The containers for preserving drive samples should be large mouth, round, screwed top, airtight, plastic or clear glass jars of sufficient size to store at least a 100 mm (4 in.) long section of full diameter sample. Project requirements may dictate total sample preservation and larger jars. The specimen should be placed in the jar as soon as taken in order to preserve the original moisture in the material. The mouth of the jar should be cleaned of all dirt and grit in order to obtain an airtight seal with the screwed top.

- Remove the sample carefully from the sampler and place it in the jar with a minimum amount of disturbance. Do not attempt to overfill or force excess material into the jar.
- Tightly cap the jar and mark the lid with the following information:

  Project Number
  Boring Number
  Sample Number
  Sample Depth Range
  Blow Counts

  Additional information may be required or desirable, which is usually added by the contractor or his supervisor at a later date in the form of a label which is affixed to the jar.
- If long term preservation or additional testing is required, seal the top of the jars with plastic electrical tape and/or a non-shrinking wax.
- Jars should be packed sequentially in partitioned, heavy-duty, cardboard boxes and be protected from freezing or excessive heat.
- Completed boxes are generally transported to the drilling firm for additional classification and labeling.
- Handle and transport the sample boxes with care to avoid breakage.
- If the samples are to be shipped by commercial or public carriers, the addition of shredded packing material (paper or plastic) around the jars will be required.
- Label the box exterior, stamp with FRAGILE and THIS END UP; insure the shipment.

7.9.2 Thin-Wall Tubes

Thin-wall tube sampling is utilized to obtain undisturbed cohesive and granular soils for laboratory testing.

7.9.2.1 Cohesive Samples

- Remove the thin-wall tube from the head assembly, taking care not to disturb the sample. Remove any disturbed "washings" from the top of the tube and measure the amount of sample recovered, in inches.
- Seal the top of the sample by pouring in slowly 25–50 mm (1–2 in.) of melted microcrystalline wax in thin layers. Fill the remaining void space in the tube with packing material to prevent slippage of the sample. Special expandable O-ring packer seals may be used in lieu of wax.
- Cap the end, tape securely, and dip the end into the melted wax several times.
- Repeat steps two and three, above, for the opposite end of the tube. It may be necessary to scrape out approximately 25 mm (1 inch) of undisturbed material to allow space for the wax to set.
- Mark the top and bottom of the tube, the project number, boring number, sample number, depth range, recovery and method employed in advancing the tube.
- Samples should be maintained in as near vertical position as possible and protected against excesses in heat or cold.
- Transport the samples by private vehicle whenever possible, and maintain them in a vertical position, either by tying to the seat of the vehicle or in a rack specially constructed for transportation of undisturbed tubes.

7.9.2.2 Granular Samples

- Remove the sampler from the borehole with extreme care, keeping it in a vertical position at all times.
- Before removing the sampler from the drill rod string, while still held in a vertical position by the driller, carefully insert an expandable O-ring packer seal in the bottom of the tube. This may require removing a small amount of soil to make room for the packer seal. Tighten the seal against the tube walls (Figure 7-46).

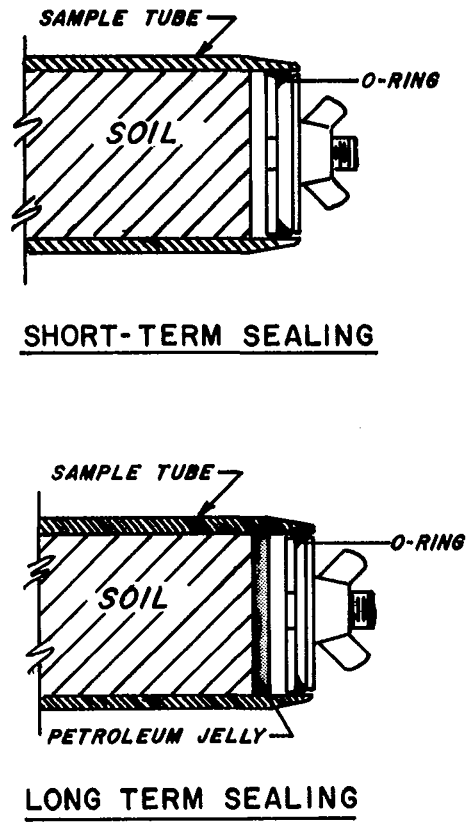


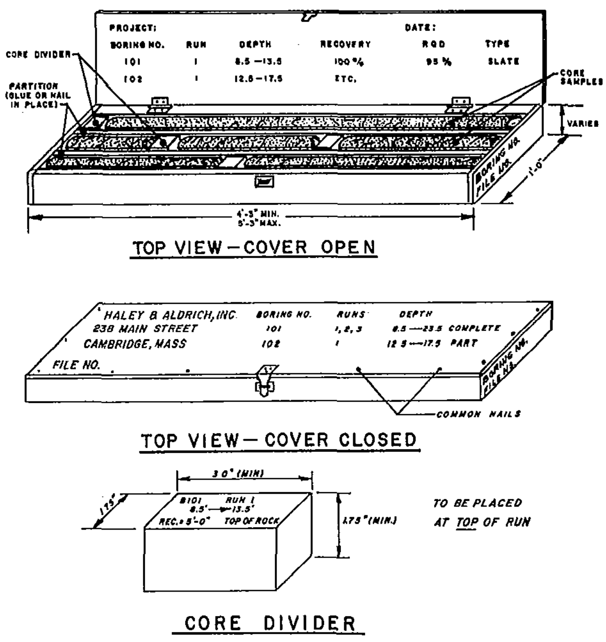
Figure 7-46. "O"-Ring expandable packer seal.

- Place an end cap on the bottom of the tube, securely wrap with electrical tape, and dip in melted wax.
- Remove the tube carefully from the drill rod string and head assembly, maintaining the tube in a vertical position at all times.
- Place tube in the tube rack and clean out any disturbed material.
- Place a thin plastic bag (firmly but gently) into the tube, fill with damp towels or newspapers, so as to occupy all remaining void space in the tube.
- Place end cap on the top of tube and secure firmly with electrical tape.
- Mark the top and bottom of the tube, the project number, boring number, sample number, and depth range recovery. Indicate that the tube contains granular soils and mark FRAGILE.
- Sample should be maintained in vertical position during all phases of packaging and transportation and protected from excess heat or cold. Whenever possible, transportation of samples should be by private vehicle and held in specially constructed tube racks.

7.9.3 Rock Core

All rock core recovered during the drilling operation is preserved for photographing, analysis and selected laboratory testing of representative samples.

- Remove rock from the core barrel with care and place in the wooden core box. Reject any boxes that are unstable or in disrepair. Core boxes are to be substantially constructed of milled lumber and equipped with partitions, cover hinges, and cover hook or latch (Figure 7-47).
- Arrange the core carefully, insuring that the top and bottom sections are in the correct position beginning at the upper left hand corner of the core box.
- Each core run should be enclosed securely at the top and bottom of the run by a wooden divider block nailed in place.
- Describe the core according to the criteria established in Appendix E, Classification of Rock. Record this information on the inside of the core box cover, including boring number, core run number, and depth range of the sample.
- Mark the bottom depth of each core run on each



NOTES

1. Core dividers to be nailed in place as required.
2. Core boxes shall be substantially constructed of dressed lumber and equipped with all necessary partitions, dividers and hardware.

Figure 7-47. Core Sample Preservation. (Haley & Aldrich, Inc.)

divider. Core dividers are also marked Top of Bedrock and Bottom of Exploration (BOE). If rock from more than one boring is placed in a single core box, it should be divided by two core dividers, clearly indicating rock from the various test borings.

- Select representative samples for additional testing if required, and package according to the guidelines summarized in Section 7.9.3.1.
- If Rock Quality Designation (RQD) is to be calculated in the laboratory after shipment, all pieces of core 100 mm (4 in.) or more in length should be marked with a continuous line along the full length of core so that any breakage during shipment can be discerned.
- Replace any void spaces left by the sample withdrawal; mark them with a notation to the effect that the sample was removed for testing.
- Securely fasten the cover to the box.
- Mark on the outside of the lid the address to which the box is to be delivered, the project title, project number and boring number. In addition, mark the project number and boring number on the ends of the box.
- It is most desirable to transport core boxes by private vehicle in order to minimize breakage of the core during transportation. If the boxes must be shipped by commercial carrier, mark the boxes FRAGILE and insure.

7.9.3.1 Rock Core Test Specimens. Representative samples of core may be selected during the exploration program for detailed laboratory testing or long term preservation. These test specimens require special packaging techniques. Refer to Figure 7-48 for details on packing test specimens.

- Select representative samples of the core approximately 150 mm (6 in.) in length (or as specified by the Project Manager) from various runs. Substitute a labeled wooden identification block to make up the same length.
- Mark on the test specimen, with an indelible pen, the boring number, run number, depth of specimen on top and bottom.
- Wrap sample in newspaper, plastic wrap or aluminum foil and pack tightly in a standard core box used to ship test specimens. In the case of particularly fragile specimens or if long term preservation is required, the specimen should be wrapped and then placed in a cardboard cylinder, and end-dipped in melted microcrystalline wax.
- Record the boring number, run number and depth range on the inside of the shipment box cover.

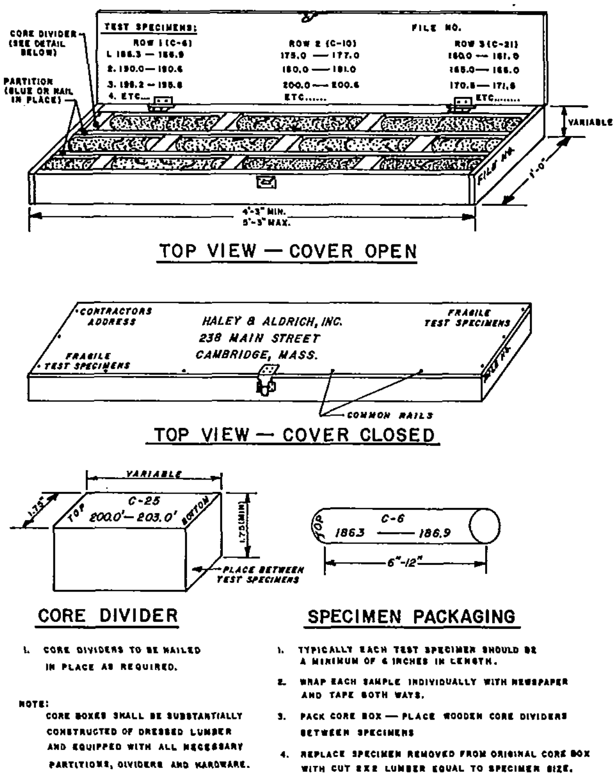


Figure 7-48. Core Sample Test Specimen Preservation. (Haley & Aldrich, Inc.)

- Securely fasten the cover to the box.
- Mark the outside of the box according to conventional rock core sampling criteria.

7.9.4 Bulk Samples

Bulk samples are obtained when larger volumes of material are required for laboratory testing. These may be obtained from test pits and trenches, during test boring programs and from borrow material surveys and permeability studies. Block sample preservation is discussed in Section 7.6.5.

- Obtain a representative sample, in the natural state, ranging in weight from 14 to 23 kg (30 to 50 lb.). If the sample is to be obtained from a spoil pile, take random samples from various locations to obtain a representative specimen.
- Place the sample inside the bag and mark two labels with the project name, file number, location, depth, date and brief description of the material. Place one label inside the bag and wire-tie another to the exterior, at the bag neck closure.
- Protect the sample from freezing.

7.9.5 Environmental Test Samples

Representative samples may be required for chemical and physical analysis associated with environmental studies.

- Samples are obtained using the technique and equipment specified by the project geologist or geotechnical engineer. The U.S. Environmental Protection Agency has established guidelines and criteria for sampling and testing for environmental regulatory purposes.
- Obtain approximately 1 kg (2 pounds) of representative sample, at field moisture; place in jar or plastic bag. It is usually not necessary to preserve samples in sterilized containers unless bacteriological testing or spectrophotometric analysis is anticipated.
- If samples can be delivered to a testing laboratory within 24 hours of sampling, they do not have to be frozen and it may be preferable to preserve them in a glass jar, suitably labeled.
- If samples cannot be delivered to a testing laboratory within 24 hours of sampling, they must be maintained in a frozen state by packing in dry ice. Plastic bags would be more preferable in this case.
- Label samples indicating project title, file number, date and time of sample, exploration number, depth range and brief description.

7.9.6 Non-Containerized Samples

Several sampling methods, which include single barrel rotary soil samplers and certain retractable piston samplers, do not possess as part of the sampling process, tubes or containers to return the sample. The entire sample must be removed from the sampler and preserved in the field. The physical state and quality of the sample as it is ejected from the sampler, will dictate the type of preservation required. The variety of methods discussed in the preceeding sections may be used separately or in combination to preserve, protect and transport the sample for additional laboratory testing.

7.10 PHOTOGRAPHIC RECORD

Rock core and certain types of drive samples are usually the only physical sample evidence of the subsurface profile that remain available for a given site. In order to maintain the integrity of this record, it is sometimes necessary to photograph the samples before parts are removed for testing purposes or otherwise disturbed. Photographs provide for the preservation of the sampling record in the event that vandalism, negligence, or natural calamity cause loss or destruction of the physical sample. It also may be desirable to photograph specific sampling techniques and equipment for future reference.

Although it is much more preferable to photograph samples under controlled conditions, including supplementary lighting and camera support devices, this is not usually the case under field conditions (Figure 7-49).

- A 35 mm camera with through-the-lens metering is preferred, especially for novice photographers, but any camera that obtains high quality photographs is acceptable.
- If at all possible, obtain the photographs at the same time of day, same azimuth direction, similar lighting conditions and with the same background. This will assist in more uniform and better quality photographs.
- Identification cards should be made out, indicating relative sample location and identification and be included with the sample or box for photographing.
- In the event that rock core boxes are being photographed, the information written on the inside cover will suffice.
- Place a rule or scale along the edge of the sample or box for size comparison.
- Lay back any protective wrappings on cohesive soil samples in the boxes. Wet all core and hard, cohesive soil with fresh water, using a fine bristle brush. Do not attempt to fully wet broken or crushed fragments.
- Align the sample box so that it appears full-framed in the camera view finder, with the long dimension of the box parallel to the long dimension of the camera format, and in a plane perpendicualr to the focal axis of the camera.
- Photograph the sample display. A minimum of two photographs, using color film, should be obtained of each sample display. One set may be retained by the project geologist or geotechnical engineer and the other may be transmitted to the client Agency, or other interested parties.
- No core or soil samples should be removed from the sample boxes until the project geologist or geotechnical engineer has received the appropriate views and is satisfied with the quality of photography and reproduction.
- Arrangements should be made to photograph samples at regular intervals during the execution of the exploration program.

Figure 7-49. Photographic setup of rock core boxes, undertaken to produce a permanent record, to provide contractor bidding information, and to safeguard against loss of core in transport or storage. (A.W. Hatheway)

7.11 SUPERVISION AND INSPECTION OF SUBSURFACE EXPLORATIONS

Exploratory test borings represent the primary source of subsurface information, relating to the geologic suitability of a specific site, from which engineering decisions are made relating to design requirements.

The personnel selected by the project geologist or geotechnical engineer for supervising and inspecting the exploration program are responsible for obtaining the best possible information which will provide the framework for the subsequent engineering analyses. These personnel should become thoroughly familiar with the project requirements, both short and long term. They should review all existing applicable information which relates to site geology and have a detailed familiarity and understanding of the contract specifications. They should be experienced in the appropriate test boring procedures, instrumentation installation and sampling requirements.

The person who actually records the data in the field will vary from organization to organization. Geologists, engineers or technicians may accompany the drilling crew and provide these services, or the drill crew foreman may be responsible for logging the borehole. It is recommended that personnel other than the drilling crew be responsible for logging and evaluating the subsurface conditions. This will allow the drilling crew to concentrate on the technical aspects of the work and the logger to concentrate on the engineering and geological aspects of the information as it is obtained. The term "logger" is used interchangeably with supervisor, inspector, and foreman, to denote the person responsible for obtaining and recording the field data.

7.11.1 Duties and Responsibilities of Logging Personnel

- Acquire reliable subsurface information of the type necessary to evaluate the geologic suitability of the site.
- Observe, describe, record and evaluate all sub-

surface information, exploration techniques, equipment and associated operations conducted as a part of the field investigation.

- Verify compliance with quality assurance provisions and contract specifications.
- Coordinate with all subcontractors.
- Examine all drilling equipment and sampling devices for defects and operational efficiency and determine that the necessary materials are readily available.
- Maintain a subsurface information summary plot for the site, so as to be aware of, and to take advantage of, previous findings and results of adjacent explorations. This should be modified and reevaluated on a daily basis as additional information becomes available.
- Maintain a Production Summary that tabulates each boring and item in the contract documents. The summary should include location, ground elevation depth of boring, tests and date at the start and finish of each boring. This summary will allow the logger to keep in continuous touch with progress and costs of the work and will be a valuable aid in making more efficient field decisions.
- Complete all logs, forms and daily reports using the established classification and testing criteria. It is important to record the maximum amount of information, even if it apepars trivial at the time.
- Insist on proper sample preservation, labeling, transportation and temporary storage.
- Select, package and transport special samples for additional testing and analysis.
- Obtain photographs, preferably in color, of the work, samples and site area.
- Prepare regular and ad-hoc verbal and written reports for the project geologist or geotechnical engineer concerning appropriate geological aspects and technical problems as they develop during the exploration program.
- Monitor groundwater levels for fluctuation over an extended period of time.
- Comply with all applicable articles of the Federal Occupational Safety and Health Act of 1970.
- Communicate with the project geologist or geotechnical engineer.

7.11.2 Logging

A significant part of the responsibility of logging personnel is concerned with the preparation of boring logs, daily reports, and the various data sheets associated with field testing. These are the only continuous field records for the project and they must be accurate and complete.

Samples of various completed forms are included in Appendix A of this Manual. The logger should make a copy for his own retention and, as soon as completed, the originals should be sent to the project geologist or geotechnical engineer. Special attention should be given to the remarks column, used to record any unusual drilling procedure or soil condition encountered. If there is some question as to whether a procedure or condition is in fact unusual, record it; in all probability the data will prove useful. The remarks section on the Daily Report forms can be used to report a variety of information such as: visitors to the site, details on rig movements, explanation of rig breakdown, time, comments on quality of the work or on the quality of the contractor's personnel, and changes in the work made by others. The logger should obtain a field copy of the Contractor's boring log at the completion of each boring and review for consistency and accuracy. These logs should be appended to the logger's information for submission to the project geologist or geotechnical engineer.

7.11.2.1 Equipment and Supplies. The following items are recommended for the logger's use at the site.

- Contract documents and boring location plans
- Correspondence pertinent to the exploration program
- Available subsurface information in the site area
- Field forms: boring logs, daily reports, summary sheets, etc.
- Soil or rock classification system applicable to the project
- Field book, standard waterproof surveyors style
- Pencils
- Pencil sharpener, pocket style
- Knife, pocket style
- Magnifying glass (10×)
- Clip board
- Plastic triangle (30–60 degree)
- Scales (engineer, architect or metric as required)
- Rule (2m; 6 ft., folding)
- 30 m (100 ft.) cloth tape with weighted end
- Carbon paper
- Envelopes
- Lumber crayons and waterproof marking pens
- Time piece

In addition, the following items, while not always essential, may be very useful:

- Site area topographic maps
- Pocket calculator
- Hand level
- Transit and/or level
- Dilute Hydrochloric acid
- Mineral hardness points
- Pocket penetrometer
- Pocket torvane
- Miniature sieve set

7.11.2.2 Format of Field Boring Log. The format of the "field" boring log or drilling report should be based upon organization policy and the information which is desired to be shown, as well as the manner in which it is to be presented. The format should be adequate for recording those items of information outlined within this section and any special information which may be required by organization policy or by unusual conditions.

It may be desirable to show by standard symbols information relative to non-core recovery, core recovery, and the taking of undisturbed samples for laboratory tests. Standard symbols may also be used to indicate the material which has been identified and logged.

Finally, there should be sufficient space for remarks, signatures, and a fully informative heading, which should be filled out as completely as practical at the site. However, the logger should not be burdened with the recording of repetitive or unnecessary information.

An example of a completed boring log is included, for reference, in Appendix A of this Manual. Other formats will be acceptable, provided that the form contains the essential information listed below.

7.11.2.3 Field Boring Log Data. The information which is recorded on the field boring log should include, but not necessarily be limited to, the following:

1. Description and classification of each rock and soil sample, and the depth to the top and bottom of each stratum.
2. The depth at which each is taken, the type of sample taken, its number, and any loss of samples taken.
3. The depths at which field tests are made and the results of the tests.
4. Information generally required by the log format, includes:
   a) Boring number
   b) Date of start and finish of the hole
   c) Name of driller (and of logger if applicable)
   d) Elevation at top of hole
   e) Depth of hole and reason for termination
   f) Diameter of any casing used
   g) Size of hammer and free fall used on casing (if driven)
   h) Blows per 0.3 M to advance casing (if driven)
   i) Description and size of sampler
   j) Size of drive hammer and free fall used on sampler in dynamic field tests
   k) Blow count for each 150 mm to drive sampler
   l) Type of drilling machine used
   m) Length of time to drill each core run or 0.3 M of core run
   n) Length of each core run and amount of core per run
   o) Recovery of sample in inches
   p) Project identification including location.
   q) Client.
5. Notes regarding any other pertinent information and remarks on miscellaneous conditions encountered, such as:
   a) Depth of observed groundwater, elapsed time to observation after completion of drilling, conditions under which observations were made, and comparison with the elevation noted during reconnaissance (if any).
   b) Artesian condition.
   c) Obstructions encountered.
   d) Difficulties in drilling (caving, coring boulders, surging or rise of sands in casing, and caverns).
   e) Loss of circulating water and addition of extra drilling water.
   f) Drilling mud and casing as needed and why.
   g) Odor of recovered sample.
6. Any other information which may be required by policy.

7.12 IMPROPER DRILLING TECHNIQUES

The majority of drilling firms and drillers perform an excellent and valuable service, typically under difficult conditions. Occasionally, through carelessness or ignorance, drilling techniques may be employed that provide questionable or misleading information. In rare instances, actual fraud may occur, when the driller falsifies records to improve his production or simply to avoid work.

The following drilling practices should be prohibited and reported to the project geologist or geotechnical engineer.

- *Careless measurements* of casing and rod lengths and of the "stick-up" of the tools during drilling. This practice leads to uncertain knowledge of the depth to the drill bit and of the relation between the drill bit location and the bottom of the casing. Thus, the driller may wash out considerably below the bottom of the casing, resulting in serious sample disturbance. Conversely, the driller may not wash out far enough, so that the next sample drive begins with the spoon inside the casing. Whenever there is a decreasing blow count on the sampler with penetration, the logger should be on guard for this latter situation. There is a trend in the industry to guess at the measurements of "stick-up," so that depths can be in error by as much as one to two feet.

  While at the rig the logger should make certain that the driller is making depth measurements with a rule and not visually judging ("eyeballing") casing and tool projections above the drill collar. At the same time it is good practice for the logger to note the approximate progress of the bottom of the hole by measuring the "stick-up," or portion of casing rising above the borehole collar. By mentally subtracting the stick-up from the total length of tools down the casing, the inspector can maintain a constant approximate check on the driller's statements of depth.
- *Washing out with a vertical, high-pressure jet.* This generally advances the boring below the bottom of the casing in all types of soils. However, depending on the soil type, it may create an intolerable disturbance. Jets on the chopping bit should be directed at a moderate angle downward and only a minimum water pressure should be used to lift the coarser particles within the casing to the surface.
- *Driving the sampling hammer with a wire winch drum.* In cold weather, friction in the moving parts of the winch and the inherent inertia of the system combine to significantly reduce the impact energy of the hammer. Hammer blow counts may then become erroneously high. Since most machines equipped with wire winch drums are also fitted with catheads, or have cathead power take-offs, the logger should insist that the only acceptable system is that of a fiber rope and a cathead.
- *Excessive turns of the drive rope on the cathead.* During standard penetration testing, a free-falling 64 kg (140 lb.) weight is required for accurate test results. To insure this free-fall, the drive rope should not exceed two full turns on the cathead in order to minimize friction and drag between the rope and the cathead.
- *Uncased borehole advancement in granular soils.* When using water as the drilling fluid, this practice may lead to erroneous information since it often allows the boring to collapse upon removal of the wash rods, making it impossible to obtain representative samples. Those materials that are too heavy to be lifted out by wash water will accumulate at the bottom of the boring. When the sampler is driven, it picks up this wash which may be erroneously classified as "coarse, well-graded sand/gravel," when, in reality, the undisturbed material may be predominantly fine grained and relatively impermeable.
- *Other mistakes in technique* contribute to inaccurate information, e.g., sampling loose, saturated sands without keeping the casing full of water.
- *Fraudulent techniques* such as falsifying the depth of the borehole or the length of casing which was utilized, or obtaining multiple samples from a single split-spoon drive and using them to represent deeper samples, or additional boreholes which were not drilled.

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8.0 HYDROGEOLOGY

Hydrogeologic data are applicable to a variety of problems both directly and indirectly affecting the success of any construction project. Subsurface water can affect the stability of structures, the costs of construction, the costs of maintenance, and the effects of structures on neighboring properties. It is important to predict adverse effects so they can be eliminated or mitigated early in the project, and not come as a surprise during or after construction. Such predictions can be made only on the basis of hydrogeologic facts, and they can be only as accurate as the data on which they are based. It is important, therefore, to gather such data as carefully, accurately, and thoroughly as possible. Although the word hydrogeology emphasizes hydrology, geology is equally important. The geologic conditions of a subsurface-water system must be clearly defined before the hydrology of the system can be correctly understood.

It is the purpose of this section to demonstrate some relationships between transportation structures and subsurface water, and to present some methods by which hydrogeologic information can be acquired, analyzed, displayed, and put to use to prevent, mitigate, or correct undesirable conflicts between transportation structures and subsurface water.

8.1 TERMINOLOGY

There are a number of terms used in subsurface-water discussions that can lead to confusion and misunderstanding. Some of these terms are not common to other disciplines. Some are often used loosely or imprecisely. Others have been refined or altered somewhat over the years, and some new terms have replaced old ones. Therefore a brief glossary is included here. Figure 8-1 illustrates some of these terms. Additional discussions, definitions, uses, and histories of the following terms may be found in publications including Meinzer (1923), Ferris, et al. (1962), Davis and DeWiest (1966), Lohman (1972), Johnson Division, UOP, Inc. (1975), Freeze and Cherry (1979), Fetter (1980), and Todd (1980).

8.1.1 Aquifer

Aquifer is historically defined as a geologic formation that will yield useful quantities of water for a water supply. The term is relative to other available sources of water and to the quantity of water required. Thus, a formation that is an aquifer in one situation may not be so in another situation. This linkage to water supply requirements has made the term difficult and misleading to use in discussions of general subsurface-water occurrence, especially outside the realm of water supply. The nearly synonymous terms *water-bearing material* and *water-bearing zone* may be defined in the broader sense as being any geologic formation or stratum, consolidated or unconsolidated, or geologic structure, such as a fracture or a fault zone, that is capable of transmitting water in sufficient quantity to be either of use or of concern.

8.1.2 Artesian

Artesian is equivalent to *confined.* It can refer to either the water-bearing material, as in *confined aquifer,* or to the water confined in the material, as in *artesian ground water.* The water in a confined material also may be referred to as occurring under *confined conditions* or *artesian conditions.* Confined water is held in the water-bearing material by an overlying material of low permeability called the *confining layer.* Confined water will rise in a well to a level above the top of the water-bearing material, defining the potentiometric surface at that point. If the potentiometric surface is above the land surface, the well will be a flowing well.

8.1.3 Groundwater

Groundwater is that portion of subsurface water that occurs in the zone of saturation. Groundwater is often

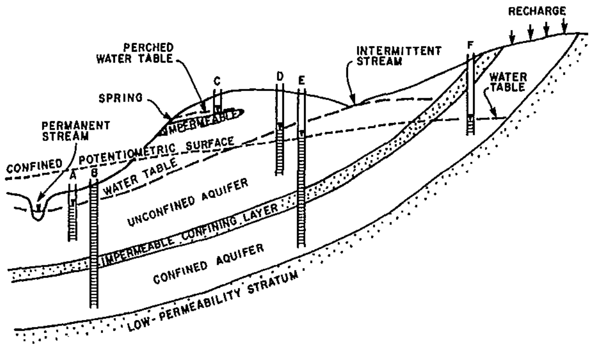

Figure 8-1. Groundwater terminology. The unconfined aquifer is recharged in the higher elevations and discharges in the permanent stream as well as seasonally in the intermittent stream. The confined aquifer is recharged in the highest elevation as shown, and discharges through wells. Wells A, D, C, and F are water-table wells. Wells A and D are in the unconfined aquifer; well C is in the perched unconfined aquifer; and well F is in the unconfined recharge area of the confined aquifer. Wells B and E are artesian wells in the confined aquifer, and B is a flowing well. Note that the potentiometric surface of the unconfined aquifer is in part above and in part below the water table of the unconfined aquifer. Breaks in the confining layer would allow water to leak from the confined aquifer into the unconfined aquifer in the former instance, and from the unconfined aquifer into the confined aquifer in the latter instance. (Haley & Aldrich, Inc.)

used loosely and incorrectly to refer to all water that occurs below the ground surface. The term *subsurface water* should be used in this general case to distinguish water occurring below the surface from water occurring on the surface, or surface water. Subsurface water includes water in the zone of aeration, where it is called *suspended water,* or vadose water, and water in the zone of saturation, *groundwater.* Suspended water is divided into the soil water belt, the intermediate belt, and the *capillary fringe.* The capillary fringe consists of water held immediately above the water table by capillary forces, the height of which depends on the diameter of the interstices. The thickness of the capillary fringe may be from a fraction of an inch in gravel to as much as 8 feet in silt and clay (Johnson Division, UOP, Inc., 1975, p. 20).

8.1.4 Hydraulic Conductivity

Hydraulic conductivity (previously *coefficient of permeability*) is the quantification of the property of permeability. Permeability may be considered in terms of the solid medium alone, in which case it is called intrinsic permeability. It is more useful and convenient to include the fluid as well. Hydraulic conductivity considers properties of both the medium and the fluid (water) that affect the permeability. The current definition of hydraulic conductivity is stated by Lohman (1972, p. 6): "A medium has a *hydraulic conductivity* of unit length per unit time if it will transmit in unit time a unit volume of ground water at the prevailing viscosity through a cross section of unit area, measured at right angles to the direction of flow, under a hydraulic gradient of unit change in head through unit length of flow." It is expressed in the dimensions of velocity.

8.1.5 Permeability

The measure of the ease with which a fluid will pass through a porous medium is called permeability. If a fluid will not pass through a material, that material is said to be *impermeable,* or impervious. In practice these terms are relative, and are used according to whether a material will pass a fluid through in sufficient quantity to be of consequence in a particular situation.

8.1.6 Porosity

Porosity is a measure of the contained interstices of a material. It is expressed quantitatively as the ratio of void space to the total volume of porous material. It is stated as either a decimal fraction or as a percentage, and is dimensionless. *Primary porosity* refers to the original interstices created when a material, such as rock or soil, was formed. Typically, primary porosity is the interstices or pore space between grains, pebbles, or crystals. It is the dominant porosity in unconsolidated materials, such as soil, and in loosely cemented or weakly indurated sedimentary rocks. *Secondary porosity* refers to interstices created after a material was formed. Examples are fractures (joints and faults), openings along bedding planes, solution cavities, cleavage, and schistosity. Secondary porosity is the dominant form in consolidated materials such as well cemented and strongly indurated sedimentary rocks, and it is the only effective porosity in most igneous and metamorphic, or crystalline, rocks.

8.1.7 Potentiometric Surface

Potentiometric surface is defined by Lohman (1972, p. 8) as " . . . an imaginary surface connecting points to which water would rise in tightly cased wells from a given point in an aquifer." Potentiometric surface replaces the older term piezometric surface.

8.1.8 Storage Coefficient

Lohman (1972, p. 8) defines storage coefficient as " . . . The volume of water an aquifer releases from or takes into storage per unit surface area of the aquifer per unit change in head." This term is dimensionless.

8.1.9 Transmissivity

Transmissivity is defined by Lohman (1972, p. 6) as " . . . the rate at which water of the prevailing viscosity is transmitted through a unit width of the aquifer under a unit hydraulic gradient." It is equal to the product of the hydraulic conductivity and the saturated thickness of the aquifer, and is expressed in the dimensions of area divided by time with the preferred units being square feet per day or square meters per day. Transmissivity replaces the old term coefficient of transmissibility.

8.1.10 Unconfined

Unconfined refers to either the water-bearing material, as in *unconfined aquifer,* or to the water in the material, as in *unconfined groundwater.* The water in an unconfined material also may be referred to as occurring under *unconfined conditions* or *water-table conditions.* The potentiometric surface of unconfined groundwater lies everywhere within the waterbearing material, and is often loosely termed the *water table.*

8.1.11 Water Table

Water table is the Upper Surface of the Zone of Saturation in an Unconfined Water-Bearing Material. The water table is the imaginary surface in an unconfined water-bearing material along which the hydrostatic pressure is equal to the atmospheric pressure (Davis and DeWiest, 1966; Lohman, 1972). In coarse grained soils, the water table is near the top of the saturated zone. A *perched water table* occurs where a layer or lens of low permeability material lies within an unsaturated permeable material and restricts the downward movement of water sufficiently to create a localized saturated zone above the general water table. In certain soils a layer of low permeability occurs in the subsoil that prevents downward percolation of water sufficiently that during wet periods a temporarily saturated zone develops. The top of this intermittently saturated zone is referred to as the *seasonal high water table.* It may or may not be perched. A perched zone of saturation that is sufficiently permanent and transmissive may be called a *perched aquifer.*

8.2 USE OF HYDROGEOLOGIC INFORMATION

Before undertaking a hydrogeologic study one must know why that study is to be done, what is the purpose of the study, and what it will contribute to the overall project. The answers determine the objectives, scope and level of effort of the study.

Transportation systems involve an extreme variety of sizes and types of structures. Land transportation systems require some type of roadway, such as a highway or railroad, that covers an area many miles long, but only a small fraction of a mile wide. Such roadways cross areas with different kinds of geologic, and hydrologic conditions so that some problems may be encountered. Other types of structures occupy more equidimensional areas of varying sizes. A very large area of pavement is involved in an airport. Smaller areas of pavement are required in parking lots and maintenance yards. Buildings such as airport terminals, administration and office buildings, maintenance buildings, and so on, also occupy more or less equidimensional areas. This type of area is going to have less variability of geohydrologic conditions than the corridor type of roadway area. Structures that occupy very small areas are more likely to involve homogeneous hydrogeologic conditions. Such structures include bridge and viaduct abutments, pier foundations, and mast and sign foundations. Subsurface structures such as tunnels, deep foundations, and large road cuts are most likely to encounter severe hydrogeologic problems because they frequently intersect, or are located in, the saturated zone. Shallow foundations and small road cuts may or may not intersect ground water, and generally may have less severe hydrogeologic problems.

Transportation structures affect subsurface water in three ways. First, they alter recharge patterns, second, they alter discharge patterns, and third, they alter chemical conditions. The first two ways affect water quantity with respect to the actual amount of water entering the subsurface, and with respect to where that water goes. Since subsurface water flows from areas of recharge (high head or potential) to areas of discharge (low head or potential), any change in hydraulic head in either of these areas will change the direction and rate of water flow. The alteration of chemical conditions includes changes in temperature, pressure, and oxidation/reduction potential in the subsurface, which affects the solubility of various elements, compounds, and minerals, with which the water comes into contact. It also includes the addition of chemicals through the use of roadway deicing salts, and spills of compounds being transported, including the fuel being carried by the transporting vehicle. Changes in chemical conditions are reflected in changes in water quality. These changes are usually, but not always, undesirable and may render the ground water unfit or unavailable for some beneficial uses.

Various types of studies can be made to provide the data required to evaluate different situations. Generally, more than one type of study and level of effort is required on a given project, and the data obtained can be used in the solution of several different problems. Regional studies identify general hydrogeologic conditions in the area surrounding the project. Aquifers or water-bearing zones are identified and their general water yield, their recharge areas, their discharge areas, and their degree of development for water supply are determined. Also general soil types and their drainage characteristics are ascertained.

Although subsurface water does not occur in underground streams (except in some cavernous formations) as popularly misconceived, groundwater flow is variable throughout the subsurface. The natural variability of rocks and soils causes variations in hydraulic conductivity both within a water-bearing material and from one water-bearing zone to another. In unconsolidated materials where primary porosity is dominant, groundwater flow is generalized throughout the material because the interstices are numerous and close together. However, within a generally fine-grained material there may be coarser layers through which water can move more rapidly and in larger quantities than it can through the material as a whole. Bedrock may be impermeable itself, but water moves through it in fractures and other such openings called secondary porosity. Groundwater flow is not so generalized in this material where the flow paths are more widely spaced. In addition there are areas within rock formations where fractures are concentrated or are more open so that water can move more readily through these areas. It is this type of situation that has supported the notion of underground streams, and that may result in unexpected water problems during construction.

The effect of subsurface water on transportation structures is basically to create problems of drainage, which, if not controlled, lead to flooding, unstable earthen structures and foundations, worker and user safety problems, and accelerated deterioration of structures. Some specific ways in which transportation structures and subsurface water interact will be discussed in the following paragraphs.

One of the common transportation structures is the roadway cut in a hillside or hilltop, Figure 8-2. Cuts often intersect a waterbearing zone or aquifer. There are two ways that the structures may be affected. First: the water will flow over the road bed and cause slippery conditions, especially in freezing weather. Second: the water will cause instability of the cut slope contributing to land slides and rock falls.

The effect of the cut on groundwater is to create a new point or area of discharge. This results in a lowering of the water table and a change in the direction of groundwater flow. The lowered water level in the affected aquifer in turn results in the problem of the "dry" well. If the new water level is below the pump intake in a well, the pump obviously cannot draw water. However, the lower water level, even though above the pump intake, can cause a loss of pumping efficiency and well yield. Although a well has not in fact become dry, the well user perceives the loss or reduction of well yield as a drying up of the well. Similarly, lowered water levels can dry up wetlands and springs.

Without geohydrologic information it is not possible to design efficient drainage to prevent flooding of the roadway, to analyze the stability of the cut bank and design stabilizing structures, to determine how

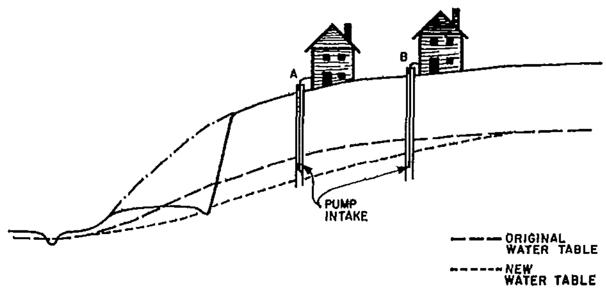

Figure 8-2. Effects of a road cut. The excavation for a road cut in a hillside intersects and lowers the water table. The new water level in well A is below the pump intake and well A "dries up." In well B the new water level has reduced the column of water over the pump intake reducing the pump efficiency that together with less available water results in lower yield from well B. (Courtesy Arch Associates)

much the groundwater level will be lowered, to what distance the water level will be effected, to determine what steps may be taken to prevent damage to wetlands or water supplies, and to determine what remedial measures may be taken to repair or compensate for unavoidable damage.

Another general type of transportation structure is the large area of pavement and drainage diversion. Highways and airports are major examples. Where the structures are over recharge areas, they can seriously reduce the amount of water available to recharge the ground water. This situation is usually advantageous in relation to the structure in areas of shallow water table. Water levels will become lower than under natural conditions, thus improving the stability of the subbase and reducing problems such as frost heave. The structure, however, can have a major impact on ground water in this situation. By eliminating major recharge the amount and direction of groundwater flow is significantly altered. Water levels are lowered, aquifers may be destroyed or seriously depleted, and water quality may be changed. Where paved areas are placed over zones of discharge, such as a highway through a marsh or an airport in a filled wetland, the adverse effects are primarily on the structures. In this case drainage is difficult to control completely, subbase may be permanently or seasonally saturated and weak, and frost heaving can be a major problem in cold climates.

One other aspect of paved areas is the diversion of drainage. When the diverted water is discharged to streams the flow in the receiving stream is invariably increased, possibly causing erosion problems. When the drainage is diverted to the land surface erosion problems may occur if proper methods of discharge are not used. The other effect is to create a new area of groundwater recharge in the area of the drainage discharge and to some extent in the area under permeable drainage ditches crossing unsaturated ground. This combines with the effects of recharge elimination previously discussed to further alter flow directions and groundwater availability.

An aspect of transportation system construction seldom considered to be of consequence is the borrow pit. Figure 8-3 shows the effect that a sand and gravel pit may have on the local groundwater table. When the borrow pit is above the saturated zone it can act to increase recharge to groundwater by creating an area of high infiltration rate. On the other side of the question, when the water table lies within desirable construction material it prevents removal of that material unless methods such as dewatering or dredging are employed. These methods add to the cost of removal of the material, and may also result in lowering

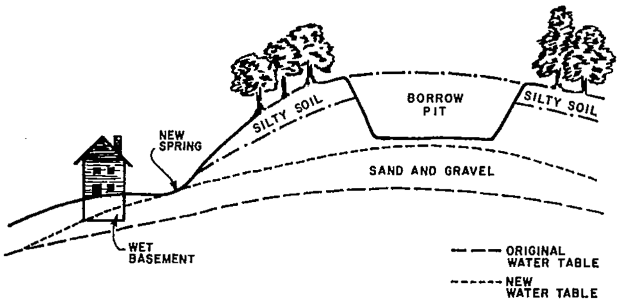

Figure 8-3. Effects of a borrow pit. A sand and gravel pit is excavated above the water table in a recharge area. Prior to excavation part of the precipitation was lost to surface runoff due to slowly permeable soil, and part was lost to evapotranspiration by the vegetation. The excavation through the silty soil and beyond the root zone, and the removal of vegetation, allows nearly all of the precipitation to infiltrate into the highly permeable sand and gravel. The amount of recharge to the groundwater has increased, and the water table rises. A new spring is formed, and a previously dry basement becomes wet. (Courtesy Arch Associates)

of the groundwater table in the vicinity of the borrow pit.

Transportation systems can affect subsurface water quality in several ways. One is that by altering the groundwater flow system the chemical equilibrium of the water and subsurface materials may also be changed. Lowering of the water table may expose minerals to an oxygen-rich environment where they were previously in an oxygen-poor environment, thus changing the oxidation/reduction potential. This will alter the rate of weathering and solubility of the minerals. Another problem may be that when groundwater flow patterns are changed areas of poor water quality may move into areas of good water quality and contaminate them. For example, a section of an aquifer may be avoided because of poor water quality, but the altered flow may now bring that poor quality water into the section of the aquifer in which water-supply wells have been developed, thereby damaging or destroying the water supplies.

A more widespread and well documented way in which subsurface-water quality is affected by transportation structures is the deliberate and accidental depositing of liquids or soluble solids on the ground. The use of salt for deicing highways is a particularly widespread example in which water supplies have been degraded by runoff either from uncovered storage piles or from the roadways themselves. Runoff water can carry chloride in concentrations up to several thousand milligrams per liter, or more than ten times the federal drinking water standard of 250 milligrams per liter. Similar problems can result from the use of herbicides to control vegetation along roadways. Again the source may be at the storage area or at the areas of roadway where the herbicides have been applied.

One other major source of polluting chemicals is entirely random. That is the accidental spillage of chemicals and fuels along roadways and railroads. The materials can be carried by drainage water to surface-water bodies, or into the ground to recharge groundwater with pollutants. With prior knowledge of groundwater occurrence and use along the route of a roadway, measures can be taken to prevent or minimize these kinds of problems. This knowledge also provides the ability to react appropriately and rapidly in case of accidental spills.

8.2.1 Environmental Effects of Construction

One of the problems that has come to the forefront in recent years is the effects of construction on the environment. The environment encompasses a multitude of concerns, among which is groundwater resources. There are many ways, often subtle, in which the groundwater system can be affected by construction; some of these have been discussed in the preceding section. However, because groundwater is rarely static, and because areas affected by the transportation system construction can also be affected by other activities of man and by natural impacts at the same time, it is usually difficult, at best, to determine what has caused what effect or portion of an effect. If there are no specific data to indicate conditions prior to construction (the usual case) it is virtually impossible to demonstrate convincingly the effects or lack of effects of the construction.

It is a good idea to establish a pre-construction baseline by conducting a hydrogeologic investigation prior to the start of any construction activities. Such a study should include all geohydrologic data developed on site for construction purposes as well as the data generated primarily for baseline purposes, including gathering data off site where possible. The object is to identify recharge areas, flow directions, discharge areas, and water quality prior to any disturbance by construction, or even certain investigation activities. In most cases it is prudent to go out well in advance of construction and inventory all water wells, springs, and other water features within one-half mile or more of the project area. The purpose of this inventory is to know where all wells and springs are located, what their present condition is, and if possible, their original condition and past history. It is also useful to obtain water samples from at least some of the water supplies and to test them for certain possible pollutants. With this information it is possible to tell ahead of time where problems may occur, and how such problems may be avoided or corrected. Also very important, the validity of complaints of damage after construction commences can be evaluated from a factual basis, saving considerable legal expense and helping to preserve good public relations.

8.3 DATA ACQUISITION

The first step in a hydrogeologic investigation is to gather information. There are two general categories of information that can be used—existing data, and new data.

The first category consists of data that may be obtained from published reports on the geology, hydrology (surface and subsurface), and soils of the area in question. These reports may be obtained from governmental agencies (federal, state, and local), universities, trade journals and other periodicals, libraries, and occasionally from private companies. Unpublished data also exist in the files of many of these sources, and it is frequently possible to examine such files. Existing data can be considered to include readily accessible data that may be obtained by reconnaissance field mapping of geologic and hydrologic data visible on the surface. This would include items such as location and cursory examination of rock outcrops and quarries, soil conditions, springs and seeps, surface drainage, and vegetation. Examination of existing aerial photographs also falls into this category. These sources of information are used in reconnaissance and preliminary-level investigations. Existing data can be gathered and evaluated in a relatively short time and at low cost. They provide a basis on which to evaluate the feasibility of a proposed project. They also provide the basis on which to evaluate the need for, and the requirements of, a more detailed investigation that would include the second category of information.

New data consist of information that is developed specifically for the project at hand. They are gathered primarily by field methods involving test pits and borings, observation wells, the use of geophysical surveys, and chemical analysis of water samples. They also involve the use of a wide variety of tests that may be conducted either in the field or in the laboratory. New data also include the examination of new aerial photographs taken of the area of interest for the par-

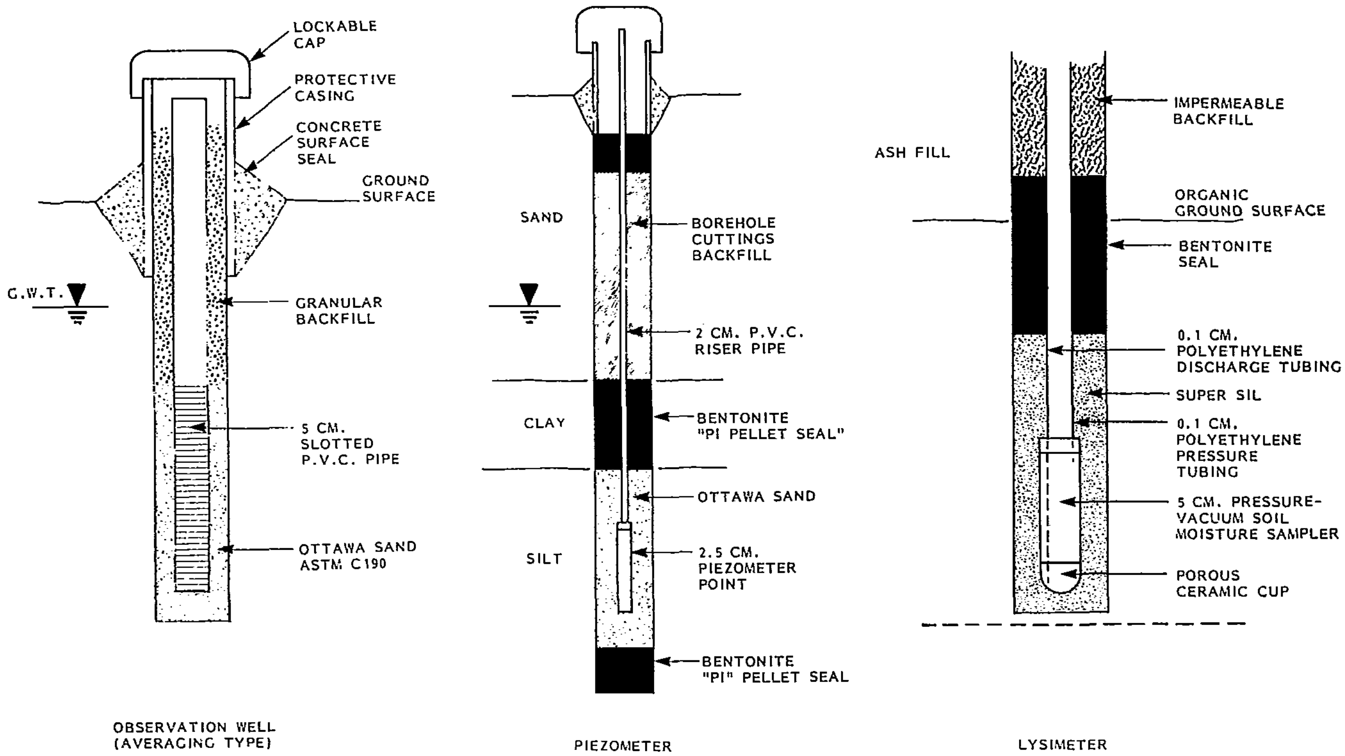

Figure 8-4. Sampling Monitoring Installations (Haley & Aldrich, Inc.)

ticular project. New data generally require a longer time frame and considerably greater cost to obtain than do existing data.

The level of effort at which a hydrogeologic investigation is conducted will depend upon the type of structure or structures involved in the project, the areal extent of the project, and the overall budget of the project.

8.3.1 Observation Wells

Most commonly an observation well is a hole that has been bored into the ground to some depth into the saturated zoned, and fitted with a casing or a well point in order to maintain an open hole over a period of time. It is good practice to place a protective cap over the casing or well point to prevent damage from both accidents and vandalism. Figure 8-4 shows typical observation well, piezometer, and lysimeter construction schemes.

Wells may be drilled purely as observation wells; however, it is common practice to convert test borings to observation wells. Existing water wells in the area can also be used. Shallow observation wells can be made by installing a well point in a backhoe pit when the pit is filled although these are of limited value in that there is such a large disturbed area around the well.

Observation wells are most often used to measure water levels. These measurements may be made periodically by hand or automatically by a continuous recorder. A common hand instrument is the steel tape. When carpenter's chalk or keel is rubbed on the lower two to three feet of a tape graduated in hundredths of a foot it can be used to measure water levels to an accuracy of about 1.5 mm (0.005 ft.). A second hand instrument is a simple, mechanical sounding device attached to a surveyor's tape. The device is a 4-inch length of 0.5-inch diameter tubing, capped at one end and open at the other. The capped end is attached to the tape by means of a swivel clip. As the tape with the sounding device is lowered into the observation well, a distinctive sound is heard when the open end of the device contacts the water surface. A third hand instrument is the electric probe, which consists of two wires and an ammeter that registers a current when the circuit is closed by the ends of the wires being immersed in the water. This instrument can be used with an accuracy equivalent to the steel tape, and it is more convenient than the tape when water depths exceed 30 m (100 ft.).

The most common method of measuring water levels in a pumping well is the air-line method, which works on the principle of measuring the air pressure needed to force the water out of a tube that extends down the well to some known depth below the water level. The air pressure is then converted to an equivalent column of water above the bottom of the air line. This method is not nearly as accurate as the steel tape or electric probe, but it is usually sufficient and the most practical method for use in a pumping well.

Sonic methods can also be used to measure approximate water levels. These methods depend on measuring the length of time required for a sound to travel down the well, reflect off the water surface, and return to the top of the well. Air temperature in the well, which can change with depth, affects the velocity of sound, and must be known for accurate readings. Sonic devices can vary from a tap on the casing times with a stop watch to sophisticated automatic electronic sounding and recording equipment.

The most widely used automatic water level measuring device is the mechanical, float-actuated, drum recorder. This device can be geared to provide a continuous water level record for periods of time ranging from 4 hours to one month, and can record water levels with an accuracy of 0.3 mm (0.001 ft.). A less common device relies on an electronically actuated pressure transducer placed in the well at some depth below the water level. Conversion of the electric signal to an equivalent column of water is made automatically and is recorded continuously on a strip chart. With proper calibration this device will also measure water levels accurate to 1.5 mm (0.005 ft.). More detailed descriptions of these devices and their operation can be found in Davis and DeWiest (1966), Johnson Division, UOP, Inc. (1975), and U.S. Department of the Interior (1977).

Observation wells are also used to measure the water-bearing characteristics of the materials that they penetrate. Bore-hole permeability tests are conducted during drilling of the observation well as the hole is advanced. These tests may also be conducted after the boring is completed but before the well is installed. Permeability tests are discussed in section 8.4.3. After the observation well is completed, pumping tests can be conducted as discussed in section 8.4.3. The observation well may be used either as a pumping well, for water level measurements during pumping of another well, or for both purposes alternately.

Finally, an observation well may be used to obtain samples of water for chemical analysis to be used in water quality studies. Water samples should be as representative as possible of water as it occurs in the water-bearing material of interest. The chemistry of the water standing in a well will change quite rapidly due to reduced pressure in the well, greater exposure to air, contact with casing and screening materials, and other factors. Therefore, water should be removed from an observation well prior to sampling in order to remove any stagnant or unrepresentative water.

Some judgment is involved in how long to pump a well, or otherwise remove water, before sampling, but a widely used standard is to remove an amount of water equal to at least three times the volume of water standing in the well. Various conditions frequently make it impossible or impractical to purge a well to the desired extent. The extreme case is when a very low-yielding material is being investigated. Then it may be difficult to obtain even enough water at a single sampling to perform the desired analyses. In any case when a sample is obtained, the duration and rate of water removal prior to sampling should be recorded along with all other conditions of the well and the water, such as whether or not the well is actively used and last usage, depth to water before and after purging and sampling, depth of pump intake or the depth to which other sampling device was lowered, clarity of the water before and after purging and sampling, water temperature, and so on. Water samples should be analyzed as soon after sampling as possible. Details of sampling procedures and sample preservation may be found in American Public Health Association (1965), Rainwater and Thatcher (1960), U.S. Environmental Protection Agency (1977), and U.S. Environmental Protection Agency (1981).

8.3.2 Piezometers

A piezometer is a specialized type of observation well designed to determine pore pressure in soil, rock, or other porous material. The piezometer differs from the general observation well in that it is open only to a particular point in the material so that the water level in the piezometer indicates the hydraulic pressure at that point. A general observation well on the other hand is usually open to some thickness of the porous material and is indicative of the average potential in the material over that interval. Pore pressure is determined by subtracting the elevation head (the distance of the point of measurement above some arbitrary datum) from the hydraulic pressure (the height of the potentiometric surface above the same arbitrary datum). The potentiometric surface can be measured in an adjacent observation well or in a second piezometer at the water surface.

A type of piezometer installation is shown in Figure 8-4. A piezometer may consist of a pipe or casing that is drilled or driven to the desired depth of measurement. With the pipe open only at the bottom, water can enter only at the depth of interest and will rise in the pipe in accordance with the hydraulic pressure at that depth. More sophisticated types of piezometers consist of a porous tip sealed into a soil layer and connected to the surface by fluid-filled tubes. Several of these devices may be placed at various depths in one boring. For more accurate readings, a mechanical or electrical pressure transducer is placed in the porous tip. This type of piezometer will only measure pore pressures in the saturated zone. Pore pressure in unsaturated materials is negative and is often called soil-moisture suction or tension, and the piezometer is called a tensiometer. More detailed discussion of pore pressure probes is found in *Drainage of Agricultural Land* (U.S. Soil Conservation Service, 1973).

8.4 DATA ANALYSIS

The second half of a hydrogeologic investigation is using the accumulated data to understand the subsurface-water conditions in and around the project area. Transportation structures and parts of structures that will be affected by subsurface water now can be identified, as can the effects of structures on the subsurface water and the surrounding environment. The magnitude of those effects can be calculated, alternative procedures to eliminate or minimize effects can be compared, and the optimum procedures can be selected. Some common techniques for analyzing geohydrologic data will be discussed in the following sections. Successful analysis of geohydrologic data requires understanding of the limitations of the data and of the techniques of analysis. In this way, confidence in the analysis can be maintained, and the extent to which actual conditions may vary from calculated conditions can be appreciated. There are two goals to be attained: One is to describe conditions as they exist in an accurate manner, in order to predict accurately the changes in those conditions that may result from various actions; the second is, to be prepared for conditions and responses as they actually occur.

8.4.1 Potentiometric Surface

A basic objective of groundwater analysis is to define the potentiometric surface. This is done by plotting water elevations from observation well data, and drawing lines of equal elevation, which are for practical purposes, equipotential lines. A minimum of three points of elevation is required to define a plane, but this will yield only the roughest approximation of the potentiometric surface, which is normally an irregular curved surface. The water table is the potentiometric surface in an unconfined water-bearing material. It usually more or less reflects the surface topography, whereas the potentiometric surface in a confined material may have little or no resemblance to surface topography. Thus, many elevation points (observation wells) are desirable to clearly define the potentiometric surfaces. In practice, it is seldom possible to achieve the desired density of data points, and the potentiometric surface map becomes less useful as the number of data points decreases per unit of area. It is sometimes necessary to attain a greater density of data around a problematic site. This results in an uneven distribution of data over the general area of interest, and a variable degree of accuracy from one part of the area to another.

The potentiometric surface map shows where recharge and discharge occur, and the directions of groundwater flow. Water flow is at right angles to the equipotential lines, from areas of high potential to areas of low potential.

Water levels may vary with time and in various cycles. Thus, it is important to obtain water-level measurements in all observation wells as close to the same time as possible. However, if fluctuations are small, a potentiometric surface can be drawn on the basis of water levels at different dates provided that a contour interval larger than the amount of fluctuation is used. Such a map is useful in reconnaissance and even preliminary investigations to approximate the flow system.

8.4.2 Flow Nets

A second technique of subsurface-water analysis is flow-net analysis. A flow net is similar to, and may be based on, the potentiometric surface map. Both are based on lines of equal potential, and both are two-dimensional representations of a three-dimensional system. The flow net, however, may portray conditions in the basically horizontal direction (map view), or in the vertical plane (cross-sectional view).

A flow net consists of two families of lines, and is constructed using some data and considerable judgment. The first family of lines to be drawn is the equipotential lines, or lines of equal head. Then the second family of lines, the flow lines, is drawn. Flow lines must be everywhere orthogonal to the equipotential lines. That is, they must intersect at right angles. Only a few of the infinite number of actual equipotential lines and flow lines are shown, and they are selected in such a way as to form a grid of squares over the flow system. However, since both families of lines are normally curves, the units of the grid are curvilinear and not true squares although the corners are right angles and the mean distance between the two pairs of opposite sides are equal.

A flow net may be used to calculate subsurface flow across a site, or to analyze the flow to a point on the site. It may be used as well to show relative volumes of flow through different sections of the system. This is possible because the net is constructed to divide the flow equally between adjacent flow lines, and to divide the total head drop across the system equally between equipotential lines. In order to make the calculations the hydraulic conductivity must be determined, by actual measurement (preferably) or by estimation. These calculations are valid only if the water-bearing material is homogeneous and isotropic, and is either of infinite areal extent or has identical boundary conditions in all directions. These conditions are seldom truly met in nature and so the calculations made by flow-net analysis are always more or less an approximation.

There are situations where the conditions are reasonably close to homogeneous and isotropic to a sufficient distance beyond the area of interest that the calculations are suitably accurate for many purposes. The further the natural conditions depart from the ideal the less useful are the calculations, although the method may still provide a useful approximation of the flow system. It is possible to adjust flow nets for variations in homogeneity, isotropy, or boundary conditions, but the point is quickly reached where the system is too complex for the amount of data available. The effort involved in constructing a valid flow net and the complexity of the analysis in these situations soon exceeds the value of the results of the flow-net analysis. Details of the construction and use of flow nets are well presented in Cedergren (1967), and Freeze and Cherry (1979).

8.5 SCHEDULING

Hydrogeology should be an integral part of any project. In the planning stages it should be used in comparing alternative routes or sites as to potential impacts on the environment and on construction with regard to required techniques, costs, and safety. The early planning stages is the time to begin gathering existing data. Literature search and inquiries as to sources of existing data can be made and enough general information acquired to identify potential problems and problem areas. When planning advances to site-specific stages, more detailed existing data can be gathered. Reconnaissance level field surveys are appropriate at this time, aerial photograph interpretation should be done, and base-line environmental surveys should commence. As soon as legal access to the route or site is established detailed field investigations should begin to develop new data and to verify existing data.

The hydrogeologic investigation should proceed in two phases. The first phase is to establish an understanding of subsurface-water conditions over the route or site so that specific effects of construction on the subsurface-water system, and of the subsurface-water system on the structure, can be identified. Generally, but not necessarily, these effects will be problems that must be resolved in some way. The second phase is devoted to developing responses to the effects identified in the first phase. This would include more detailed investigation and testing as needed to develop methods of preventing the occurrence of adverse effects, of mitigating or correcting unavoidable adverse effects, and taking advantage of favorable conditions. This work should be completed well in advance of construction as it will be important in developing construction specifications and as a basis for contractors to bid on the work. The environmental base-line data collection can, and often should, continue until the start of construction, at which point the base line necessarily ends. Subsurface-water monitoring should continue during the construction phase to verify predicted effects, and to modify construction practices as actual conditions are discovered. In some cases monitoring should continue for some time after completion of the project, and may be an appropriate part of routine maintenance of the structure.

8.6 PRESENTATION

The final step in the use of the data is to present it in a form that is meaningful to others. This section will suggest ways in which geohydrologic information can be presented or displayed so that the uses of those data and the conclusions drawn from them can be clearly presented.

Well logs, test boring logs, and test pit logs are similar means of clearly and concisely presenting all the information concerning subsurface explorations. They generally contain verbal descriptions and symbolic notation explaining the equipment used, methods of sampling the soil and rock, intervals sampled, depth of the exploration, materials encountered, unusual conditions, water conditions, tests run, observation well installation, and other pertinent data. Detailed logs of installed observation wells and piezometers should also be maintained.

Other graphic and tabular forms of presentation can be used to display basic data, calculations, and extensions of the data. All types of test data, such as pump tests, permeability tests, and chemical analyses should be tabulated for easy retrieval, comparison, and transfer to other forms of display. Three widely used graphic forms of display of water quality information are bar and line graphs, shape diagrams, and trilinear diagrams (Freeze and Cherry, 1979). These methods provide clear comparisons of samples so that the differences or similarities are readily apparent.

Any number of maps can be compiled to present various aspects of the geohydrologic system of the area of investigation. Basic geologic maps can be modified to show differences in water-bearing characteristics of the soil and rock materials present. These maps can be based on hydraulic conductivity, transmissivity, water use, or any other characteristic of interest. Potentiometric surface maps (section 8.4.1) and flow nets (section 8.4.2) can be drawn. These show areas of recharge and discharge and patterns of ground-water flow. A preconstruction potentiometric surface can be drawn to show undisturbed conditions. A during-construction or after-construction map can then be drawn to show the effects of dewatering or other changes in the flow system resulting from the construction. Water quality maps can be drawn that show by symbols the patterns of occurrence of various elements, or groups of elements, in the subsurface water, or they may show patterns of overall quality. Areas of high quality, poor quality, and pollution can be designated. Comparisons of these various types of maps by transparent overlay or simply side-by-side comparison may reveal why the water moves in the direction that it does, or why certain chemical variations occur where they do, or why an area is or is not affected by certain changes in the flow system. Finally, aspects of all of these types of maps can be combined into one map of groundwater hazards or subsurface-water problem areas. These show what transportation structures will be adversely affected by subsurface water, as well as showing in what ways adjacent areas will be affected by the transportation structures.

An additional technique, usually used in conjunction with various maps, is the construction of cross sections. These vertical slices through the area show the same kinds of information that are displayed on the maps, but they present that information in the third dimension, depth. Understanding the vertical variations in a flow system is often the key to understanding the lateral variations.

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9.0 LABORATORY TESTING OF SOIL AND ROCK

The purpose of laboratory testing is to provide the basic data with which to classify soils and to quantitatively assess their engineering properties. Laboratory tests should be carefully performed following the proper testing procedures for the soil involved and the information desired. A thorough understanding of the engineering properties of soils is essential not only to the use of current methods in the design of foundations and earth structures, but also as the key to further progress in geotechnical engineering.

Laboratory tests of soils may be grouped broadly into two general classes:

- *Classification tests:* may be performed on either disturbed or "undisturbed" samples.
- *Quantitative tests:* for hydraulic conductivity (permeability), compressibility and shear strength. These tests are generally performed on undisturbed samples, except for materials to be placed as controlled fill or materials that do not have an unstable soil structure. In these cases, tests may be performed on specimens prepared in the laboratory.

Test results are no better than the samples on which they are performed, or the care used in performing them.

Procedures for most soil tests are given generally by AASHTO and by ASTM. Appropriate test procedures are referenced in Appendix D for the soil tests discussed in the following sections. Techniques for dynamic testing are in a state of development. Consequently, they are changing rapidly and standardized test procedures do not exist. Before undertaking dynamic tests, recent literature should be reviewed and the assistance of an expert in the field sought.

9.1 REQUIREMENTS OF THE LABORATORY

9.1.1 Equipment

In general, a laboratory should be located on a ground floor or basement with a solid floor free of traffic or machine vibrations. The laboratory should be fully equipped with modern soil testing equipment suitable for performing the required classification and property tests. Ideally, separate areas should be designated for dust producing activities, such as sieve analysis and sample processing.

In general, equipment is arranged in areas according to the class or type of testing to provide the most efficient use of personnel and space. If possible, the temperature of the entire laboratory should be controlled. However, if temperature controlled space is limited, this space should be used for consolidation, triaxial and permeability testing. A humid room large enough for storage of undisturbed samples and preparation of test specimens should be available.

Regular inspection and calibration of testing equipment should be performed to maintain accuracy. Malfunctioning equipment should be removed from service until repair, replacement and recalibration have been completed.

9.1.2 Personnel

All laboratory testing should be performed and supervised by personnel who are qualified by training and experience to undertake the assigned testing. They must be thoroughly familiar with the equipment, test procedures and good laboratory techniques in general. Personnel must appreciate the purpose of each test they perform.

Insofar as possible, there should be programs for the indoctrination and training of personnel. New personnel or personnel seeking qualification in additional procedures should receive extensive on-the-job training.

9.1.3 Quality Assurance (Control)

In general, quality assurance (control) should provide control over activities affecting the quality of laboratory testing to an extent consistent with their importance to the results. Quality control should provide for the review and assessment of the following activities as a minimum:

- Handling and storage of soils.
- Specimen preparation.
- Adherence to proper testing procedures.
- Accuracy in measurements.
- Equipment maintenance.
- Review and checking of test data.
- Presentation of test data.

Personnel who are involved in soil testing must constantly be aware of the importance of accuracy in measurements. Inaccurate measurements will produce test results which are useless and misleading. The general philosophy in the laboratory should be that one good test is not only far better than many poor tests, but it is less expensive and less likely to permit a misjudgement in design.

9.2 PLANNING PROJECT-RELATED TEST PROGRAMS

The amount of laboratory testing required for foundation design will vary with each project depending on whether the foundation soils within a given geographic area have been adequately defined by previous explorations, the character of the soils and the requirements of the project. The decision regarding the type and number of laboratory tests to be performed for a project should be based on the complexity of the subsurface conditions, the magnitude and distribution of foundation loads, importance of differential settlement, and local experience.

Laboratory tests should be selected to give the desired and necessary data as economically as possible. Complicated and expensive tests are justified only if the data will reduce costs or risk of a costly failure. In general, relatively few carefully conducted tests on specimens selected to cover the range of soil properties with the results correlated by classification or index tests will give good usable data.

The primary tests of importance to geotechnical engineers, in approximate order of increasing cost, are:

- Visual examination
- Natural moisture content
- Liquid and plastic limit
- Grain-size analysis (mechanical)
- Laboratory vane shear
- Unconfined compression
- Moisture-density or relative density
- California Bearing Ratio
- Permeability
- Direct shear
- Triaxial compression
- Consolidation

9.3 SAMPLE HANDLING

9.3.1 Storage and Preparation

Samples should be identified and logged in when received at the laboratory. Each sample, as well as boxes of samples, should be properly labeled as to name and number of project, boring and sample number, date of sampling, borehole location and depth of recovery. Any field notes relative to deviations from standard drilling and sampling procedures, state of disturbance or unusual characteristics should be recorded for later use.

Samples should be tested as soon as possible after their arrival. Samples which are not scheduled for immediate testing should be resealed, if necessary, to minimize loss of moisture and maintain samples at natural moisture content prior to testing. Although immediate testing is desirable, sample storage may be necessary.

Undisturbed samples of cohesive soils should be sealed with a nonshrinking flexible micro-crystalline wax. The wax should be installed in several layers to minimize shrinkage and cracking. Samples should be stored in a humid room with relative humidity near 100 percent, if possible. The temperature of the humid room should be approximately at mean ground temperature in order to minimize bacterial action in the organics that may be in the soil samples. Tubes should be stored vertically to minimize the formation of air channels by plastic flow of the wax. Samples should not be stored one on top of the other since higher stress increases plastic flow.

Samples which are generally unaffected by changes in moisture content may be stored in glass jars, canvas or heavy bags, cans or bins. Each container should have a label or tag giving the necessary sample data.

The handling and exposure to atmosphere of undisturbed samples should be kept to a minimum. If possible, all preparation of undisturbed test specimens should be done in a humid room. Sealing wax should be cut into strips with a sharp knife or saw during removal to minimize disturbance. Extrusion of a sample should be in the same direction as sampling to minimize disturbance. Excess portions of tubes should be removed prior to extrusion. In order to minimize disturbance due to side friction, the sample tube may be cut into predetermined lengths for tests prior to extruding. However, if the sampled soil is layered, it may be necessary to extrude the sample from the full-length tube and assign tests appropriate to portions of the sample as it is extruded. Any burrs should be removed from the inside of the tube prior to extrusion.

The sample should be extruded using a device that applies a steady force. Hand screw and hydraulic devices are the most commonly used equipment. A record of tube sample should be maintained indicating the sample depth, the sample length, soil description, location of test specimens, results of classification tests and any apparent disturbance or unusual characteristics. Test specimens should not be handled with bare hands nor should moisture be wiped off the specimen. A sheet of wax paper can be used to minimize moisture loss during handling and specimen preparation. The specimen should be supported over its entire length when transporting from one area to another.

Proper care in trimming test specimens helps to minimize disturbance. Trimming of test specimens is normally performed using a piano wire trimmer and soil lathe. Test specimens for consolidation and direct shear testing are often trimmed into collars using specialized lathes or cutting shoes to minimize handling.

Classification tests such as liquid and plastic limits, grain size analysis and specific gravity do not require undisturbed samples. However, care should be given not to mix soils from different layers prior to testing. It is also important that laboratory tests be performed on samples representative of the soil encountered in the field. It is also important that the technician identify and record the nature and description of the test specimen.

9.3.2 Disturbance

Disturbances to soil samples may be classified in five basic types (Hvorslev, 1949), proceeding from relatively slight to more severe:

- Change in stress conditions
- Change in water content and void ratio
- Disturbance of the soil structure
- Chemical Changes
- Mixing and segregation of soil constituents

The influence of disturbance on laboratory test results depends on the type and degree of disturbance and on the nature of the soil and the type of testing.

9.3.2.1 Change in Stress Conditions. The stress changes which occur during boring and sampling can be minimized by the use of proper methods and equipment. However, a total stress reduction to atmospheric pressure cannot be avoided when the sample is removed from the tube or liner and during preparation of the test specimens. Hvorslev (1949) presents a discussion of the consequences of such stress reduction for various soil types.

9.3.2.2 Change in Water Content and Void Ratio. In a non-gaseous, fully saturated soil, a change in void ratio (volume) is accompanied by a corresponding change in water content. However, the void ratio of gaseous soils can be changed without a change in water content and the water content of partially saturated soils with interconnecting voids may be changed with only minor changes in void ratio.

Changes in volume may occur before, during and after sampling. Volume changes resulting from expansion and displacement of soil during drilling and sampling usually affect only the upper part of the sample taken from a borehole. Volume changes associated with extrusion from sampling tubes can be minimized by cutting the tube into appropriate lengths for testing prior to extruding the soil.

9.3.2.3 Disturbance of the Soil Structure. Disturbance of soil structure can occur before, during and after drilling and sampling. For sampling in boreholes, the disturbance before sampling is usually limited to the upper part of the sample. By the use of proper equipment and methods, disturbance can be minimized, especially for the central portion of the sample. However, the lower part may be disturbed when separating the sample from the subsoil. Disturbance after sampling can be minimized by proper care in sealing, shipment and handling of the sample.

9.3.2.4 Chemical Changes. Disturbance associated with chemical changes is usually caused by infiltration of wash water or drilling fluid in the sample, oxidation after sampling and during specimen preparation, contact with the sample containers and electrical charge. The greatest danger of chemical changes is associated with samples stored in untreated steel containers for long periods of time. Containers should be coated with lacquer and the sealing caps should be an inert material or of the same type as the container.

9.3.2.5 Mixing and Segregation of Soil Constituents. Mixing and segregation of constituents is generally associated with sampling operations and can be minimized by using proper drilling and sampling procedures. When only soil layers in close proximity have been mixed, the sample as a whole may be representative of the average condition and acceptable for identification and determination of the suitability for construction purposes.

9.3.3 Undisturbed Soil Samples

Due to the reduction of total stresses during sampling and specimen preparation, a truly undisturbed sample cannot be obtained.

However, a sample may be suitable for laboratory testing and for practical purposes considered undisturbed if the following requirements are met:

- No disturbance of the soil structure
- No change in water content or void ratio
- No change in constituents or chemical composition

Because it is very difficult to evaluate whether these requirements are satisfied, Hvorslev (1949) proposed that the strict requirements for undisturbed sampling be replaced by the following practical or modified requirements.

- The specific recovery ratio shall not be greater than 1.00 nor smaller than (1–2Ci), where Ci is the inside clearance ratio of the cutting edge. If entrance of excess soil is prevented, it is generally sufficient that the total recovery ratio (ratio of sample length to push length) be equal to or slightly smaller than 1.00.
- On the surface or in sliced sections of the sample, there must be no visible distortions, planes of failure, or pitting attributed to the sampling operation or handling of the samples.
- The net length and weight of sample and the results of other control tests must not change during shipment, storage and handling of the sample.

9.4 LABORATORY ASPECTS OF SOIL CLASSIFICATION

The Unified Soil Classification System (USCS) (U.S. Army, 1953) discussed in Appendix E, is based on the identification of soils according to the type and predominance of the constituents considering the following:

- Grain size
- Gradation (shape of grain size distribution curve)
- Plasticity and compressibility

The system divides soils into three major divisions:

- Coarse grained (more than 50 percent retained on the No. 200 sieve)
- Fine grained (more than 50 percent passing the No. 200 sieve)
- Highly organic (peaty) soils

Coarse grained soils are classified as to their particle size and shape of the grain size distribution curve. Fine grained soils are classified as to their position on the plasticity chart.

9.4.1 Grain Size Analysis

The grain size distribution of a coarse soil can be very useful for both classification and evaluation of specification criteria. To some extent, the grain size curve for sands can be related to engineering behavior such as soil permeability, frost susceptibility, angle of internal friction, bearing capacity and liquefaction potential. The behavior of fine grained soils (silty clays and clays) are more a function of the degree of plasticity, type of mineral and geologic history.

The grain size distribution of a soil is expressed as a plot of percent finer by weight versus diameter in millimeters. The grain size distribution of a coarse-grained soil is determined by sieve analysis while a hydrometer test is used for fine-grained soils. As a general note, if nearly all (approximately 80 percent) of the particles of a soil are greater than a No. 200 sieve (openings of 0.074 mm), the sieve analysis is used. For soils which are nearly all finer than a No. 200 sieve, the hydrometer test is used. Soils which have portions of their particles both larger and smaller than a No. 200 sieve require a combined analysis.

A sieve analysis consists of passing a sample through a set of standard sieves and weighing the amount retained on each sieve. The recommended test procedure for grain size analysis is included in Appendix C. The results are plotted on a grain size distribution curve in the form of percent fines by weight versus particle size to a log scale. The shape of the grain size curve is indicative of the grading. A "uniformly" graded soil has a grain size curve that is nearly vertical and a "well-graded" soil has a more flat curve that extends across several log cycles of particle size.

The uniformity of a soil may be represented by the uniformity coefficient, Cu, defined as D60/D10, where D60 is the particle size for which 60 percent of the specimen weight is finer and D10 is the particle size for which 10 percent of the specimen weight is finer. The coefficient of curvature (Cc), defined as $(D30)^2/(D60 \times D10)$, is also used to describe particle size characteristics. In accordance with the Unified Soil Classification System:

Soil	*Range*
Poorly (uniform) graded	Cu < 4
Well-graded gravel	Cu > 4, 1 < Cc < 3
Well-graded sand	Cu > 6, 1 < Cc < 3

The hydrometer (sedimentation) analysis is based on Stokes law, which relates the velocity at which a spherical particle falls through a fluid medium to the diameter and specific gravity of the particle and the viscosity of the fluid. The particle size is obtained by measuring the density of the soil-water suspension using a hydrometer. The hydrometer test is generally performed on soil passing the No. 10 sieve.

For soils with both coarse and fine constituents, a combined analysis should be performed. The sieve analysis is performed on soil retained on the No. 200 sieve and the hydrometer analysis is performed on soil passing the No. 10 sieve.

9.4.2 Liquid and Plastic Limits

Liquid and plastic (Atterberg) limits are empirical boundaries which separate the states of fine grained soil. For example, a soil at a very high water content is in a liquid state. As the water content decreases, the soil passes the liquid limit and changes to a plastic state. As the water content decreases further, the soil passes the plastic limit and changes to a semi-solid state.

The liquid limit (LL) is defined as the water content at which a standard groove closes after 25 blows in a liquid limit device. The plastic limit (PL) is the water content at which the soil begins to crumble when rolled into 3.2 mm (0.125 in.) diameter threads. The thread should break into numerous pieces between 3.2 mm (0.125 in.) and 9.5 mm (0.375 in.) long. Refer to Appendix C for recommended test procedures for liquid and plastic limits. The purpose of the limits is to aid in the classification of fine-grained soils (silts and clays) to evaluate the uniformity of a deposit and to provide some general correlations with engineering properties.

In accordance with the Unified Soil Classification System, a fine-grained soil is classified as to its position on the plasticity chart, Figure E-4. The uniformity of a fine grained soil deposit can be evaluated by plotting the test results of natural water content and Atterberg limits versus depth or elevation.

The liquid and plastic limits are not well correlated with engineering properties that are a function of soil structure or its undisturbed state. However, some general empirical correlations for fine-grained soils have been developed based on index properties, natural water content and Atterberg limits.

9.4.2.1 Correlation of Various Properties

- *Rebound or Swelling:* According to U.S. Navy, 1971.[1]
- *Consolidation Stress versus Liquidity Index:* According to U.S. Navy, 1971.[1]
- *Coefficient of Consolidation versus Liquid Limit:* According to U.S. Navy, 1971.[1]
- *Angle of Shearing Resistance versus Plasticity Index (PI):* According to U.S. Navy, 1971.[1]

9.4.2.2 Other Controls Over Atterberg Limits. Experience has shown that the liquid and plastic limits of some fine-grained soils are sensitive to the pore fluid (salt concentration for marine illitic clays) and the pretreatment (air or oven dried or natural water content) before running the tests. It has been shown that soils sensitive to oven drying generally contain one of the following:

- organic matter
- high montmorillonite content
- hydrated halloysite
- hydrous oxides

It is recommended that limits be determined on fine grained soil starting with the soil at or near the natural water content (Lambe, 1951). Soil that has been dried (air or oven) should be thoroughly mixed with water and allowed to equilibrate for several days before testing. Soils with organic content should not be dried prior to testing.

9.4.3 Specific Gravity

The specific gravity of a soil is the ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at a temperature of four degrees Celsius. The specific gravity of a soil is used in computations for most laboratory tests. In addition, the specific gravity is often used to relate the weight of a soil to its volume of solids for use in phase relationships, such as unit weight, void ratio, moisture content, and degree of saturation.

The specific gravity is of only limited value for identification or classification of most soils because the specific gravities of most soils fall within a narrow range.

9.5 SHEAR STRENGTH

The shear strength of a soil is determined by the resistance to sliding between particles that are trying to move laterally past each other. The laboratory tests most commonly used to determine shear strength are direct shear, unconfined compression and triaxial compression.

Shear resistance of soil is due to both cohesion and friction. The shear strength of a soil is expressed by the Mohr-Coulomb failure criteria, shown graphically in Figure 9-1:

$$s = c + \bar{\sigma} \tan \phi$$

where s = shear strength
c = cohesion
$\bar{\sigma}$ = effective stress normal to the shear plane
ϕ = angle of internal friction of the soil.

Coarse-grained soils generally exhibit little or no cohesion (i.e., cohesionless) and therefore the shear strength depends primarily on the frictional resistance. An estimate of the shear strength of the cohesionless soil *in situ* can be difficult to determine in the laboratory because the strength can vary with density or critical void ratio, composition of the soil (particle size, gradation and angularity of soil particles), nonhomogeneity of the deposit and the loading conditions. Therefore, the soil should be tested in the laboratory under conditions which simulate the most critical condition in the field.

The shear strength of fine grained (cohesive) soils is a complex subject. In terms of total stress, the shear strength may be expressed as

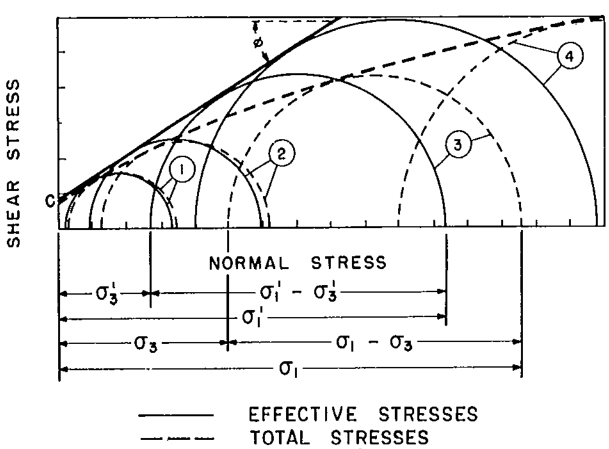

Figure 9-1. Mohr-Coulomb failure criteria. (Haley & Aldrich, Inc.)

$$s = c + (\sigma - u_f) \tan \phi$$

where u_f = pore pressure at failure

For some foundation problems, the pore pressure at failure is unknown or cannot be readily evaluated. For such problems, it is appropriate to use undrained strength (S_u, "total-stress" strength parameter) in analyses to determine the factor of safety or lateral loading, rather than "effective-stress" strength parameters, $\bar{c}$ and $\bar{\phi}$.

Experience has shown that undrained strength is independent of changes in the total stress, unless a change in water content occurs. Because undrained strength is determined by the initial conditions prior to loading, it is not necessary to determine the effective stresses that would exist at failure. The undrained shear strength of cohesive soils, as determined by laboratory tests, can be difficult to determine. Estimating strength from the results of laboratory tests ideally calls for performance of tests that will duplicate *in situ* conditions. It is very difficult to achieve this situation for many reasons; such as, effects of sample disturbance, lack of knowledge of the *in situ* stresses and equipment and testing limitations that impose non-uniform stresses or the wrong stress system.

The appropriate strength parameters for given field conditions are discussed in Section 9.5.4.

9.5.1 Loading Devices

Loading devices used to test laboratory specimens of soil can be classified as either strain-controlled or stress-controlled. Strain-controlled loading devices apply strain to the specimen at a predetermined, controlled, constant rate of strain. A stress-controlled loading device applies a constant load or stress to the specimen, generally in increments and at predetermined time intervals, by using dead weights, applied either directly or by a lever system or by using air or hydraulic pressure controlled by very precise pressure regulators.

Measurement of the load applied to a laboratory soil specimen is usually accomplished using a proving ring or an electronic load cell. Load cells and proving rings should be calibrated periodically to maintain accurate measurements.

9.5.2 Direct Shear

In a direct shear test, the soil is placed in a split shear box and stressed to failure by moving one part of the container relative to the other (Figure 9-2). The specimen is subjected to a normal force and a horizontal shear force. The normal force is kept constant throughout the test and the shear force is increased

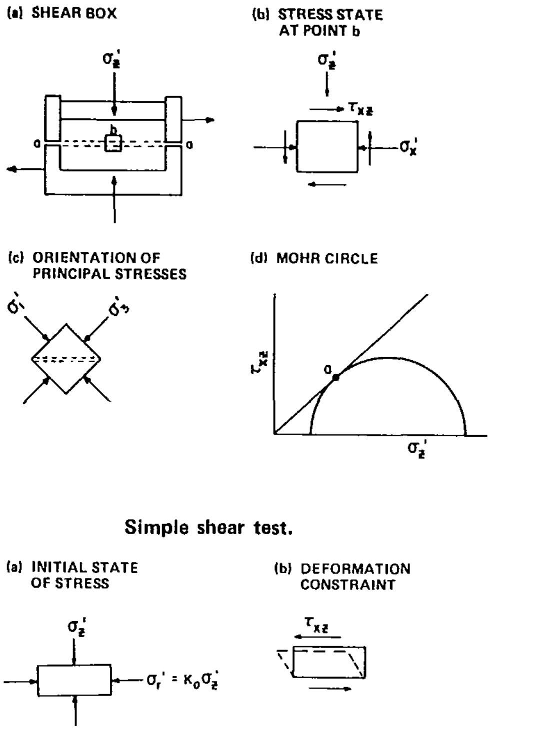

Figure 9-2. Laboratory shear tests. Above, is shown the direct shear test in which shear failure is induced along a specified plane, and its relationship to a Mohr concept for cohesionless material. Below is illustrated the simple shear test that is performed in a triaxial test cell on undisturbed samples (From Wu and Sangrey, 1978).

usually at a constant rate of strain to cause the specimen to shear along a predetermined horizontal plane.

The use of the direct shear test to determine the shear strength of soil has been questioned. In the direct shear test, only the normal and shear stresses on a single, predetermined plane are known. Hence, it is not possible to draw the Mohr Circle giving the state of stress. However, if it is assumed that the horizontal plane is equivalent to the failure plane for the soil, then the friction angle can be calculated from the results of a series of tests performed at various normal stresses. Lambe and Whitman (1969) report that comparisons between the value of ϕ, from triaxial and direct shear tests, after averaging out experimental errors in the determination of the values, yield results that differ generally by no more than two degrees.

The direct shear test offers the easiest way to measure the friction angle of a sand or other dry soil. It is not useful for testing soils containing water unless they are free draining and have a very high permeability, because it is difficult to control the drainage and thus volume changes during testing. For this reason, the direct shear tests should be used with caution in determining the undrained shear strength of cohesive soils.

9.5.3 Unconfined Compression Test

The unconfined compression test measures the compressive strength of a cylinder of cohesive soil which has no lateral confinement (unconfined). The undrained shear strength is normally taken as approximately equal to one-half the compressive strength.

The test is generally performed on an undisturbed specimen of cohesive soil at its natural water content. Cohesionless soils, such as sands and non-plastic silts and fissured or layer materials, should not be tested unconfined because the shear strength of these types of soils is a function of the *in situ* confining stress.

Because no lateral confinement is used in the unconfined compression test, it has several features:

- It is the simplest, quickest and least expensive laboratory test to measure the undrained shear strength of a cohesive soil.
- Unconfined compression tests may be performed in the field using portable equipment for rapid measurement of undrained shear strength.

Refer to Appendix D for recommended test procedures for this test.

9.5.4 Triaxial Compression Test

The triaxial test is the most common and versatile test available to determine the stress-strain properties of soil. In the triaxial compression test, a cylindrical specimen is sealed in a rubber membrane and placed in a cell and subjected to fluid pressure. A typical triaxial cell is shown in Figure 9-3. A load is applied axially to the specimen increasing the axial stress until the specimen fails. Under these conditions, the axial stress is the major principal stress, σ_1, and the intermediate and minor principal stresses, σ_2 and σ_3 respectively, are equal to the cell pressure. The increment of axial stress, $\sigma_1 - \sigma_3$, is referred to as the deviator stress or principal stress difference.

Drainage of water from the specimen is controlled by connections to the bottom cap as shown in Figure 9-3. Alternatively, pore water pressures may be measured if no drainage is allowed. Triaxial tests are generally classified as to the condition of drainage during application of the cell pressure and loading, respectively, as follows:

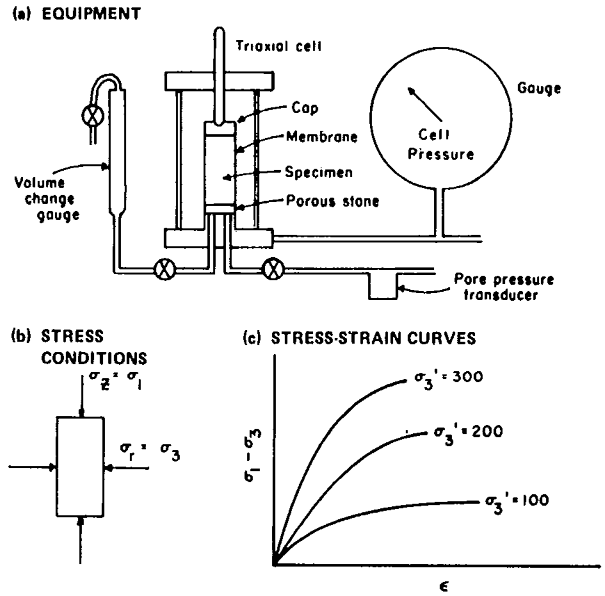

Figure 9-3. The triaxial test employed on undisturbed and remolded soil samples. A variety of *in situ* stresses and stresses related to expected structural loading conditions can be modeled into the test, showing that the soil shear strength parameters vary dramatically under different conditions of pore pressure accumulation and stress and strain levels as well as strain rates. As shown in this illustration, the deviator stress (sigma one minus three) varies considerably with the cell pressure (sigma three) utilized in the test (From Wu and Sangrey, 1978).

- *Unconsolidated-Undrained (UU).* No drainage is allowed during application of the cell pressure or confining stress and no drainage is allowed during application of the deviator stress.
- *Consolidated-Undrained (CU).* Drainage is allowed during application of the confining stress so that the specimen is fully consolidated under this stress. No drainage is permitted during application of the deviator stress.
- *Consolidated-Drained.* Drainage is permitted both during application of the confining stress and the deviator stress, such that the specimen is fully consolidated under the confining stress and no excess pore pressures are developed during testing.

9.5.4.1 Unconsolidated-Undrained (UU) Test. This test is generally performed on undisturbed saturated samples of fine grained soils (clay, silt and peat) to measure the *in situ* undrained shear strength ($\phi = 0$ analysis). For soils which exhibit peak stress-strain characteristics, the failure stress is taken as the maximum deviator stress ($\sigma_1 - \sigma_3$) measured during the test. For soils which exhibit an increasing deviator stress with strain, the failure stress is generally taken as the deviator stress at a strain equal to 20 percent. The undrained shear strength, S_u, is taken as one-half the deviator stress or $S_u = \frac{\sigma_1 - \sigma_3}{2}$.

The *in situ* undrained shear strength is applicable to conditions in which construction occurs rapidly enough so that no drainage and hence, no dissipation of excess pore pressures occur during construction. Examples of typical situations in which the *in situ* undrained shear strength would govern stability include construction of embankments on clay deposits or rapid loading of footings on clay.

Unconsolidated-undrained tests are also performed on samples of partially saturated cohesive soils. The principal application of tests on partially saturated samples is to earth-fill materials which are compacted under specified conditions of water content and density. It also applies to undisturbed samples of partially saturated (i.e. residual soils) and to samples recovered from existing fills. However, because the tests are performed on partially saturated soil, the deviator stress at failure will increase with continuing pressure. Bishop and Henkel (1962) indicate that the failure envelope expressed in items of total stress is non-linear and values of c and ϕ can be reported only for specific ranges of continuing pressures. If pore pressures are measured during the test, the failure envelope can be expressed in terms of effective stress.

9.5.4.2 Consolidated-Undrained (CU) Test. This test is performed on undisturbed samples of cohesive soil, on reconstituted specimens of cohesionless soil and, in some instances, on undisturbed samples of cohesionless soils which have developed some apparent cohesion resulting from partial drainage.

Generally, the specimen is allowed to consolidate under a confining stress of known magnitude and is then failed under undrained conditions by applying an axial load. The volume change that occurs during consolidation should be measured. The results of CU tests, in terms of total stress or undrained shear strength, must be applied with caution because of uncertainties in the effects of stress history and stress system (isotropic consolidation) on the magnitude of strength increase with consolidation.

If the pore pressure is measured during the test, the results can be expressed in terms of effective stress, $\bar{c}$ and $\bar{\phi}$.

The principal application of results of CU tests on cohesive soils is to the situation where additional load is rapidly applied to soil that has been consolidated under previous loading (shear stresses). The principal application to cohesionless soils is to evaluate the stress-strain properties as a function of effective confining stress.

9.5.4.3 Consolidated-Drained (CD) Tests. Consolidated drained tests are performed on all types of soil samples, including undisturbed, compacted and reconstituted samples.

In a standard test, the specimen is allowed to consolidate under a predetermined confining stress and the specimen is then sheared by increasing the axial load at a sufficiently slow rate to prevent development of excess pore pressure. Since the excess pore pressure is zero, the applied stresses are equal to the effective stresses and the strength parameters, $\bar{c}$ and $\bar{\phi}$, are obtained directly from the stresses at failure. The volume changes that occur during consolidation and shear should be measured.

The principal application of the results of CD tests on cohesive soils is for the case where either construction will occur at a sufficiently slow rate that no excess pore pressures will develop or sufficient time will have elapsed that all excess pore pressures will have dissipated.

The principal application to cohesionless soils is to determine the effective friction angle.

9.5.4.4 Young's Modulus. The triaxial test may be used to determine Young's modulus for a soil. The standard triaxial test, with increasing axial stress and constant continuing stress, provides a direct measure of Young's modulus. The secant modulus (drawn from zero deviator stress to 1/2 peak deviator stress on a stress-strain curve) is the modulus value generally quoted for soil.

9.5.5 Laboratory Vane Shear

The laboratory vane shear test uses a system of vanes or blades attached to a shaft that is inserted into the exposed ends of undisturbed tube samples of cohesive soil. The torque required to cause failure of the soil is related to the undrained shear strength.

It is assumed that the soil fails along the edges of the vane. Because the vane imposes a stress system during shear that is unlike any mode of failure encountered in practice, the vane test should be treated as a strength index test. That is, the vane strength must be correlated with the results of other undrained strength tests and used as an index property.

There are numerous devices on the market to perform laboratory vane tests. The most common and inexpensive types are hand operated and can be inserted into undisturbed samples of cohesive soils. The use of the vane should be restricted to homogeneous clays without shells, stones, fibers, sand pockets, and other anomalies.

9.6 CONSOLIDATION

Consolidation may be defined as volume change at "constant" load caused by transfer of total stress from excess pore pressure to effective stress as drainage occurs. When load is applied to a saturated soil mass, the load is carried partly by the mineral skeleton and partly by the pore fluid. With time, the water will be squeezed out of the soil and the soil mass will consolidate.

The permeability or rate at which the water can be squeezed out and thus the rate of consolidation, varies with the soil type. Cohesionless soils are generally quite permeable and the rate of consolidation is very rapid and generally not of a concern to foundation engineers. The permeability of cohesive soils such as clay is quite low and the rate of consolidation is quite slow. The remainder of this discussion will deal with consolidation of saturated cohesive soil, specifically clay.

When a load is applied to a saturated deposit of clay, there will be three types of settlement:

- *Initial settlement:* associated with undrained shear deformation of clay.
- *Consolidation settlement:* volume changes associated with the dissipation of excess pore pressure.
- *Secondary Compression* (consolidation): volume changes associated with essentially constant effective stress, after complete dissipation of excess pore pressure.

The relative importance of the three types of settlement depends on such factors as:

- Type and stress history of the soil, i.e., normally consolidated or overconsolidated
- Magnitude of loading
- Rate of loading
- Size of the loaded area in relation to the thickness of the clay deposit

The initial settlement of footings on heavily overconsolidated clay is often a significant portion of the total settlement.

The initial settlement of a clay deposit that is subjected to a load area very large in relation to the clay thickness (one-dimensional consolidation) will be very minor and the consolidation settlement will be of primary importance. In sand drain installations and for one-dimensional compression of organic soils, secondary compression is often of practical significance.

In general the magnitude of consolidation settlement will be of greatest concern for most cases. The laboratory test most commonly used to evaluate consolidation settlement is the oedometer test or one-dimensional consolidation test.

The stress-strain or compressibility characteristics of clays are highly dependent upon their stress history. The stress history of a clay deposit refers to the existing stresses and the degree of overconsolidation. If the vertical consolidation stress $\overline{\sigma}_{vc}$ acting on the clay is the greatest that has ever existed, the clay is called normally consolidated. If the existing stress is less than the maximum value that has ever existed, referred to as the maximum previous stress, $\overline{\sigma}_{vm}$, the clay is called overconsolidated.

If the clay is stressed within the limits of the maximum previous stress, the strain (settlement) will be a function of the recompression ratio (RR) determined from laboratory consolidation tests. If the applied stress exceeds the maximum previous stress, the strain will be proportional to the virgin compression ratio (CR).

9.6.1 Consolidation Tests

In an oedometer (consolidation) test, the soil is placed in an oedometer ring and stress is applied to the soil specimen along the vertical axis. Because strain in the horizontal direction is prevented, the vertical strain is equal to the volumetric strain.

The test is generally performed on a specimen of clay that is 19 or 25 mm (0.75 or 1.0 in.) in thickness and 64 mm (2.50 in.) in diameter. The 64 mm ring is the most common size ring because the specimen can be trimmed from a 76 mm (3-in.) thinwall tube sample.

The load applied to the specimen is generally doubled (Load Increment Ratio equal to unity) and readings of vertical deformation versus time are obtained during each load increment. The information that can be obtained from the test include:

- *Compressibility of the soil* for one-dimensional loading as defined by the compression curve, (vertical strain, ϵ_v, or void ratio, e, plotted versus log consolidation stress, $\overline{\sigma}_{vc}$).
- *Maximum previous stress,* $\overline{\sigma}_{vm}$, as determined by empirical procedures from the compression curve.
- *Coefficient of consolidation,* c_v, using curve fitting techniques, based on the Terzaghi theory of consolidation, applied to the deformation versus time curves.
- *Rate of secondary compression* as defined by the slope of the deformation versus log time plot after primary consolidation is completed.

9.6.2 Presentation of Consolidation Test Data

There are two widely used curve fitting methods that are applied to the deformation versus time curves, the log time and the square root of time method. The square root method places emphasis on the early stages of consolidation whereas the log time method emphasizes the latter stages of consolidation.

The results of consolidation tests are generally presented as a graph of void ratio, e, or vertical strain, ϵ_v, versus consolidation stress, $\overline{\sigma}_{vc}$, plotted to a log scale. This type of plot is used because it exhibits certain characteristic shapes and behavior that have proved useful.

When void ratio is used, compressibility parameters are defined as follows:

C_c = virgin compression index = slope of compression curve in virgin region.
C_r = recompression index = average slope of unloading-reloading cycle.
C_s = swelling index = slope of swelling (rebound curve).

When test results are plotted in terms of strain instead of void ratio, the corresponding parameters when strain is used are:

$CR = C_c/(1 + e_o)$ = virgin compression ratio
$RR = C_r/(1 + e_o)$ = recompression ratio
$SR = C_s/(1 + e_o)$ = swelling ratio

and strain $= \Delta e/(1 + e_o)$

The void ratio versus log stress plot is more commonly used than the strain versus log stress plot. However, the latter has several advantages (Ladd, 1971):

1. Strains are easier to compute than void ratios, which require a knowledge of specific gravity and weight of soil solids.
2. Settlements are directly proportional to strains, whereas, use of Δe data also requires a knowledge of $(1 + e_o)$. Thus, the latter introduces two variables, Δe and $(1 + e_o)$.
3. It is easier to standardize strain plots than void ratio plots.
4. The strain curve can be plotted as the consolidation test is in progress. Any major discrepancies in the test could immediately be noted and corrected, if possible.

It should be noted that the maximum previous stress for a given test will be the same, regardless of whether the results are plotted in terms of void ratio or vertical strain. The final consolidation settlement of a clay stratum for the general case consisting of both recompression and virgin compression may be expressed by,

$$\rho cf = \Sigma H \left(RR \log \frac{\overline{\sigma}_{vm}}{\overline{\sigma}_{vo}} + CR \log \frac{\overline{\sigma}_{v}}{\overline{\sigma}_{vm}}\right)$$

where

$\overline{\sigma}_{vo}$ = initial vertical stress
$\overline{\sigma}_{vf}$ = final vertical stress
$\overline{\sigma}_{vm}$ = maximum previous stress

The test procedures for consolidation testing are referenced in Appendix C.

9.7 PERMEABILITY

In general, all voids in soils are interconnected and water can flow through the densest of natural soils. Lambe (1951) describes permeability as a soil property which indicates the ease with which water will flow through the soil. A knowledge of the permeability of soil is important for solving problems associated with seepage, dewatering, drainage, and settlement. Permeability is also referred to, interchangeably, as *hydraulic conductivity.*

The behavior of fluid flow through most soil is related to Darcy's Law, which states that the rate of flow is proportional to the hydraulic gradient and area:

$$Q = k i A$$

where

Q = rate of discharge through soil
k = coefficient of permeability (hydraulic conductivity)
A = total cross-sectional area
$i = \frac{\Delta_h}{\Delta_L}$ = hydraulic gradient, which is the loss of hydraulic head per unit distance.

The permeability of a soil is influenced by the following characteristics:

- Particle size and graduation
- Void ratio
- Mineral composition
- Fabric

In general, the coefficient of permeability increases with increasing grain size but the size and shape of the void spaces also have a major influence. Small voids decrease the flow and, therefore, the permeability. A relationship between permeability and particle size is more likely to exist in silts and sands than clays, because the silts and sands are more uniform.

Soil composition and fabric component of structure have little effect on the permeability of gravel, sands and silts, but are important for fine grained soils (clay). In general, for clays, the lower the ion exchange capacity of the soil, the higher the permeability. Likewise, the more flocculated (open) the structure, the higher the permeability.

Because of the complex relationships between factors influencing permeability, it is important that laboratory tests be performed under conditions which duplicate field conditions as closely as possible. The methods most commonly used to determine permeability in the laboratory include:

- Constant head test
- Falling head test
- Direct or indirect methods during a consolidation test.

9.7.1 Constant Head Test

In general, the constant head permeability test is widely used on all types of soils. In a constant head test, a soil sample is placed in a cylindrical container (permeameter) and a constant head is applied to the sample. The amount of water passing through the specimen in a given time period is determined and the following equation is used to determine K:

$$K = \frac{\Delta Q L}{\Delta t h A}$$

where

Q = volume of water passing through specimen
L = Length of specimen
A = Cross sectional area of specimen
h = head
t = time

The recommended test procedures for constant head permeability tests are referenced in Appendix D.

9.7.2 Falling Head Test

In general, since a relatively large permeability is required to obtain good precision with a falling head test, it is limited to pervious soils. This test is performed generally in the same manner as the constant head test except that the head of water is not constant. Instead, the water head normally falls in a graduated standpipe connected to the specimen. The following equation is used to determine K:

$$K = 2.3 \frac{a L}{A(t_i - t_o)} \log_{10} \frac{h_o}{h_i}$$

where

a = cross-sectional area of standpipe
L = Length of specimen
A = cross sectional area of specimen
t_o = time at which water level in standpipe is at h_o (initial)
t_i = time at which water level in standpipe is at h_i (intermediate or final)
h_o, h_i = appropriate heads for which permeability is determined.

9.8 SWELLING AND COLLAPSE POTENTIAL

Laboratory testing of unstable soil and soft sedimentary rock differ from ordinary engineering property testing of soil (Table 9-1). This is because the appropriate, direct engineering design parameters are not attainable from laboratory testing; the mechanisms responsible for the behavior of unstable soils stem from microscale physical properties, largely related to the presence and orientation of the minus 200 sieve fraction particles. The goal of laboratory testing of unstable soil and soft rock should be to verify their potential for expansion, collapse, or catastrophic loss of strength (as in the case of sensitive clays). The conditions governing laboratory testing of the three types of unstable materials are as follows:

- expansive; the potential is related primarily to the presence and percentage of expansive clay minerals; remolding will do little to decrease their expansion potential
- collapsing; inherently unstable soil fabric will generally fail completely in one instance, at the introduction of increased moisture and increasing normal loads; remolded materials should not be collapse prone
- sensitive clays; catastrophic loss of shear strength through thixotropic rearrangement of the internal fabric is essentially a one-time phenomenon; completely remolded specimens should not experience such behavior

Soil and soft sedimentary rock that are inherently unstable should be detected by field personnel in the course of literature reviews, in preparation for field work or through recognition of physical indicators of the presence of such material. As discussed in Section 5, specific geologic formations and types of Holocene-aged surficial geologic units and soils are well known to contain representative beds or horizons that are unstable.

Subsequent laboratory testing may be required to verify the presence of these types of soils and provide a quantitative basis for prediction of the magnitude of volume change and stresses associated with such changes. Design parameters are extremely difficult to determine.

Snethen (1975) summarizes the nature of expansive soil testing, noting that laboratory tests of such materials fall into three general categories:

- soil suction (thermocouple psychrometer);
- Oedometer swell (swell pressure), and;
- empirical techniques such as Potential Vertical Rise (PVR).

9.8.1 Soil Suction (Thermocouple Psychrometer) Test

The soil suction test is performed using a thermocouple psychrometer (Figure 9-4) and small cubes of undisturbed soil placed in sealed environmental chambers. The magnitude of soil suction is measured by the psychrometer and the measurements are made on a number of similar cubes with variable moisture contents. After temperature and physical property stabilization for a 48-hour period, the psychrometer output voltage is measured as a function of the stabilizing temperature. The microvolt output is converted to soil suction in tons per square foot using a calibration relationship for the specific psychrometer. A number of data points are collected, establishing a semi-logarithmic relationship between soil suction and temperature (arithmetic).

Snethen (1975) has devised a prediction relationship in which parameters measured by the soil suction test are applied toward an estimate of the one-dimensional vertical expansion that would be expected from a stratum of similar expansive material. The procedure is referenced in Appendix D.

9.8.2 Oedometer Swell Test

The rationale of this test is to induce swelling in a soil sample and to measure the relationship between applied load and absorbed, distilled water. The water is introduced until equilibrium is reached, both in swell and water intake. The test comes in a variety of forms, all of which attempt to model the *in situ* reaction of a swelling soil to moisture imbibed over its field moisture content. The tests and the conversion to predicted volumetric swell are representative of reverse consolidation theory. Snethen (1975) has compiled a literature review of oedometer swell tests and has applied the theory to an Overburden Swell Test.

If design engineers are interested in restraining the

Table 9-1.
Direct Techniques for Quantitatively Measuring Volume Change of Expansive Soils

Method	Description
Navy method	Odometer test on remolded or undisturbed samples in which deformations under various surcharges are measured to develop a surcharge versus percent swell curve. The surcharge versus percent swell curve is related to the depth of clay versus percent swell curve from which the magnitude of volume change is calculated as the area under the curve
Potential vertical rise (PVR)	The correlation of measured volumetric swell of a triaxial specimen (all around pressure of 1 psi) with classification test data (LL, PI, SR, and percent soil binder) to determine the Family Number (predetermined correlations) for the soil. The vertical pressures at the midpoints of strata are calculated and used in conjunction with Family Curves to obtain percent volumetric swell under actual loading conditions in each strata. The linear swell is taken as one-third of the volumetric swell which is cumulatively summed to calculate the potential vertical rise
Noble method	Odometer test on statically compacted samples (total four, two initial moisture contents under two surcharge pressures) measuring deformation. Previously correlated data are consulted to determine the magnitude of volume change with changing loading and initial moisture conditions
Double odometer	Odometer test in which two adjacent undisturbed samples are subjected to differing loading conditions. One sample is inundated and allowed to swell to equilibrium, then consolidation-tested using routine procedures. The second sample is consolidated-tested using routine procedures at its natural moisture content (NMC). The virgin portion of the NMC curve is adjusted to coincide with the swell-consolidation curve, and relationships from consolidation theory are used to estimate volume change
Simple odometer	Odometer test using one undisturbed sample which is loaded to its *in situ* overburden pressure then unloaded to a seating load, inundated, and allowed to swell to equilibrium, then consolidation-tested using routine procedures. Analytical procedures are same as double odometer method
Sampson, Schuster, and Budge	Odometer test in which two undisturbed or remolded samples are subjected to different loading conditions. One sample is loaded to the testing machine capacity (32 tsf reported) and consolidated to equilibrium, inundated, unloaded to 0.1 tsf, and allowed to swell to equilibrium. The second sample is loaded to its *in situ* overburden pressure, inundated, unloaded to the planned structure load, and allowed to swell to equilibrium. The swelling index and changes in void ratio and consolidation theory are used to determine amount of volume change
Lambe and Whitman	Odometer test in which undisturbed or remolded samples are consolidation-tested using routine procedures including rebound. Effective stresses are calculated before and after testing, and the associated void ratio changes are determined. From this $\Delta e/1 + e_o$ or $\Delta H/H^*$ versus depth curves are plotted. Magnitude of volume change is equal to area under the curve
Sullivan and McClelland (constant volume swell)	Odometer test in which an undisturbed sample is loaded to its *in situ* overburden pressure, inundated, and swell pressure measured by maintaining constant volume, then unloaded to a light seating load and the swell measured. Changes in void ratio are taken from the curve corresponding to the initial and final effective stress conditions of the *in situ* soil. Consolidation theory is used to estimate volume change
Komornik, Wiseman, and Ben-Yaacob	Odometer test on undisturbed samples in which swell is measured under corresponding overburden pressures to develop depth versus percent swell curve. Magnitude of volume change is equal to area under curve
Wong and Yong	Same as previous procedure except that an additional surcharge equal to the pore water suction at hydrostatic conditions is added. Same analytical procedures
Expansion Index (Orange County)	Odometer test on compacted samples measuring volume change under 1-psi surcharge
Third cycle expansion pressure test	Used in conjunction with standard R-value test. Swelling pressure is measured at the end of the third cycle of volume change development (i.e., swell pressure is developed and relieved twice, then measured after developing the third time)

* Δe = change in void ratio; eo = initial void ratio; ΔH = change in height; H = height.
NOTE: "Odometer" as used in this table is the same instrument as "Oedometer" as shown in the text
(From FHWA RD-75-48, 1975, Federal Highway Administration, USDOT)

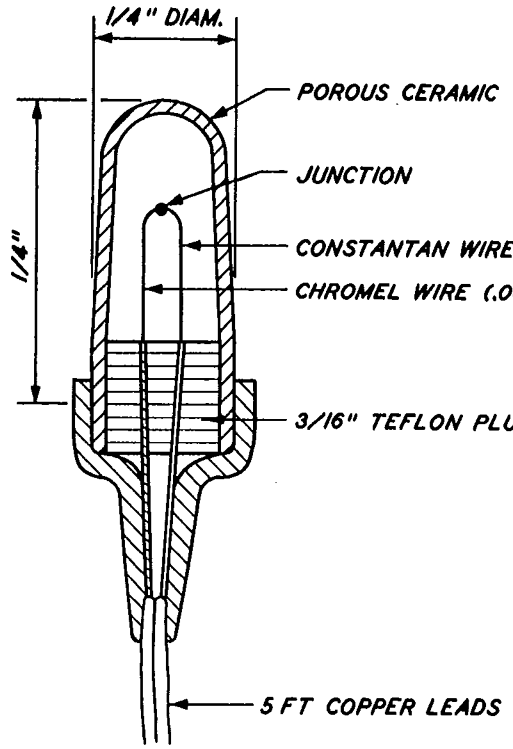

Figure 9-4. Schematic diagram of a thermocouple psychrometer.

swelling tendency of the soil through structural reinforcement, then the test may be conducted as the swell-pressure variety in which successive, increasing, oedometer loads are added to counter the swell as it develops due to moisture intake. At the equilibrium point between moisture intake and swell, the sample can be rebounded and measured for equilibrium moisture.

9.9 COMPACTION TEST

Compaction may be defined as densification at a constant water content (i.e. decreasing air voids) through the rapid application of mechanical energy. Most earth construction projects require the use of man-placed soil to which compaction must be applied to place it in a dense state. To compact a soil (i.e. to place it in a dense state) is desirable for three reasons: (1) to increase shear strength, (2) to decrease future settlements, and (3) to decrease permeability.

The purpose of a compaction test is to determine the maximum density and optimum water content values for a soil. The maximum density of a soil is the maximum dry unit weight that can be produced with a given compactive effort as the water content of the soil is varied. The optimum water content of a soil is the water content at which the greatest dry unit

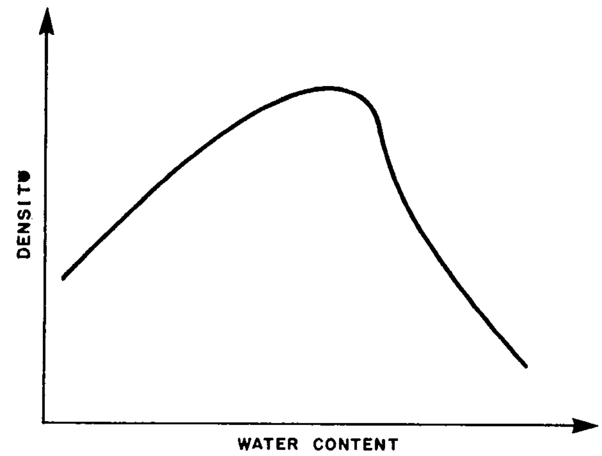

Figure 9-5. Typical compaction curve for cohesive soil.

weight is produced for a given compactive effort. The main problem in predicting compaction effectiveness is that laboratory tests do not simulate field compaction methods. Most present day laboratory compaction is by impact methods because of satisfactory correlations with field data.

For most impact methods, the sample is placed in a test mold and subjected to a specified compactive effort by a tamper of a given weight falling a given distance. In most tests, the sample is compacted in layers. The process is repeated at various water contents until the characteristic curve as shown in Figure 9-5 is developed.

Compaction of cohesionless soils, like gravelly sands and sandy gravels, is also performed by impact compaction. However, impact tests are sometimes not suitable for clean granular soils because their densities are not significantly affected by changes in water content. In free-draining cohesionless soils, the maximum dry density is sometimes obtained by vibratory methods. Present vibratory methods consist of testing cohesionless soils in a cylindrical mold with a surcharge weight, with vibration accomplished by means of a vibrating table.

9.10 LABORATORY BEARING-RATIO TEST

The California Bearing-Ratio (CBR) test was developed by the California Division of Highways in 1929 as a means of classifying the suitability of a soil for use as a subgrade or base course material in highway construction. Briefly, the test consists of causing a cylindrical plunger (cross-sectional area equal to 1935 sq. mm) to penetrate a sample of soil at a specified rate (1.3 mm per min.) and measuring the load required to cause a penetration 2.5 mm or 5.0 mm. This load is expressed as a percentage of a standard load and is known as the California Bearing-Ratio. The standard U.S. Corps of Engineers procedure requires that the sample be soaked for four days prior to testing to simulate the worst possible subgrade conditions.

9.11 DYNAMIC PROPERTIES

The response of soils to dynamic loadings has been the subject of increased attention in civil engineering practice in recent years. Some of the more common types of dynamic loadings are:

- Earthquakes
- Traffic vibrations (highway, railroad, etc.)
- Blast vibrations
- Water waves
- Machine vibrations
- Construction vibrations (e.g. pile driving vibrations)

The dynamic soil properties of interest for a particular loading depend on the amount of strain produced in the ground by the loading. In dynamics problems the strains in the soil are much smaller than those of concern in conventional static problems. Traffic and machine vibrations cause only very small shear strains in the soil (less than 10^{-4} to 10^{-3} percent). For these problems, the soil behaves essentially as a linearly elastic medium. The determination of the small strain elastic soil properties (shear modulus, Young's modulus, Poisson's ratio) is required. On the other hand, earthquake and water wave loadings may cause comparatively larger strains (as large as 10^{-3} to 10^{-1} percent) in the soil. The determination of the dynamic shear strength of the soil may be required to assess the potential for soil deformation under such loadings.

It is important to note that the laboratory test chosen to determine the dynamic soil properties properly account for the expected field conditions (i.e., the magnitude of shear strain).

9.11.1 Elastic Soil Properties

The stress-strain behavior of soil is distinctly nonlinear and largely inelastic. The use of a linear elastic soil model is justified only at shear strains less than 10^{-3} percent. At these low strain levels, the elastic soil properties are determined in the laboratory by the velocity of wave propagation in a cylinder or rod of soil:

$$G = \rho V_s^2$$
$$E = \rho V_c^2$$
$$\gamma = \frac{E}{2G} - 1$$

in which G is the shear modulus, E is Young's Modulus, γ is Poisson's ratio, ρ is the mass density of the soil, V_c is the velocity of propagation of a compressional (push-pull) wave in the soil cylinder and V_s is the velocity of propagation of a shear or torsional wave in the soil. Measurement of V_s and V_c in the laboratory provides sufficient information for the calculation of low strain elastic soil properties. It should be realized that many factors influence V_s and V_c, including magnitude of shear strain, effective mean confining pressure, soil void ratio, and degree of saturation. The laboratory test conditions must take these factors into consideration.

At strains greater than 10^{-4} to 10^{-3} percent, the determination of dynamic soil stress-strain characteristics must account for non-linear and inelastic soil behavior. In the laboratory, the large-strain soil properties are determined by repeated loading of a soil specimen and observation of the response of the soil. Factors such as the cyclic stress level, the number of cycles of applied load, the shape of the applied cyclic load (sinusoidal, rectangular, triangular, etc.) and the drainage conditions become important in these tests.

Results from large-strain tests are stress-strain curves such as those shown in Figure 9-6. Equivalent linear modulus values are defined by the endpoints of the hysteresis loop. The moduli defined in this manner are only valid for the specific test conditions (i.e., shear strain amplitude, confining pressure, etc.) used. The modulus results are often presented in the form of:

$$E \text{ or } G = K(\overline{\sigma}_o)m$$

where

E = Young's Modulus
G = Shear Modulus
$\overline{\sigma}_o$ = effective mean confining pressure
K = function of shear strain amplitude and void ratio
m = empirical constant (normally 0.33 to 0.5)

Under normal large-strain test conditions, only the shear modulus or Young's modulus can be evaluated from a single test. Accurate determination of Poisson's ratio from these tests is difficult. Values of Poisson's ratio are usually estimated based on engineering judgment.

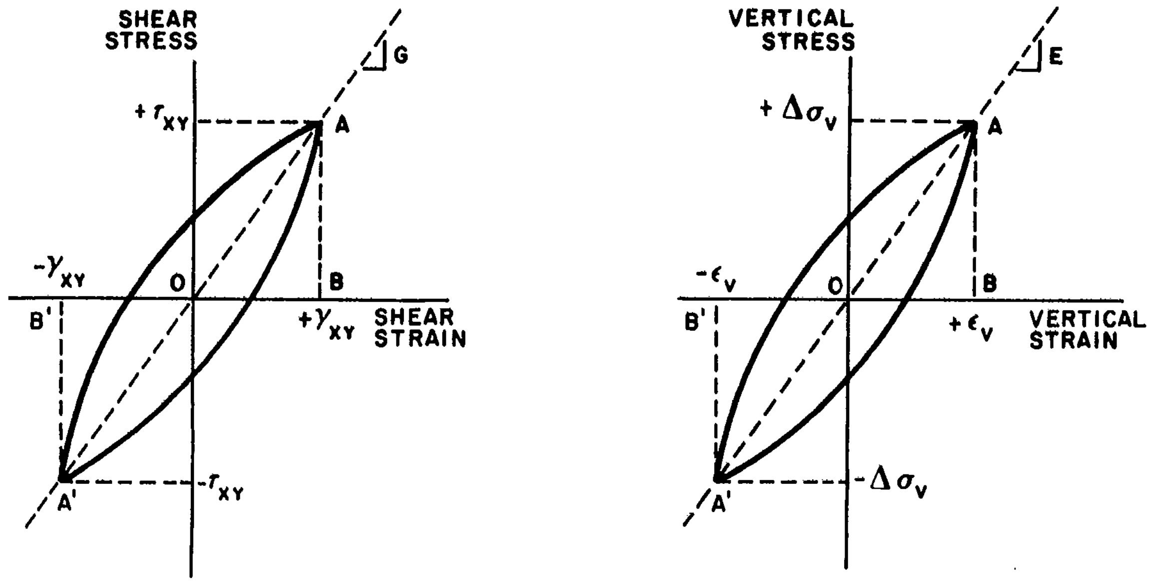

Figure 9-6. Typical stress-strain curves for large-strain cyclic tests. (Haley & Aldrich, Inc.)

9.11.2 Damping Ratio

Damping is the ability of a system or material to mute vibrations by absorbing energy. The simplest mathematical technique for modeling damping is the viscous damper or dashpot. This technique is used for convenience in the modeling of soil damping, although damping in soils is not believed to be viscous in nature.

The parameter that quantifies viscous damping is the damping ratio. A damping ratio of zero (or zero percent damping) means the system or material is undamped. Free or unforced vibrations will never be "damped" out. A damping ratio greater than or equal to 1 (or 100 percent damping) means that free vibrations cannot occur; the system is too heavily damped to allow unforced vibrations. Damping ratios between zero and 1 are typically dealt with in soil.

Damping in soil can be geometrical and/or internal. Internal damping is the capacity of the soil material itself to absorb energy and deaden free vibrations. Damping ratios associated with internal damping are in the range of 0.0 to 0.3. Geometrical damping results from the radiation of energy away from the vibration source. Damping ratios of 0.0 to 0.9 typically occur because of geometrical damping.

The damping ratio for internal damping is highly dependent on the magnitude of shear strain produced in the soil by the dynamic loading. At shear strains less than 10^{-3} percent internal damping of soil is often regarded as negligible. Geometrical damping is not considered to be shear strain dependent as it is a function of the geometry of the system and is not a property of the soil material.

At small shear strains, internal damping is determined in the laboratory by measuring the decay of free vibration in the soil. If the internal damping is modeled as viscous, the vibration decay is described by the logarithmic decrement, which is the natural logarithm of the ratio of two amplitudes of vibration:

$$\delta = \frac{1}{n} \ln\left(\frac{A_1}{A_n}\right)$$

in which δ is the log decrement, A_1 and A_n are the amplitudes of the first cycle and n-th cycle of the vibration decay, respectively. The damping ratio for internal damping can then be calculated from

$$\delta = \frac{2\pi D}{\sqrt{1 - D^2}}$$

in which D is the damping ratio. It should again be noted that for small shear strains, internal damping in the soil material may be negligible in comparison to the geometrical damping present in the soil-structure system.

At larger strains, internal damping is usually determined from the hysteresis loop in the laboratory cyclic stress-strain curve.

The basis of this calculation is that the area of the hysteresis loop represents the energy absorbed by the soil while the area of the triangles represents the potential energy at maximum stress of the loading cycle. This type of damping is called hysteretic damping, but the equation used above converts hysteretic damping to equivalent viscous damping for use in analysis.

9.11.3 Shear Strength and Pore Pressure Response

The potential for catastrophic loss of strength in soils because of porewater pressure build-up during cyclic loading (by earthquake forces) is discussed in Appendix H. In the laboratory, this phenomenon is observed by the repeated loading of a soil specimen under undrained conditions. The pore water pressure in the specimen is monitored in conjunction with the number of cycles and the magnitude of the applied loading. A plot of cyclic stress level versus number of cycles to failure is usually generated for the soil type in question. From this plot, the potential for soil failure under a given design load (e.g., a design earthquake) can be assessed.

9.11.4 Resonant Column Test

The resonant column test is the most widely used laboratory procedure for assessing the small strain properties (elastic moduli; damping ratio) of soil.

Through the use of plots such as Figure 9-7, the resonant column test may even be used to estimate large-strain soil properties, once the small-strain properties are known.

In this test, a column or cylinder of soil is vibrated at various frequencies until the resonant frequency of the specimen (the frequency that causes maximum end motion of the cylinder) is determined. From this resonant frequency, in combination with the test system geometry, the velocity of wave propagation in the soil can be determined. The test is performed in a triaxial test chamber so that the effect of confining pressure can be assessed. If the specimen is vibrated axially or longitudinally, the rod compression wave velocity (V_c) is determined. If the specimen is vibrated torsionally by torquing one end of the specimen, the shear wave velocity (V_s) is determined. The calculation of the wave propagation velocities depends on the details of each particular resonant column device. The torsional resonant column is currently the most commonly used form of the test.

One way the damping ratio may be determined is by shutting off the driving mechanism of the device while the specimen is vibrating at the resonant frequency. The decay of the vibration is measured and the damping ratio is calculated from the log decrement, as previously described.

A test procedure for the performance of resonant column tests has been proposed by Drnevich, Hardin and Shippy (1978). This procedure has been generalized to account for the wide variety of resonant column devices currently in use. This recommended test procedure for resonant column tests is referenced in Appendix D.

A study was performed in the mid-1970's (Skoglund, Marcuson and Cunny, 1976) to compare the results obtained from a number of different resonant column devices. The most important factor influencing results was found to be the specimen prepara-

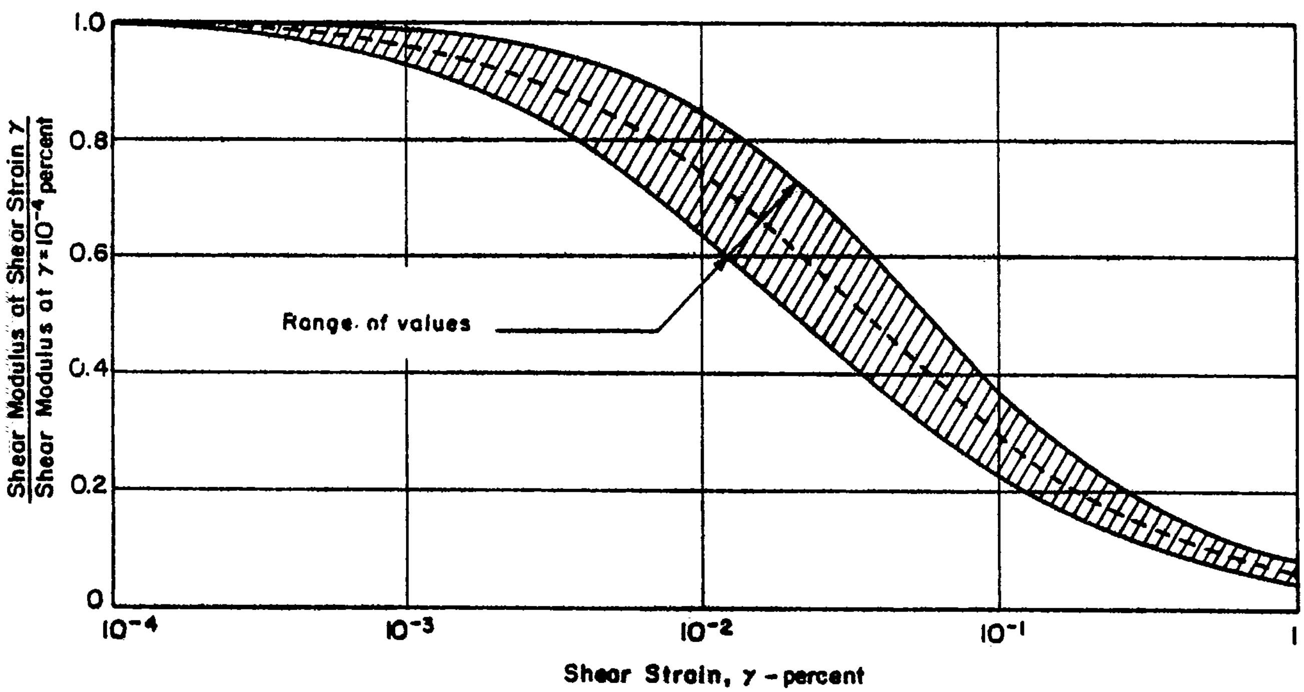

Figure 9-7. Typical plot showing variation of shear modulus with shear strain.

tion technique. This concept may be extended to include sample disturbance in "undisturbed" specimens. The sample used in the laboratory must represent the field condition of the soil as closely as possible to obtain meaningful results.

9.11.5 Cyclic Triaxial Test

The most common laboratory test for the determination of shear-strain S-S (10^{-2}) soil properties is the cyclic triaxial test. The test is basically the same as the triaxial compression test described in Section 9.5.4, except that a provision is made for applying a cyclically varying axial load. The test is used for two basic reasons:

- To assess the potential for catastrophic loss of soil strength under cyclic loading.
- To evaluate the cyclic stress-strain characteristics and damping ratio of the soil under large strain cyclic loading.

Tests to evaluate loss of soil strength are performed on saturated soil samples under undrained cyclic loading. These tests are stress-controlled; the amount of cyclically varying load is kept constant throughout the test. The resultant strains are measured independently.

In the evaluation of the damping ratio, the cyclic triaxial test may be performed on saturated, partially saturated, or dry samples under drained or undrained conditions. The tests are performed with strain control: the amount of axial strain produced by each cycle is kept constant. The axial load is varied to maintain constant cyclic axial strain.

The cyclic triaxial test has been the subject of great controversy in the last decade. The test has been criticized as to how well it models actual field conditions and as to the severe stress gradients to which the soil specimen is subjected. Numerous correction factors have been proposed for cyclic triaxial tests. These have been summarized by Seed (1976). It is important to note that the validity of this test has not been verified by field observations. Great care must, therefore, be taken in the interpretation of the results of the cyclic triaxial test.

A large number of factors have been found to influence the results of the cyclic triaxial test. These have been summarized by Townsend (1978) and they include:

- Method of Specimen Preparation
- Loading Frequencies
- Specimen Size
- Wave Form of the Loading
- Sample Disturbance

These factors should be recognized during testing and accounted for in interpreting the results.

Recommended test procedures for cyclic triaxial tests are referenced in Appendix D.

9.11.6 Other Dynamic Tests

Several types of dynamic laboratory tests other than those already described are in use throughout the world. These include:

- Pulse Tests
- Cyclic Simple Shear Tests
- Cyclic Torsional Shear Tests

These tests are not as widely used as the resonant column and cyclic triaxial tests, and so will be described only briefly.

9.11.6.1 Pulse Tests. These tests are used to determine small strain elastic soil properties. The travel time for shear and/or compressive waves to travel the length of a soil specimen is measured.

Wave propagation velocities (from which elastic soil properties may be determined) are calculated by dividing the specimen length by the measured travel time. The difficulty with this test is the accurate determination of the travel time. Small errors in travel time measurement may lead to large errors in the calculation of elastic properties.

9.11.6.2 Cyclic Simple Shear Tests. For this test, a soil specimen is subjected to a cyclically varying horizontal shearing force on its top face, while its bottom face is held fixed. The specimen is normally encased in a membrane and subjected to a confining pressure. Shear stress and shear strain can be directly measured from this test. Large strain stress-strain behavior (only strains of 0.01 percent or greater can be accurately measured) as well as the potential for pore pressure build-up in soil samples may be assessed. The test is believed to model actual field loading conditions better than the cyclic triaxial test.

However, as with the cyclic triaxial test, a relatively small soil sample is subjected to severe stress gradients during the test, so the accuracy of the results is uncertain.

9.11.6.3 Cyclic Torsional Shear Tests. In this test, a cylindrical soil specimen is loaded by a cyclically varying torque on its top face. The bottom face is held fixed. Since the torsional strain in such tests on soil cylinders varies from zero at the center to a maximum value at the outer edge, hollow cylindrical specimens are normally used. These hollow specimens essentially have a uniform torsional strain distribution across the specimen thickness.

The primary advantage of this test is that the test may be performed at practically any level of cyclic strain. Small strain or large strain soil properties can be assessed using only one test apparatus. The main disadvantage lies in the preparation of the hollow cylindrical specimen. Preparation of "undisturbed" or hollow cylinder soil specimens, particularly for cohesionless soils, is extremely difficult.

9.11.7 Summary

It should be recognized that some of the laboratory test procedures and equipment are not standardized, and interpretation of test results requires thorough knowledge of the test details. New information on techniques and apparatus continues to be published in professional journals and a detailed investigation should consider these sources.

Finally, laboratory testing is not the only way to establish dynamic soil properties. Testing in the field to determine small-strain dynamic soil properties is common and provides the advantage of measuring the properties in their natural environment under natural field stress conditions. Small-strain laboratory testing may be preferred for economic reasons or for investigating soil behavior under conditions other than those *in situ*. Large-strain dynamic testing in the field is generally not feasible nor possible. Some field tests do provide information on dynamic soil behavior, but direct use of simulated earthquake or other larger strain dynamic loadings is rarely done.

9.12 LABORATORY TESTS OF ROCK

Experience has shown that laboratory testing of rock has very limited applicability for measuring significant rock properties. Significant rock properties are defined as those that are of concern in rock design such as:

- Compressive Strength,
- Shear Strength,
- Hardness,
- Compressibility, and
- Permeability.

The basic problem is that rock samples small enough to be tested in the laboratory are usually not representative of the entire rock mass. The best example of this is rock permeability. No laboratory test would be able to approximate the permeability of a rock mass that is primarily related to the number, type and continuity of fractures in the mass.

Hence, laboratory testing of rock is used primarily for classification of intact rock samples, and, if performed properly, serves a useful function in this regard. Rock engineering design is largely a blend of theory, engineering property values and individual experience with the various rock types. Characteristic rock engineering problems relate to strength, compressibility and permeability. Laboratory tests on intact samples may provide upper bounds on strength and lower bounds on compressibility. Frequently, laboratory tests can be used in conjunction with field tests to give reasonable estimates of rock mass behavioral characteristics.

Given below is a listing of some laboratory tests for rock. Rock tests are referenced in Appendix C.

- Microscopic Petrographic Analysis
- Density (Unit Weight)
- Water Content
- Porosity
- Absorption
- Permeability
- Uniaxial Strength in Unconfined or Confined Compression (May be conducted with strain measurement for the purpose of measuring deformation properties such as Modules and Poisson's ratio.)
- Direct Tensile Strength
- Point Load Test
- Hardness
- Taber Abrasion Test
- Los Angeles Abrasion Test
- Swelling
- Slake-Durability
- Sonic Velocity (May be conducted at various stress levels.)
- Direct Shear Strength
- Torsional Shear Strength
- Various Dynamic Properties
- Various Non-Mechanical Properties such as Electrical, Magnetic and Thermal Properties.

Ideally, a few simple index tests can be selected from this list to provide reasonable estimates of significant rock properties for a particular project. If testing is not possible, microscopic petrographic analyses should be conducted as a minimum in order to obtain dependable geologic rock classifications. If only a small rock testing budget is available rock tests such as density, point load test, and hardness could be used.

For engineering purposes, rock density is most commonly determined simply by weighing the sample and determining its volume by water displacement. Point load testing is relatively inexpensive and has been shown to be related to uniaxial compressive strength in unconfined compression. In general, the uniaxial compressive strength is approximately 25 times the point load strength.

Two common and relatively inexpensive methods of hardness testing are the Shore Scleroscope and the Schmidt L-Type Hammer. These hardness tests have been shown to be a good measure of rock rebound and impact hardness and to be related to other rock properties such as the uniaxial compressive strength in unconfined compression and the tangent modulus of elasticity at 50 percent of the ultimate unconfined compressive strength.

If a larger rock testing budget is available, unconfined compressive strength, abrasion hardness and tangent modulus of elasticity at 50 percent of the ultimate compressive strength can be measured directly. Direct measurement of these properties is recommended for larger projects. Occasionally, some of the other properties given earlier in the section will be measured either as index tests or as a direct measure of a significant engineering property. Some of the more commonly conducted tests are uniaxial strength in confined compression, sonic velocity, slake-durability and direct shear strength. A description of these test procedures and the criteria for when they should be used is beyond the scope of this manual.

9.13 USE OF STANDARDS

Wherever possible, the laboratory testing procedures followed should be based on the standard specifications of AASHTO and/or the American Society for Testing and Materials (ASTM). There are no standards for some tests, but in these cases, commonly accepted procedures should be followed.

Deviations from standards or accepted procedures may be necessary on occasion, based on the judgement of testing or design engineers, their experience with local soils and peculiarities of a project, procedure, or test equipment. In order to insure that the test methods remain compatible with the purpose of the tests and that the results will be acceptable, every such deviation should be discussed in advance with the engineer assigning the tests. Also, a description of any non-conventional procedure should accompany the test data.

9.14 RECORD KEEPING

All laboratory test data forms should be filled out accurately and completely. In general, the originals of all laboratory test data, when completed and checked, should be sent to the appropriate engineer for use and filing. The laboratory testing file should contain as a minimum, the following records:

- Sample receipt logs.
- Sample disposition logs.
- Soil test assignment sheets.
- Laboratory test data.
- Calculations.

Well planned data sheets can improve the efficiency of testing and, by encouraging the recording of data which otherwise might be lost, can lead to better testing. Each laboratory should adopt whatever data sheets are most suitable for their practice and apparatus.

9.15 PRESENTATION OF DATA

The results of laboratory tests are generally presented in tabular and/or graphical form. Any anomalous behavior noted during the test or deviations from test procedures should be noted on the results.

Graphs should normally show all the plotted points, not just smooth curves, and be given scales, as large as possible, in easily read units such as 1, 2, or 5 divisions per unit. When the results of several tests are shown on a single graph, a legend should be used to identify the data plotted from the different tests, a title block should be shown on each graph which includes:

- Title of project.
- File or project number.
- Date of work.
- Type of soil.
- Scale (if appropriate).
- Boring/sample number and depth or elevation.
- Other pertinent data that identifies the test specimen.

Triaxial test results are normally plotted as stress-strain curves. Pore pressure, effective stress or stress path plots should be included as appropriate. Effective consolidation stress versus void ratio or vertical strain is normally plotted for consolidation tests. The results of classification and index tests are normally presented in tabular form. It is often useful to present the results of tests on cohesive soils in the form of a plot of Atterberg limits, compressibility and stress history, and strength data versus depth or elevation. A generalized soil profile can also be shown to indicate the variation, or lack of, in soil properties with soil type and depth.

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10.0 COMPILATION AND PRESENTATION OF GEOTECHNICAL INFORMATION

The various items of geotechnical information that are gathered and developed for a project can be generally classified as *factual* or *interpretive,* depending on their basis or origin and, to some extent, the type of information. Some items are clearly either factual or interpretive; others may be classified by agency policy, or possibly by legal definition.

10.1 TYPES OF INFORMATION

10.1.1 Factual Information or Data

Factual information or data are the actual results of individual remote sensing surveys, subsurface explorations, field or laboratory tests, and observation well or instrumentation monitoring. It is information that has been obtained by standard or recognized techniques, presumably correctly and accurately, and has not been extended or modified by geologic or engineering judgment or interpretation.

Factual information or data may be used or interpreted by qualified personnel with reasonable confidence that it represents an actual condition that existed at a specific location at a specific time. There is no presentation of the meaning of the facts or data, nor is there any assurance that different or additional data could not be obtained at a different location or time, or by a different method.

In some situations, the classification of data may also be applied to pre-existing information in the form of published geologic reports or maps, records of previous test borings on a site or route, or other similar reference material. The extent to which pre-existing information is factual may not be known; it should be clearly identified as to its source and apparent or possible limitations. Pre-existing information is subject to interpretation by qualified personnel, with the understanding that all or part may be neither factual nor applicable to the current project.

10.1.2 Interpretive Data

Investigations and evaluations for a particular project will interpret factual information or data to develop further geotechnical information for project planning and construction. Interpretive information involves professional judgement; it represents the opinion of qualified personnel as to the meaning of data, and presents geologic or engineering conclusions or recommendations based in part on the factual information and data. Interpretive information is prepared for the particular project, and is not necessarily applicable to another project on the same site or route.

10.2 USES OF INFORMATION

The geotechnical information that is compiled for a particular project is primarily intended for use during project planning and design. However, there may also be some use of the same information by the Engineer or Contractor during construction.

Ordinarily the initial use of preliminary information on a design project will be for route or site selection, environmental impact assessment and planning design-phase subsurface investigations. Preliminary geotechnical information may be based entirely on the interpretation of pre-existing data, but it will usually include some mapping or explorations that have been carried out for the particular project.

Most of the project geotechnical information will be developed and used during the design phase. To the extent possible the necessary data will have been obtained early in the design phase, so that the data and its interpretation can be available throughout detailed project planning. Agency policy or procedures may call for pilot subsurface explorations at the start of design, or permit the accomplishment of design-phase explorations in two stages, so that both early design studies and the final project configuration are based on adequate information.

The geotechnical information that is developed during design-phase investigations may have a variety of uses, including input to the following:

- Comparisons of the feasibility or cost of alternative foundation or structure types, or alignments.
- Development of specific design recommendations and specification requirements for geotechnical aspects of the project.
- Estimates of soil, rock and foundation quantities for estimating costs and bidding the project.
- Backup information for the as-designed project so that construction-phase personnel will be aware of the reasons for the design and contract provisions.
- Description of anticipated subsurface conditions to provide basis for bidding and definition of changes, particularly if project is to be bid as a lump sum.

For some agencies the last of these purposes may be unintended and contrary to agency policy. Past practice in contracting for the construction of transportation projects has commonly been to place as much as possible of the responsibility for determining and contending with subsurface conditions on the Contractor. Boring information has sometimes not been provided in the contract documents, or more often, has been made available with a disclaimer as to its accuracy. There have also been contract provisions to the effect that bidders should make their own subsurface explorations. The intent of these practices has been to avoid claims for "changed" subsurface conditions that would increase project cost.

Courts have not upheld contract provisions that intend to have the Contractor take all of the risk for unknown or changed subsurface conditions. Furthermore, there have been added costs to agencies when prudent Contractors have included substantial contingencies in their bids, to allow for unknowns, or when there have been delays and disputes due to claims when imprudent Contractors have encountered unexpected subsurface conditions.

For other types of projects, such as the investigation and repair of deterioration or damage, or a feasibility or cost study, there may be more or fewer geotechnical considerations than for a typical design project. However, the geotechnical information that is developed will still have basic application to whatever engineering analyses and evaluation are carried out.

10.3 PRESENTATION OF FACTUAL INFORMATION OR DATA

Preceding sections of this manual have presented comprehensive listings of methods for obtaining various types of geotechnical data, and have detailed both the procedures and the data to be obtained. The following subsections of this section include selected listings of frequently-used factual information or data. They are not necessarily either complete or in the order in which the information will be obtained; rather, they are intended to demonstrate types of data and more commonly used items.

10.3.1 Pre-Existing Data

To the extent possible or appropriate the information listed in this section will be compiled and evaluated prior to the commencement of actual subsurface exploration for a project. Some of it may be located or obtained at the time of the initial site or route reconnaissance. Pre-existing data will ordinarily form the basis for planning project geotechnical investigations. It can be added to as more pre-existing information becomes known during the course of the project.

- Topographic maps of site or route (U.S. Geological Survey quadrangle maps, state or municipal photogrammetric maps, etc.)
- Geologic maps of site or route (U.S. Geological Survey or state quadrangle maps showing bedrock or surficial geology)
- Soil survey maps and reports (U.S. Department of Agriculture, Soil Conservation Service)
- Water supply papers and maps (U.S. Geological Survey or state agencies)
- Aerial photographs (State or municipal agencies)
- Climatological data (U.S. Department of Commerce)
- Published geologic or engineering reports applicable to project or area. (Federal, state, municipal or private)
- Available results of previous test borings or other subsurface explorations, including drilled wells, on or near site or route (State, municipal or private)
- Statewide data-bank compilations of geotechnical properties and conditions related to specific geologic units, as observed by various DOTs
- Records of construction or performance of foundations or other engineered works on or near site or route (State, municipal or private)
- Records of past stability, settlement or other

geotechnical problems on or near site or route (State, municipal or private)

10.3.2 Remote Sensing

This information is commonly obtained by means of airborne equipment as an early planning supplement to or substitute for pre-existing data. The below-listed types of remote sensing provide information that usually requires interpretation to be useful to geotechnical personnel:

- Low or high-altitude black and white or color photography (high-altitude imagery is commonly available through the EROS Data Center of the U.S. Geological Survey, Rapid City, SD)
- Thermal infrared and side-looking airborne radar (SLAR)
- LANDSAT (satellite) and manned space mission imagery (EROS Data Center)

For a more detailed discussion on remote sensing, refer to Section 5.5, Remote Sensing.

10.3.3 Geophysical

The data source for this information can range from airborne to ground-surface surveys, including sensing in individual boreholes. Geophysical information, such as that listed below, requires interpretation for geotechnical use, and generally provides relative rather than absolute geologic information as well as indications of average engineering property data.

- Seismic refraction or reflection surveys
- Resistivity or conductivity surveys
- Electromagnetic, magnetic or gravimetric surveys
- Borehole logging and seismic velocity measurements

10.3.4 Subsurface Explorations

Most design-phase geotechnical information is obtained by subsurface exploration, with drive-sample test borings being the primary source of data in many areas. The listing that follows includes only the basic types of data that commonly result from subsurface explorations; on any individual project there may be a variety of methods of exploration that seek to obtain other items of specific information pertaining to that particular project.

- Logs of test borings, with disturbed and "undisturbed" soil samples, and rock cores
- Logs of test pits and test trenches, with disturbed samples
- Records of observation well installation and monitoring
- Records of hand-rod, drive-rod or percussion drill probings or soundings

10.3.5 Field Testing

In many cases *in situ* testing of soil or rock will provide better or more accurate information than can be obtained by laboratory testing of disturbed or undisturbed samples. The information provided by some of the below-listed tests can be directly applicable to design, to the extent that field conditions and test procedures are representative of the design condition; other field tests require interpretation of the data.

- Vane shear
- Cone penetrometer
- Pressuremeter or borehole shear device
- Plate bearing
- Field CBR
- Borehole permeability
- Water pressure
- Pumping
- Percolation

10.3.6 Laboratory Testing

Laboratory testing also provides a basic component of design-phase geotechnical information, commonly utilizing disturbed and "undisturbed" samples of soil and rock from test borings and test pits. Data from the following categories of tests are usually directly applicable to design.

- Soil classification and physical property tests, including grain size distribution, Atterberg limits, water content, organic content, unit weight and specific gravity
- Soil performance tests, including unconfined and triaxial compression, shear, consolidation, permeability, CBR and compaction
- Soil dynamic property tests, including triaxial and resonant column
- Rock property tests, including hardness, abrasion resistance and compression

10.3.7 Construction-Phase Testing and Monitoring

Most of the soil information that is developed during construction will be for quality control purposes, but there can be geotechnical investigations in connection with the confirmation of design analyses or the resolution of problems. The field data can be in one or more of the following categories.

- Results of subsurface explorations, including borings and test pits;
- Results of soil tests, such as gradation, water content and unit weight;
- Results of load tests, such as pile or plate bearing tests;
- Records of monitoring of instrumentation, including observation wells, settlement measuring devices, inclinometers, extensometers, strain or pressure gages, and vibration monitoring equipment, and;
- Rock structure mapping.

10.4 PRESENTATION OF INTERPRETIVE INFORMATION

Factual information or data, in conjunction with geologic or engineering judgement and analyses, forms the basis for interpretive geotechnical information. As previously noted, there can be agency or legal differences of opinion concerning the dividing line between fact and interpretation. The items of interpretive information that are presented in this section are considered to require professional judgement or interpretation that extends beyond the simple application of standardized techniques or procedures.

10.4.1 Design or Analytical Considerations

The interpretation of data will normally have as its objective the development of design or construction recommendations. Feasibility will usually be the initial engineering consideration—whether or not a particular design and construction approach can accomplish the project objective.

After feasibility has been established, the primary design consideration will be safety, such that the project can be expected to perform satisfactorily for its design life without foreseeable unacceptable danger to people or property.

It is these considerations of feasibility and safety that necessitate the use of factors of safety, to provide a design margin that will allow for uncertainty as to data reliability, analysis applicability, loading conditions or use, and material or construction quality. Cost is an opposing consideration that mitigates against over-conservatism in the selection of factors of safety. Project budgets will always have finite limits, and the cost of an increased factor of safety has to be weighed against the possible use of the same funds toward other objectives.

Many aspects of project design are subject to factors of safety that have been set on the basis of long experience. Geotechnical engineering analyses commonly also utilize factors of safety that have been predetermined by engineering practice or agency policy. However, the interpretation and use of geotechnical data must consider the reliability of that data and the certainty of the interpretation with respect to the feasibility, safety and cost of the project. These considerations may dictate departures from usual factors of safety, and should be made known to those who have design responsibility.

10.4.2 Geologic Interpretation

This is often a process of refinement throughout planning, design and construction. Pre-existing data and field reconnaissance may be the original basis for geologic maps. Interpretation of the data by qualified geologists can increase the reliability of information that necessarily has to be inferred between points or locations of factual data, but it cannot provide certainty as to subsurface conditions.

Geologic interpretation can extend beyond the development of soil and rock maps and profiles, to the detailed evaluation of specific aspects or items that will have particular engineering impact on a project. Such evaluations could include rock structure for tunnel projects, aquifer delineation for dewatering considerations, and clay stratum history for the prediction of settlement.

10.4.3 Design Evaluation and Recommendations

As has been stated, the geotechnical engineering interpretation for most projects will be directed toward the development of design recommendations. Geologic interpretation will be extended to develop design subsurface conditions and engineering properties for soil and rock. This interpretation will include consideration of the reliability and applicability of the available data, and the effect that inaccuracies may have on geotechnical design evaluations.

As part of the design process there will then be evaluation of the relationship between established project requirements, such as structure geometry and loads, and the interpreted subsurface conditions. This general evaluation will often be a basis for consideration of alternative design approaches for the various geotechnical aspects of the project. The comparison of alternatives will include consideration of geotechnical feasibility and cost, but the final choice will usually also have to reflect compatibility with project requirements based on other concerns, such as environmental impact, structural design, available right-of-way and schedule limitations.

Specific design recommendations are the primary component of geotechnical interpretive information. These recommendations should address both the engineering aspects of design and the construction considerations that relate to implementation of the recommendations. The following subsections present a listing of items that should normally be considered in developing design recommendations for transportation-related projects. There will certainly be variation between projects; some will involve items that are not listed and others will involve only selected items.

10.4.3.1 Structures

Type of Foundation Support General Considerations

- Loading conditions—vertical and horizontal, static and dynamic and various combinations.
- Settlement—requirements of structure vs. soil conditions.
- Effects on adjacent construction—for example, excavation or dewatering limitations.
- Scour depth—for foundation protection.

Footings

- Elevation of footing—for structure geometric requirements or subsurface conditions.
- Allowable bearing pressure—for bearing capacity and for settlement; considering soil or rock, adjacent foundations, water table, and other factors.
- Material on which footing is to be placed—excavation requirements, possible removal of unsuitable material, use of structural fill, and dewatering.
- Estimated settlement—footings on earth.
- Resistance to sliding—footings on earth.

Piles

- Method of support—friction or end-bearing, in rock or soil or both; settlement potential.
- Suitable pile type or types—reasons for choice and/or exclusion of types.
- Pile tip elevations.
- Estimated—average values, with range of variation if desirable.
- Specified—explain reasons, such as driving through fill, negative skin friction, scour, underlying soft layers, piles uneconomically long, etc.
- Allowable pile loading—for pile type, structure load, method of support, and soil or rock conditions.
- Pile driving requirements—hammer, tolerances, etc.
- Cut-off elevations—water table, marine borer problems, etc.
- Test piles required—location for maximum utility.
- Load tests required and use of dynamic pile driving formula—confirmation of capacity.
- Corrosion effects of various soils and waters, and possibility of galvanic reaction—protective provisions.
- Potential for obstructions—pre-augering or tip reinforcement.
- Lateral forces—batter piles or lateral soil resistance.
- Unusual conditions—difficult driving, tremie seals, etc.

Other Foundation Types

- Type and Method of support—caisson, drilled pier, etc., to soil or rock; reason for selection.
- Allowable capacity and support elevation—design and construction criteria.
- Tests required, and other considerations as for "Piles."

Approach Fill Considerations

- Settlement—may require surcharging or time delay, additional bridge spans, negative skin friction load on piles.
- Stability—may require stage construction, height limitations, berms, removal of unsuitable material, use of select fill materials.

Wall Considerations

- Lateral forces—wall type, earth pressure or resistance, surcharge or dynamic loads.
- Backfill—material or placement limitations.
- Drainage—waterproofing, underdrains or weepholes.

10.4.3.2 Cuts and Fills

Excavation Considerations

- Materials—classification, rock profile and rippability, boulder content, use or disposition, shrinkage or swell during excavation.

- Procedures—equipment, time, or sequence limitations or requirements.
- Groundwater—levels and seepage, probable variations, control during excavation, permanent control by underdrains.
- Slope stability—soil, rock and water conditions in problem areas, construction-phase and long-term stability, permanent slopes and slope treatment, excavation limitations, static and dynamic conditions.

Embankment Considerations

- Foundation—unsuitable materials or sidehill slopes, stability, stabilization or removal, seepage or groundwater problems, settlement.
- Procedures—equipment, time or sequence limitations or requirements.
- Filling—available materials or borrow requirements, placement and compaction, slopes and slope protection.

10.4.3.3 Pavements or Roadbeds

Subgrade

- Quality—unsuitable materials, variability, frost susceptibility, expansive or corrosive soil conditions.
- Support capacity—wheel or truck loads, traffic intensity, dynamic conditions, bearing capacity, improvement.

Pavement or Roadbed Section

- Material—surface type, base and subbase availability and quality, processing
- Thickness—design requirements, maintenance

Drainage

- Groundwater control—levels and variations, pavement or roadbed and subgrade protection, underdrains.
- Soil movement—erosion potection, filter materials or fabrics.

10.4.3.4 Tunnels or Underground Structures

In Soil

- Design considerations—opening size and shape, cover depth, soil characteristics, lining, groundwater conditions.
- Constructability—excavation and face stability, shield or pressurization, rate of advance, temporary support, adjacent structures and underpinning, access and material disposal, machine feasibility.
- Groundwater control—waterproofing or permanent drainage.

In Rock

- Design considerations—opening size and shape, cover depth, rock geologic characteristics and physical properties, lining, groundwater conditions.
- Constructibility—face or heading geometry, rate of advance, rock support and bolts, rock performance and overbreak, blasting limitations, access and material.
- Groundwater control—during construction and permanent drainage.

10.4.3.5 Construction Considerations

- Potential problems—data limitations, unexpected subsurface conditions, quantity estimates, contractor performance, adverse weather.
- Environmental concerns—noise or vibration, runoff and erosion, dust or siltation.
- Excavation—control of earth and rock slopes including shoring, sheeting, bracing, and special procedures, variation in type of material encountered, slope and bottom stability.
- Groundwater—fluctuations, control in excavations, pumping, drawdown.
- Adjacent structures—protection against damage from excavation, pile driving, drainage.
- Quality control—monitoring, sampling, laboratory and field testing, project performance.

10.4.3.6 Instrumentation

- Soil, rock or structure movement—settlement platforms and points, inclinometers, tiltmeters, extensometers, heave stakes, optical survey.
- Groundwater level or movement—observation wells or piezometers.
- Soil, rock or bearing pressure strain gages, pressure cells or plates.
- Rock or structure vibration—seismograph.

10.5 GEOTECHNICAL REPORT PRESENTATION

As indicated in section 10.3, there is variation among agency policies with respect to the dissemination of geotechnical information to contractors. In some cases it has been deliberately withheld, as being for design purposes only, and in other cases it has been formally included in the contract documents, with the recent trend being toward the latter. Court decisions have influenced the trend.

10.5.1 Contractual/Legal Implications

It is generally considered desirable and prudent to make all pertinent geotechnical data available to bidders, and to require contractor acknowledgment of the availability, either in writing or by the inclusion in the documents. There should be appropriate contract clauses clearly stating the limitations and applicability of the data that is made available. It is also desirable to make pertinent interpretive information available to bidding contractors to clarify geotechnical aspects of the project and provide a uniform basis for bidding. However, there is less agency acceptance of a policy of disseminating interpretive information, particularly if it is to be included in contract documents, and there is a greater need for clear contract stipulations as to the purpose of the information and the obligation of the contractor to draw his own conclusions.

One increasing contractor use of available geotechnical information, or the absence of such information, is in the area of "changed" conditions claims. To some extent this can be avoided by the use of unit price contracts, or by the use of base geotechnical data or its interpretation as the basis for adjustments in payments to the contractor. These approaches place part of the risk of added costs due to unexpected subsurface conditions on the agency, but they have the benefit of reducing contingency allowances in the bids of responsible contractors.

Whatever provisions there may be with respect to the relationship between geotechnical information and the contract documents, much of the geotechnical information that is generated during planning and design is not ordinarily useful or pertinent during construction. Such information may relate to superseded alignments or locations, technical or economic comparisons of design alternatives, details that are not adopted, or items deleted from the project scope. These items do not benefit the Resident Engineer or a responsible contractor; however, they may become the basis of claims by some contractors. Thus there is added reason to exclude non-pertinent information from that which is available to contractors. The decision as to what is pertinent and what is not can be on a project by project basis, or can be established by agency policy; it is also subject to legal interpretation.

The potential for claims and legal questions relative to geotechnical information makes it particularly important that all data be accurate, complete, and properly labelled, as to applicability and limitations, complete and checked.

10.5.2 Informal Planning and Design Submittals

When factual data and interpretive information are to be formally presented in one or more project geotechnical reports, there can be a need for the submittal of less formal data packages, preliminary reports and memoranda during the course of project planning and design. Subsurface explorations and testing may be conducted in phases or by areas, or for specific individual design concerns. Feasibility or economic studies may be carried out for alternative designs or routes, many of which will not be part of the final project design. Sometimes there will be detailed geotechnical investigation and analysis to respond to a particular design-phase concern, such as corrosion or expansive soils.

Preliminary geotechnical reports and memoranda should be prepared for timely submittal to appropriate members of the design team whenever this information will benefit the design process. In addition to project subsurface exploration, field and laboratory test and observation well monitoring results, preliminary data submissions can include pre-existing information and results of remote sensing or geophysical surveys. Early submittal of geologic or engineering evaluation of preliminary data may be necessary to establish a basic design concept or design criteria. Study profiles based on available subsurface information may warrant interim dissemination to other members of the design team.

As long as there will be wrap-up formal reports, the individual informal preliminary reports and memoranda do not necessarily have to be complete in themselves. They are a part of the design process, and will be superseded by the formal reports that may become part of the construction process. Informal reports and memoranda do have a need for accuracy and completeness with respect to the planning or design aspect to which they apply, since they can be a basis for major design decisions.

Informal preliminary reports and memoranda should clearly define the basis, limits, applicability and intent of the information that is presented. It is important that the information not be subsequently misinterpreted, incorrectly extrapolated or otherwise misused because of uncertainty or misunderstanding as to its applicability to another area or concern. Any report or memorandum should also reference other related reports that are superseded, amplified or revised.

10.5.3 Data Reports

Factual geotechnical information that has been compiled and developed for a project (Section 10.3) should be presented in one or more formal data reports. There are often separate subsurface exploration and laboratory testing reports, and there may be other reports covering specialized methods of exploration, such as seismic survey. Geotechnical data reports should be completed and distributed to appropriate members of the design team as early as practicable. In final form data reports will also provide reference information during contract bidding and construction.

Data reports should include factual descriptions of the types of data and the methods by which the information has been obtained. Wherever possible the methods should be referenced to standard AASHTO or ASTM procedures, with any departures noted. It is necessary to locate individual subsurface explorations. The locations are ordinarily presented on an enclosed plan, although it is sometimes sufficient to provide coordinate or station locations on individual logs or in a table. It may also be desirable to summarize or tabulate other specific items from the data.

The data itself is the reason for geotechnical data reports, and it should be complete, clear and accurate. For the most part the information should be presented in the manner detailed in preceding sections for individual exploration and testing techniques. Plotted curves based on raw data and standard calculations should be utilized wherever they will provide a more effective presentation for the information. Individual data sheets should always identify the following:

- Project name and number
- Agency and/or firm producing information
- Type of exploration, test or monitoring
- Exploration or test number or other identification
- Exploration or test location or specimen source
- Date

Often it is helpful to show the results of field and laboratory tests on the finished boring log to provide a quick reference to the soil properties of each stratum. Agency policy or project considerations may also dictate departures from or additions to the more usual content of data reports.

10.5.4 Interpretive Reports

Geotechnical information that has been developed by geologic or engineering evaluations and analyses for a project should also be presented in one or more formal interpretive reports. These reports should be as brief, concise and definite as possible. An interpretive report in the early stages of a project may be geology-oriented, presenting an evaluation of route or site geology as it may affect the project, while a latter report may emphasize engineering evaluation and recommendations.

The development of interpretive geotechnical information for a project will not normally be complete until planning and design have progressed to the point where specific recommendations can be made for all of the geotechnical aspects of the work. Final alignment and/or geometry will have been selected, and the magnitude of design loads will be known. If there is to be a single formal geotechnical interpretive report for a project it will be prepared at this stage to present final recommendations for the designed project.

The analyses upon which informal or preliminary reports and the formal geotechnical interpretive report are based will have addressed various applicable geotechnical considerations. The formal interpretive report will present the results of the analyses, along with design recommendations and a discussion of construction considerations. However, the formal interpretive report will not normally include information, such as studies of possible alternatives, that does not pertain to the as-designed project.

The following generalized outline of the contents of a formal interpretive report for a design project is included as a guide. It will not apply directly to all projects, being incomplete for some and too broad for others. Each project should be considered individually to develop a report outline that is appropriate for the particular project.

- Site conditions pertinent to geotechnical evaluations.
- Geology and subsurface soil and rock conditions on and near the site or alignment, including descriptions of major soil strata. This would normally be referenced to a location plan and/or soil profile sheets, which would be part of the report.
- Groundwater conditions on and near the site or alignment. This would also normally be referenced to profile sheets.
- Soil and rock properties as determined from laboratory tests and literature reviews. Recommended properties for design would be assigned to basic strata.
- Foundation conditions, and recommended foundation type and criteria for design of bridges, retaining walls, buildings and other structures.

- Estimated settlement of structural elements at recommended loadings.
- Recommended lateral earth pressures for design of temporary earth support and permanent walls.
- Evaluations of short and long-term stability and settlement of excavations and embankments, including recommendations for cut and fill slopes.
- Evaluations of the materials that will be encountered in excavations, the uses of these materials, and the availability of borrow materials.
- Evaluation of anticipated "zone of influence" of proposed construction on adjacent property and structures, and consideration of factors contributing to the influence.
- Evaluation of anticipated dewatering procedures and their effects, and recommendation of measures to minimize adverse effects on adjacent property and structures.
- Evaluation of anticipated excavation support procedures and potential ground movements in areas of adjacent property and structures, including consideration of underpinning, where appropriate.
- Recommended pavement sections and floor slab support, with associated waterproofing and/or drainage considerations.
- Description of recommended program of instrumentation and monitoring.
- Recommended technical provisions of specifications related to geotechnical aspects of the project.

It should be noted that soil and rock profiles and geologic mapping are interpretation to the extent that they depict conditions at points between exploration locations. Indicated conditions at intermediate points are "probable." Actual conditions may differ, and the map or profile should clearly so indicate. There should also be a reference to the location or availability of the data upon which the map or profile is based. Soil profiles are commonly drawn with the vertical scale exaggerated and the strata outlined or "hatched" to emphasize the interpreted soil, rock and water conditions. The amount of detail that is presented on a map or profile will be a function of both the available information and the engineering requirements of the project. Details relative to project design, such as planned foundation locations or elevations, can be helpful in the evaluation of profiles, but they are subject to change after completion of the geotechnical report. Such presentations should be avoided or clearly labelled as approximate or preliminary.

10.5.5 Contractor Investigations and Briefings

Contract documents will frequently contain some reference to it being the responsibility of the contractor to make his own investigation of subsurface conditions in order to obtain information for bidding. In some circumstances, generally for a major or complex project or when there is particular uncertainty as to subsurface conditions, one or more contractors may elect to carry out borings or test pits on their own. However, such explorations will generally be limited and infrequent because of time and cost limitations.

For major projects there are often pre-bid or preconstruction briefings, at which time information on unusual or complex subsurface conditions or foundation construction can be presented. The presented geotechnical information should also be available to contractors in the contract documents, or should be distributed by addendum, so that all involved parties will have access to the same information. There should also be a distinction between information that is considered to be fact and that which has been inferred or interpreted, and is thus subject to the judgement of the contractor as to its applicability or validity.

10.6 REFERENCES

"Specifications For Subsurface Investigations." Columbus, Ohio: Ohio Department of Transportation, 1984.

"Wyoming Highway Department Engineering Geology Procedures Manual, 1983." Cheyenne, Wyoming: Wyoming State Highway Department, 1983.

APPENDIX A
Drilling, Sampling and Installation Procedures

A selected summary of the various drilling sampling and instrumentation installations procedures required to obtain the necessary subsurface information and the associated forms to record the data are included within Appendix A. This summary is not intended to be all inclusive of the variety of methods and procedures discussed throughout the various sections of this Manual. The more basic and simple procedures, which will vary from organization to organization, and a detailed discussion of the very complex methods which are beyond the scope of this Manual, are not discussed in more detail.

The various field procedures are included only for general guideline purposes, realizing that specific operational details are dependent upon local practice, available equipment and subsurface conditions. Standard and acceptable procedures in one part of the country may be prohibited or not available in other areas.

Various field logging forms are also included with the applicable procedure for general information purposes. Specific field forms will also vary between organizations and any format which records all the necessary information is acceptable.

FIELD REPORT FORMS

1. Daily Report—Test Borings
2. Test Boring Report
3. Core Boring Report
4. Groundwater Observation Well Report
5. Piezometer Installation Report
6. Test Probe Report
7. Test Probe Summary
8. Test Pit Report
9. Field Production Summary Report

GENERAL FIELD PROCEDURES

1. Rock Coring
2. Observation Wells
3. Piezometers
4. Exploratory Probes
   - Hand Probes
   - Percussion Probes
   - Acoustic Probes
5. Exploratory Test Pits
6. Thin-Wall Open Drive Sampling
7. Mechanical Stationary Piston Sampling
8. Hydraulic Piston Sampling
9. Denison Sampling
10. Pitcher Sampling

ROCK CORING

Purpose

Rock coring techniques are used to obtain a continuous sample of rock at the project site for field logging and to provide samples of intact rock for laboratory testing. Additional information about the rock mass is available by careful observation of rig performance during drilling; such factors as drilling rate, bit wear and loss of drilling fluid. Rock coring usually only provides a small amount of information about the rock mass at a project site. This information must be extrapolated toward developing engineering recommendations concerning the entire rock mass. Hence, careful observations during drilling and careful logging of the recovered core are absolutely essential to each investigation.

Equipment

1. Drill rig
2. Barrel, core, and bit (Variety available)
3. Rods, drill (AW size or larger; occasionally may require NW rods)
4. Water supply ("Mud" optional)
5. Time piece
6. Boxes, core
7. Pen, indelible
8. Labels
9. Camera (optional)
10. Hammer, Schmidt (optional)
11. Knife, pocket
12. Lens, hand, magnifying
13. Reference material and forms
14. Protractor
15. Ruler
16. Tape, cloth
17. Device, sounding

There is no universal core barrel or drilling equipment for rock coring. The geologic and topographic conditions, in addition to the engineering requirements will dictate the type of equipment to be employed on any specific project. The following factors lead to good core production:

1. Insure a level and stable drilling platform before commencing boring.
2. Insure that the drill stem remains as nearly vertical as possible. On deep core holes, true alignment of the casing is critical. The driller may elect to use a heavy drilling mud instead of casing to support the borehole walls; this procedure is not as desirable under some conditions, but acceptable if satisfactory information is obtained.
3. Wash the casing out thoroughly.
4. Inspect the selected core barrel and bit for wear, general cleanliness, and free movement of all parts. Reject any barrel or bit that appears unsatisfactory.
5. Pump recirculated drill fluid down the drill rods and observe a return flow before commencing drilling operations.
6. Carefully measure all lengths of rod, core barrel, and stick-up through all phases of drilling to insure accurate depth determination.
7. Drill with minimal vertical pressure and rotation. Most rigs are equipped with a selection of gear ratios and a variable hydraulically-controlled feed mechanism. Driller expertise in selecting the correct combination of speed and feed rate is invaluable.
8. Water return should be no more than what is just sufficient to bring the borehole cuttings to the surface.
9. Record the drilling time per foot, type of bit, estimate of bit wear, drill rig R.P.M. and feed pressure.
10. Place the core carefully in the core box from left to right, top to bottom. Carefully examine and classify the rock, and measure the recovery and Rock Quality Designation (RQD) in percent. Record all information on Core Boring Report.
11. If 100% recovery was not obtained, sound the borehole to determine if the missing core still remains in the bottom of the borehole.
12. Terminate each boring, whenever possible, with 100% recovery, in order to insure that appropriate knowledge is available of these materials.
13. The Field Inspector at the test boring site must insure that all technical information is obtained regarding the recovered core specimens, and that all rock core is suitably packaged and shipped. Refer to Section 6 for preservation and shipment of samples.
14. It may be desirable that the rock core be photographed in color. After photographic recording, selected specimen(s) of rock core may be designated for removal and testing. All rock removed for laboratory testing should be replaced by a block of wood bearing notation of the missing footage. (Refer to Section 6).

The procedures for drilling angle-holes is essentially the same as for the vertical variety, except that specially-designed drill rigs and derricks are required to produce the desired angle of inclination. Several types of instruments are available to monitor the angle and drift of the borehole.

The procedures for wireline drilling are also the same as for conventional holes, with the exception that the core barrel is designed so that the inner core barrel can be raised on a wireline without removing the entire drill string and outer core barrel and bit. The drilling rig must be equipped with a wireline hoist.

If precise spatial orientation of rock bedding, foliation and discontinuities are required special orienting core barrels, equipment and procedures are necessary. Refer to Section 6.

Shotcore drilling is usually employed to produce large-diameter rock core (2 to 6 ft., and larger). The core is cut by the abrasive action of chilled steel shot fed to a rotating soft steel bit.

DAILY REPORT - TEST BORINGS

FILE NO. 1000 DATE 7 May 1980 REPORT NO. 7

PROJECT & LOCATION EXAMPLE

CLIENT EXAMPLE INSPECTOR(S) Ed Smith INSPECTION TIME 8 hrs.

CONTRACTOR C. L. Guild DRILLER(S) Don White

TYPE OF RIG(S) Acker Skid NO. RIGS WORKING 1 WORKING TIME 7 hrs.

NON-PRODUCTIVE TIME

WEATHER ______ EQUIPMENT BREAKDOWN ______ LATE 1 hr. OTHER ______

WORK PRODUCTION TODAY

NO.	TYPE	OVER BURDEN (LIN. FT)	ROCK CORE (LIN. FT)	SHELBY TUBES TYPE & NO.	MISC. (WELLS ETC.)	MATERIALS ENCOUNTERED (TYPE & DEPTH RANGE)
B5	0	23	5	1T,1U	-	0-1 Topsoil, 1-3.5 Loess, 3.5-8.0
						gravelly Sand, 8.0-12.0 Silts &
						Sands, 12.0-15.0 Stiff Clay,
						15.0-20.0 soft Clay, 20.0-23.0
						Till, 23.0-28.0 Bedrock

STATUS OF TOTAL PRODUCTION

ITEM	BORING FOOTAGE & SAMPLES				NO. BORINGS		RIG-DAYS	
	TODAY	TOTAL TO DATE	ENG. EST.	% COMPLETE	TO DATE	REMAIN	TO DATE	REMAIN
2½ STANDARD		50	100	50	1	1	2	2
3½ STANDARD		100	100	100	2	0	4	0
2½ CORE PREP	23	23	40	100	1	0	2	0
CORE PREP								
2½ ROCK CORE	5	5	5	100	1	0	1	0
ROCK CORE								
2" SHELBY TUBES	1	1	4	25	-	-	-	-
3" SHELBY TUBES	1	1	1	100	-	-	-	-

SEE OTHER SIDE FOR SYMBOLS & REMARKS

SYMBOLS

M: 2½" standard
N: 3½" standard
O: 2½" core prep
R: 3½" core prep
W: water borings
X: continuous samplings
Z: probes or soundings

C: rock core
U: 3" piston tube
T: 2" shelby tube
V: vane shear test
P: permeability test
Y: pressure test
S: observation well

REMARKS

Client visited site to check on progress.

Inspector located remaining borings and obtained elevations.
Driller 1 hour late in arriving at site. Production good.

PROJECT SUPERVISOR M.C.M. REVIEWED BY J.T.H. DATE 10 May 1980

Appendix A

TEST BORING REPORT

HOLE NO.	B5
FILE NO.	1000
SHEET NO.	1 of 1
LOCATION:	Sta 2+0, 10'lt.
ELEVATION:	16.8
DATE START:	5 May 70
DATE FINISH:	7 May 70
DRILLER:	Don White
INSPECTOR:	Ed Smith

PROJECT: EXAMPLE
CLIENT:
CONTRACTOR: C.L. Guild Drilling & Boring Co., Avon, MA

GROUNDWATER		DEPTH TO:		
DATE	TIME	WATER	BOTTOM OF CASING	BOTTOM OF HOLE
7 May	½ hr.	3.0	23.0	28.0
7 May	2 hr.	5.0	5.0	20.0
8 May	24 hr	8.0	none	15.0

	CASING	SAMPLER	CORE BARREL
TYPE:	flush	S/S	D.D(M)
SIZE I.D.:	2½	1-3/8"	1-3/8"
HAMMER WT.:	300#	140#	
HAMMER FALL:		30"	

SCALE IN FEET	STRATA CHANGE	CASING BLOWS PER FOOT	SAMPLER BLOWS PER 6 INCHES	SAMPLE NUMBER	SAMPLE DEPTH RANGE	FIELD CLASSIFICATION AND REMARKS
		1	2, 3	S1	0.0 – 1.0	Loose dark brown sandy TOPSOIL, trace roots
	1.0	2	5, 6, 7	S2	1.0 – 2.5	Medium compact yellow-brown, fine sandy SILT, trace fine gravel, cobbles & roots - LOESS -
		2				
	3.5	20				Compact brown gravelly coarse to fine SAND, trace silt & small cobbles
		82				
5		15	10, 13, 20	S3	5.0 – 6.5	
		76				
		22				
	8.0	10		T1	8.0 – 10.0	Medium compact grayish brown, interbedded silty fine SANDS & non-plastic SILTS -- partings & layers. Drove 2" Shelby tube-Rec 20"
10		11				
		15	8, 9, 10	S4	10.0 – 11.5	
		17				
	12.0	23	5, 6, 7	S5	12.0 – 13.5	Stiff yellow-brown, mottled, silty CLAY
		29		N.R.	13.5 – 15.0	Attempted 3" undisturbed piston sample--no recovery
		30				
15	15.0	5	2, 2, 2	S6	15.0 – 16.5	Soft, blueish gray CLAY
		6				Pressed 3" undisturbed piston sample 24', Rec.20"
		7				
		4		U1	18.0 – 20.0	
		3				
20	20.0	68	23, 36, 40	S7	20.0 – 21.5	Very compact, gray medium to fine sandy SILT, little coarse to fine gravel, trace clay, cobbles & boulders. Cored 18" granite boulder
		105		C1	21.5 – 23.0	
		210				Top of Bedrock
	23.0	9		C2	23.0 to 28.0	Hard, gray fine-grained Quartzite, faint bedding dipping 30° to 40°, few tight near vertical joints, slight weathering. Rock is sound. Numbers in left column are drilling time/min.
		10				
25		7				
		8				
		15				
	28.0					Bottom of Exploration

BLOWS FT.	DENSITY	BLOWS FT.	CONSISTENCY	SAMPLE IDENTIFICATION	
0-4	VERY LOOSE	0-2	VERY SOFT	S	SPLIT SPOON
4-10	LOOSE	2-4	SOFT	T	THIN WALL TUBE
10-30	MEDIUM COMPACT	4-8	MEDIUM STIFF	U	UNDISTURBED PISTON
30-50	COMPACT	8-15	STIFF	O	OPEN END ROD
50+	VERY COMPACT	15-30	VERY STIFF	W	WASH SAMPLE

SUMMARY	
OVERBURDEN	23
ROCK	5
SAMPLES	6S, 1T, 1U, C
HOLE NO.	B5

CORE BORING REPORT

HOLE NO. B4 | PAGE 2 OF 2

GEOLOGIST Gevalt

Scale in Feet	Drill Rate Min. per Foot	Core No. / Depth Range	Recovery in.	RQD %	Graphic Log / Weath.	Strata Change / Tests	FIELD CLASSIFICATION AND REMARKS
							- OVERBURDEN - (see page 1) Top of Rock at 22.0 Ft.
						22.0	
	6	22.0					Soft, severely weathered, brown, fine grained QUARTZ BIOTITE GNEISS, trace garnet, joints close to very close, irregular at random angles, several joints parallel to foliation at 75°. Weathering decreases with penetration.
	6	C1	30/20	50/33	Sev.		
25	5						
	6						
	7	27.0					
	5	27.0					
	5						
30	6	C2	40/30	66/50	Mod.	PT1	Pumped 6± gals. per minute into rock for 30 minutes. Test terminated. (see attached test results).
	6						
	7	32.0					
	7	32.0					
	4	C3	20/18	57/50	Sev.	33.5	Major shear zone 33.0 - 33.5
35	8	35.0					
	10	35.0					
	11				V.sl		Very hard, very slightly weathered to fresh dark greenish gray, amorphous, QUARTZ HORNBLENDE GNEISS, joints widely spaced, semi-planar to irregular at 55° to 75° with few at 10° to 20°. Foliation poorly developed at 55° to axis of core.
	12	C4	59/55	98/92		L=35	
	12						
40	15	40.0					
	16	40.0				L=40	
	17						
	16						
	17						
45	18	C5	120/120	100/100	Fr.	PT2	Pumped 0.5 to 0.1 gals. per minute into rock for 15 minutes. Test terminated. (see attached test results).
	18						
	17						
	17						
	18					L=50	
50	19	50.0				50.0	
							Bottom of exploration at 50.0'
							L - Schmidt Hardness Number Rock Core obtained with Christensen NWD-3, 2-1/8" I.D. 5.0 ft. barrel

FIELD HARDNESS	WEATHERING	BEDDING/JOINT SPACING	ROCK CONTINUITY
V. Hard - Knife can't scratch Hard - Scratches difficultly Med. Hard - Scratches readily Medium - Grooves or gouges diff. Soft - Grooves or gouges read. V. Soft - Carves	Fresh V. slight Slight Moderate Med. Severe Severe V. Severe Complete	V. thin - V. Close - <2" Thin - Close - 2" - 12" Medium - Med. Close - 12" - 36" Thick - Wide - 36" - 120" V. thick - V. wide - >120"	Ext. Fractured - Core <1" Med. Fractured - Core 1" - 4" Sl. Fractured - Core 4" - 8" Sound - Core >8"

Appendix A

Ground Water Observation Well Report

PROJECT: PROPOSED MANUFACTURING FACILITY	FILE NO. 9230
LOCATION: LIVERMORE FALLS, MAINE	WELL NO. OW5
CLIENT: BLACKBURN & JONES, BOSTON, MASS	BORING NO. B107
CONTRACTOR: NEVER MISS DRILLING CO., PORTLAND, MAINE	LOCATION Project Base
DRILLER: BILL WHITE INSPECTOR: AL SMITH	Sta. 6+0, 25' Rt.
INSTALLATION DATE JANUARY 18, 1974	SHEET 1 OF 2

SURVEY DATUM USGS, MSL

GROUND ELEVATION 105.0

SUMMARIZE SOIL CONDITIONS (NOT TO SCALE)

Miscellaneous FILL

El. 103

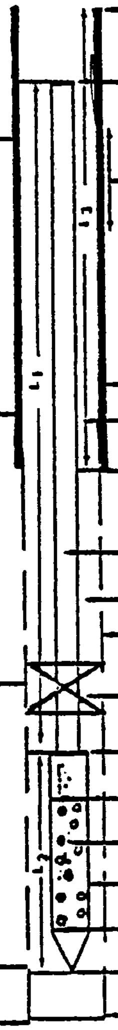
ORGANIC SILT

El. 90

Gravelly coarse to fine SAND

El. 60 TILL

El. 50



ELEVATION ~~XXXXXXXX~~ ABOVE ~~XXXX~~ GROUND SURFACE OF CASING ~~XXXXX~~ ~~XXXXX~~	107.0
ELEVATION ~~XXXXXXXX~~ ABOVE ~~XXXX~~ GROUND SURFACE OF RISER PIPE.	106.0
THICKNESS OF SURFACE SEAL	1.0'
TYPE OF SURFACE SEAL	Concrete
[INDICATE ALL SEALS SHOWING DEPTH, THICKNESS AND TYPE]	
TYPE OF CASING	"NX" F.J.
INSIDE DIAMETER OF CASING	3.0"
ELEVATION/DEPTH OF BOTTOM OF CASING	102.0
INSIDE DIAMETER OF RISER PIPE	1.0"
TYPE OF BACKFILL AROUND RISER	c-f SAND
DIAMETER OF BOREHOLE	3.0"
Bentonite Seal at El. 90 to 100	
ELEVATION/DEPTH OF BOTTOM OF RISER	70.0
TYPE OF POINT OR MANUFACTURER	Jet Screen
SCREEN GAUGE OR SIZE OF OPENINGS	40 MM
DIAMETER OF WELLPOINT	2.0"
TYPE OF BACKFILL AROUND POINT	Ottawa Sand
ELEVATION/DEPTH OF BOTTOM OF POINT	68.0
ELEVATION/DEPTH OF BOTTOM OF BOREHOLE	50.0

[FIGURES REFER TO: EL. X DEPTH ___]

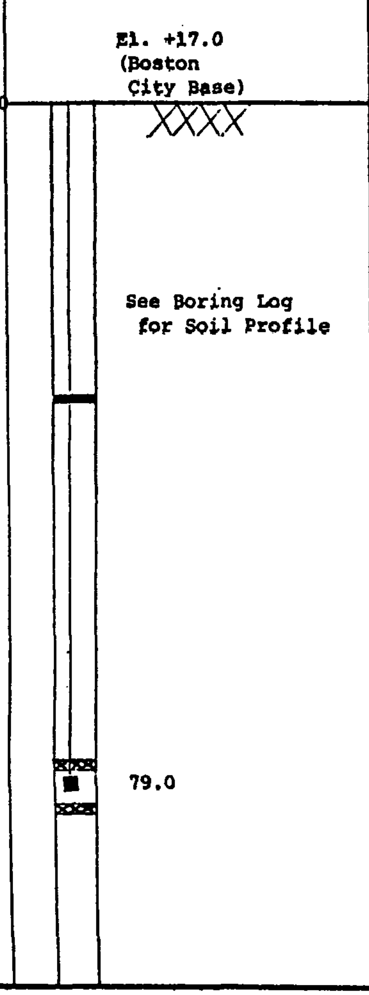
[5.0' LENGTH OF CASING L_3] [35.0 LENGTH OF RISER PIPE (L_1) + 2.0 LENGTH OF POINT (L_2) = 42.0 PAY LENGTH]

Ground Water Observation Well Report

WELL NUMBER: OWS			ELEVATION SUBTRAHEND 105.0		FILE NO. 9230 PG. NO. 2	
DATE	TIME	ELAPSED TIME	DEPTH OF WATER FROM GRD. SURF.	ELEVATION OF WATER	REMARKS	READ BY
7-25-75	-	-	-	-	O.W. Installed	CHM
7-4-75	0900	10	10.8	94.2		CHM
8-22-75	0910	28	11.2	93.8		CHM
8-28-75	0830	34	11.0	94.0		DEA
8-28-75	1600	34	11.0	94.0		DEA

PIEZOMETER INSTALLATION REPORT

PROJECT: EXAMPLE	FILE NO. 1000
LOCATION:	INSTRUMENT TYPE Porous Tip
CLIENT:	INSTRUMENT NO. 101-1
CONTRACTOR: BAY STATE TEST BORING, INC.	LOCATION B101
DRILLER: F. PERRY INSPECTOR: J. McRAE	
INSTALLATION DATE 6 July 1976	SHEET 1 OF 1

DEPTH IN FEET	ELEV. IN FEET	INSTALLATION DIAGRAM	INSTALLATION COMMENTS
0	17.0		Roadway Box Installed Over Piezometer Leads 0.25" Polyethylene tubing 0.0 - 34.0 Granular Backfill
20			34.0 - 35.0 Bentonite Seal (tamped)
40			35.0 - 77.5 Granular Backfill
60			77.5 - 77.0 Seal consisting of 6" Bentonite, 6" Sand, 6" Bentonite (tamped) 6" Bentonite (tamped) 77.0 - 80.0 Granular Collection Zone 79.0 Tip of Piezometer Sensor
80			80.0 - 81.5 Seal consisting of 6" Bentonite 6" Sand, 6" Bentonite (tamped)
100			81.5 - 100.0 Granular backfill

[2.1 LENGTH OF CASING] [78.0 LENGTH OF RISER PIPE] + [1.0 LENGTH OF POINT] = [79.0 PAY LENGTH]

TEST PROBE REPORT

PROJECT: PROPOSED BUILDING ADDITION	PROBE NO. 5
CLIENT: EXAMPLE	FILE NO. 1000
CONTRACTOR: DRILL MORE CONSTRUCTION CO	SHEET NO. 1 of 1
	LOCATION: See Plan

	TOP OF ROCK (FEET) BASED ON:	
Diameter of Hole $2\frac{1}{2}$"	RATE OF PENETRATION 9.5	GROUND EL. 44.5
Drill Bit Type Carbide	ACOUSTIC DEVICE 9.5	WATER EL. Not determined
Drill Bit O.D. $2\frac{1}{2}$"	DRILLERS OPINION 9.0	DATE 6-6-61
	ELEVATION TOP OF SOUND ROCK 35.0	DRILLER: D. Rodman
		INSPECTOR: -
		GEOLOGIST: P. Morris

EQUIPMENT USED: Joy Ram Airtrack; Ingersol Rand 900 Compressor

SCALE IN FEET	RATE OF PENETRATION (min./ft)	DECIBEL READING	CUTTING DESCRIPTION AND REMARKS
	0:03	0.10	NOTE: Gray - brown silty coarse to fine SAND, trace-little fine gravel cuttings
	0:03	0.15	
	0:03	0.10	- ARTIFICIAL FILL -
	1:15	0.15	
5	1:30	0.20	Boulder or
	1:20	0.10	Concrete - 6.0'
	0:05	0.15	
	0:06	0.20	- NATURAL SOIL -
	0:05	0.15	9.0'
10	1:30	0.40	
	2:15	0.55	
	2:00	0.50	
	2:20	0.60	- BEDROCK -
	2:30	0.55	
15	2:25	0.55	
	2:25	0.57	
	2:30	0.56	
	2:35	0.54	
	2:35	0.56	
20			BOTTOM OF PROBE AT 19 FT.
25			

TEST PROBE SUMMARY

PROBE NO. P1-P6

PROJECT: PROPOSED WAREHOUSE ADDITION

CLIENT: EXAMPLE

CONTRACTOR: AL SHINER TEST BORING

EQUIPMENT USED: BOMBADIER

FILE NO. 1000
SHEET NO. 1 of 1
PROBE TYPE Solid Augers
WATER EL. See Remarks
DATE 6-5-70
DRILLER: P. Cecil
INSPECTOR: ---
GEOLOGIST: J. Hull

PROBE NO. OR LOCATION	SURFACE ELEVATION	DEPTH PROBED FROM (FT.)	DEPTH PROBED TO (FT.)	SOIL STRATA DESCRIPTION	REMARKS
P1	103.5	0.0	5.0	Br. SAND & Gravel	No water
P2	102.7	0.0	6.3	Br. SAND, little gravel	Water @ 6.0'
P3	103.0	0.0	7.0	Br. SAND, li.gravel,tr.silt	Refusal@ 7.0'
P4	102.5	0.0	5.5	Br. sandy Gravel	No water
P5	103.0	0.0	6.0	Br. gravelly SAND	Water @ 5.8'
P6	103.2	0.0	6.5	Br. SAND, li. gravel	No water

TEST PIT REPORT

TEST PIT NO. 1

PROJECT: EXAMPLE

CLIENT:

CONTRACTOR: CHARLES D. JONES INC.

EQUIPMENT USED: CASE 580, 3/8 CUBIC YARD BACKHOE

FILE NO. 1000

LOCATION: See Plan

ELEVATION: 202.7

EXPLORATION DATE: 10 Nov'75

INSPECTOR: John Smith

SCALE IN FEET	STRATA CHANGE	SAMPLE NUMBER	SAMPLE DEPTH RANGE	DESCRIPTION OF MATERIALS	REMARKS
				Dark brown loamy TOPSOIL	
2	1.0	Jar # 1	1.0 – 3.5	Yellow-brown fine SAND, trace silt - LOESS -	Note: Walls of pit caving in from 1.0-2.0'
4, 6	3.5			Dark brown-black fine sandy ORGANIC SILT with dessicated tree trunks	Note: Very strong organic odor
8, 10	8.0	Bag # 1	8.0 – 11.0	Gray-brown gravelly coarse to fine SAND, trace silt with occassional cobble - GLACIAL OUTWASH -	Note: Water entering pit at 8.0'
12	11.0			Gray sandy SILT, little coarse to fine gravel, trace cobbles and boulders - GLACIAL TILL -	
				Refusal on large boulder at 12.0'	

GROUNDWATER

DATE	TIME*	DEPTH FT.
10/11/75	0.0	8.0
10/11/75	2.0	4.0
NOT ENCOUNTERED		* HRS. AFTER COMPL.

PIT DIMENSIONS

12 (L) × 5 (W) × 12 (D) = 720 Cu. Ft.

BOULDERS

8" to 18" DIAM: No. 5 ; Vol. 7.5 Cu. Ft.

Over 18" DIAM: No. 2 ; Vol. 32.0 Cu. Ft.

SUMMARY

DEPTH 12.0'

JAR SAMPLES 1

BAG SAMPLES 1

GROUNDWATER 4.0

TEST PIT NO. 1

PRODUCTION SUMMARY (FINAL FIELD REPORT)

PROJECT: EXAMPLE

CONTRACTOR: C.L. GUILD DRILLING & BORING CO., INC.

FILE NO. 1000

REMARKS	BORING NUMBER	GROUND SURFACE ELEV. (FT.)	2½" SHALLOW SOIL (FT.)	2½" DEEP SOIL (FT.)	2½" CORE PREP (FT.)	3½" SOIL (FT.)	1 3/8" ROCK CORE (FT.)	2" THIN WALL TUBES (NUMBER)	3" PISTON TUBES (NUMBER)	DATED STARTED	DATE FINISHED	DRILLER	RECEIVED DRILLERS LOG	HEA LOGS MAILED
	B601	22.8	12.0							5-30-72	5-30-72	E.W.	✓	✓
CORE BEDROCK	B602C	22.3			217.0		9.5	4		5-31-72	6-13-72	A.W.	✓	✓
PISTON SAMPLES	B603R	20.4		21.5		180.5			1	6-1-72	6-9-72	E.H.	✓	✓
	B604C	20.8			211.0		12.5	4		5-31-72	6-13-72	J.P.	✓	✓
	B605	20.6	32.0							5-30-72	5-30-72	J.P.	✓	✓
	B606	22.6	8.0							5-31-72	5-31-72	C.P.	✓	✓
	B607	20.4	27.5							5-30-72	5-31-72	F.W.	✓	✓
PISTON SAMPLES & CORE	B608C	20.2		30.5		172.0	10.0			6-13-72	6-23-72	K.F.	✓	
	B609	20.5	34.0							6-2-72	6-2-72	F.C.	✓	✓
	B610	21.3	32.0							5-30-72	5-31-72	J.P.	✓	✓
	B611	20.5	32.0							5-30-72	5-30-72	J.P.	✓	✓
	B612C	22.5			200.5		10.0	2		6-14-72	6-21-72	J.P.	✓	
REFUSAL	B613R	20.8		75.5		120.0		1	1	6-9-72	6-14-72	J.P.	✓	✓
	B614	20.6	36.0							6-1-72	6-1-72	E.H.	✓	✓
ADD BORINGS:	B601A	22.8	36.0							5-30-72	5-30-72	C.P.	✓	✓
	B606A	22.6	35.0							5-3-72	6-1-72	F.W.	✓	✓
	Totals:		284.5	127.5	628.5	472.5	42.0	11	4					

OBSERVATION WELLS

Purpose

Groundwater observation wells are installed where fairly regular groundwater conditions are encountered in relatively porous soils or bedrock. Figures A and B show two possible types of observation well installations. Observation wells generally consist of 2.5–5 cm (1–2 in.) diameter metal well points or slotted p.v.c. pipe. However, many other types and sizes available may be suitable for the project involved.

Equipment

1. Wellpoint (various sizes and types available)
2. Pipe, riser (to match wellpoint)
3. Casing, protective with vented cap, or Roadway Box (gatebox)
4. Soil, granular, backfill, common
5. Sand, Ottawa, or clean, well-graded pea gravel
6. Bentonite, ball-form, or other sealant
7. Concrete (optional)
8. Ruler
9. Tape, cloth
10. Forms

Procedure

1. Complete exploratory borehole to required depth, and determine the static water level.
2. If necessary, backfill and tamp hole to the depth at which wellpoint is to be installed. Information required will dictate type of backfill. Wellpoint may be installed in a completed borehole or in a supplementary hole made specifically for the observation well.
3. Attach wellpoint to riser pipe and lower to required depth. Pour Ottawa sand (per specification of ASTM Designation C-190) or clean pea gravel slowly around wellpoint to approximately 5 feet above the point.
4. Backfill annular space between riser pipe and borehole wall with common fill material.
5. Add backfill around riser pipe to bottom of protective casing. Install impervious seals as required to isolate specific zones or to insure surface waters do not percolate to the wellpoint area. Bentonite and/or concrete seals may be adjusted as program specifications require.
6. Install casing with vented cap or roadway box at ground surface to protect riser pipe. If necessary, secure installation with a locking cap device such as that shown in Figure C.
7. Install surface seal of concrete around protective casing at ground surface.
8. Fill the riser pipe with clean water to ground surface. Monitor, by means of measuring and recording the fall of the water level inside the riser pipe, to determine the operability of the wellpoint.
9. Locate and tie-in the installation to visible surface features.
10. Record installation procedure on Groundwater Observation Well Report.
11. Record groundwater observations as required during the duration of the project.

PIEZOMETERS

Purpose

A piezometer is an instrument which provides measurements of pore water pressure at the elevation of the installed sensor. These instruments are frequently installed in soil or rock to observe natural groundwater levels, to detect artesian water pressures, to observe the effects of pumping from adjacent wells, and/or to monitor the immediate and long-term effects on pore water pressure due to construction activities (e.g., embankment loadings, deep excavation, dewatering, etc.).

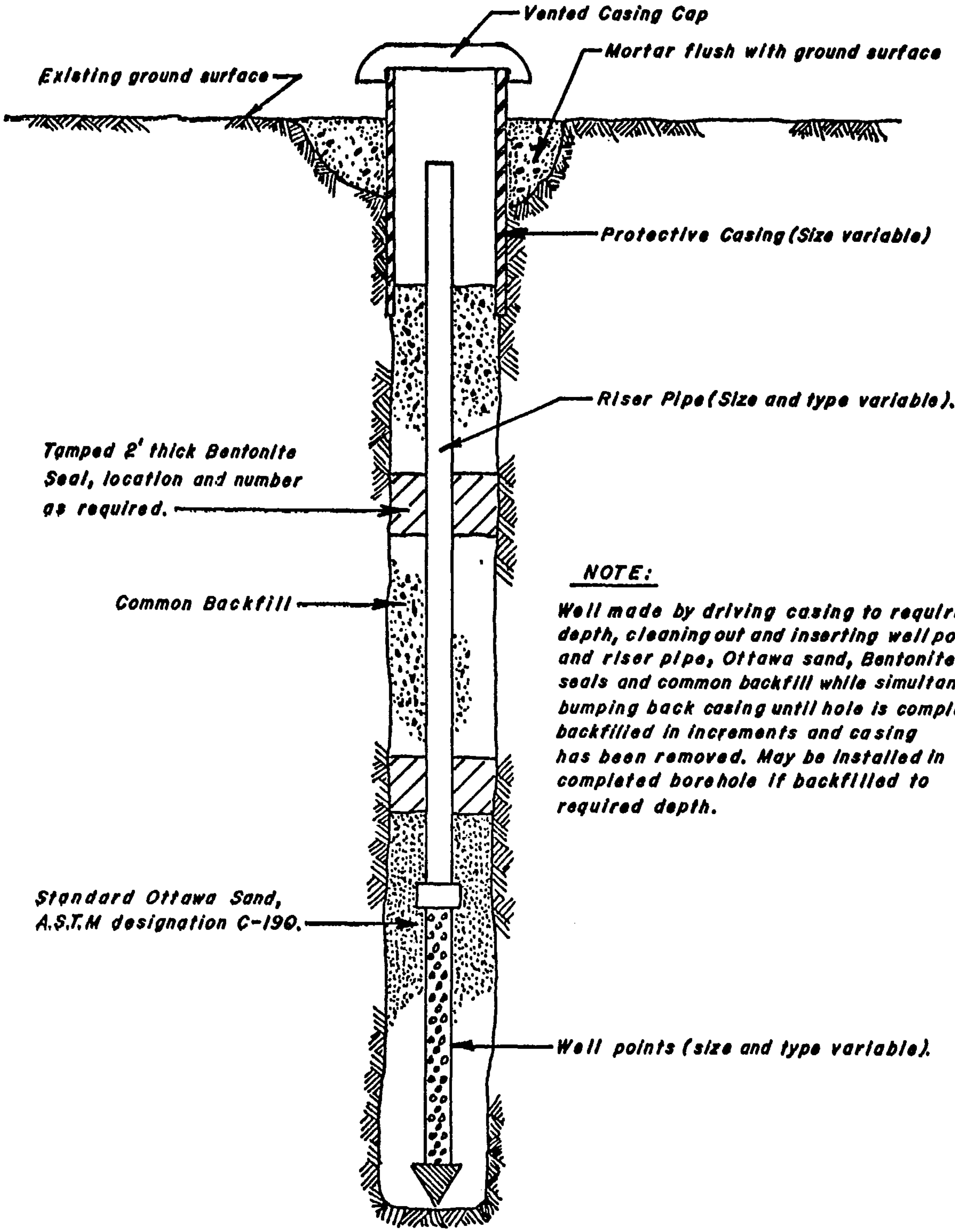
There are various types of piezometers, ranging from the extremely simple standpipe to the complex and expensive pneumatic or the electrical vibrating wire types. See Figs. A and B for details of two types of piezometers and piezometer installations.

The piezometer specified must be tailored for each individual project, so as to provide for the appropriate level of accuracy and evaluation. Some of the various factors influencing choice include:

1. Properties of soil or rock into which the piezometer will be placed.
2. Data requirements related to project specifications (i.e., response time, durability, instrument life, accuracy, reliability).
3. Read-out or monitoring requirements (frequency of readings, access, location, etc.).
4. Installation restraints.
5. Cost.

Equipment

1. Drill rig
2. Piezometer Sensor (electrical, hydraulic, pneumatic, etc.)



NOTE:

Well made by driving casing to required depth, cleaning out and inserting well point and riser pipe, Ottawa sand, Bentonite seals and common backfill while simultaneously bumping back casing until hole is completely backfilled in increments and casing has been removed. May be installed in completed borehole if backfilled to required depth.

Figure A

OBSERVATION WELL

(ISOLATED TYPE)

NOT TO SCALE

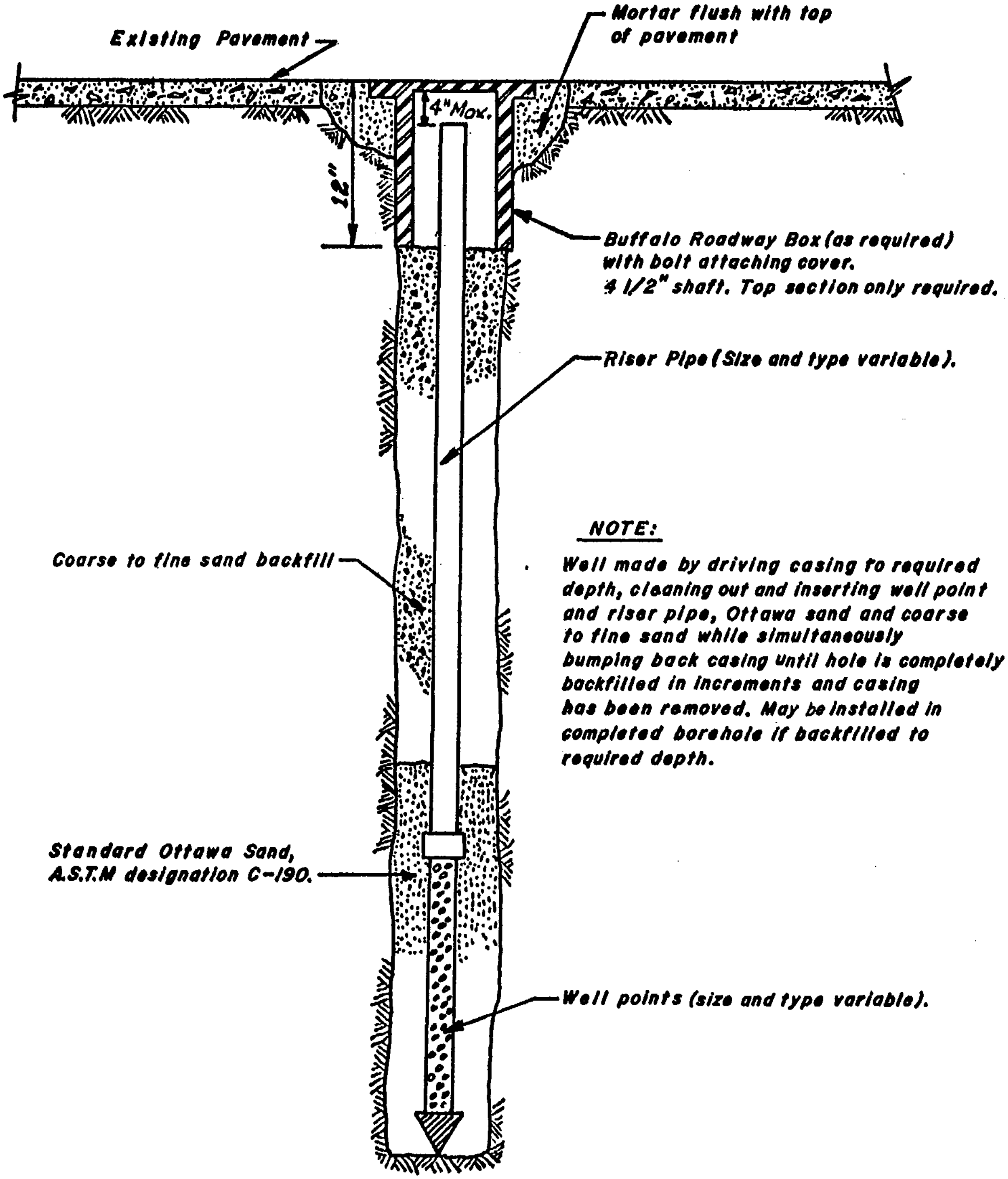
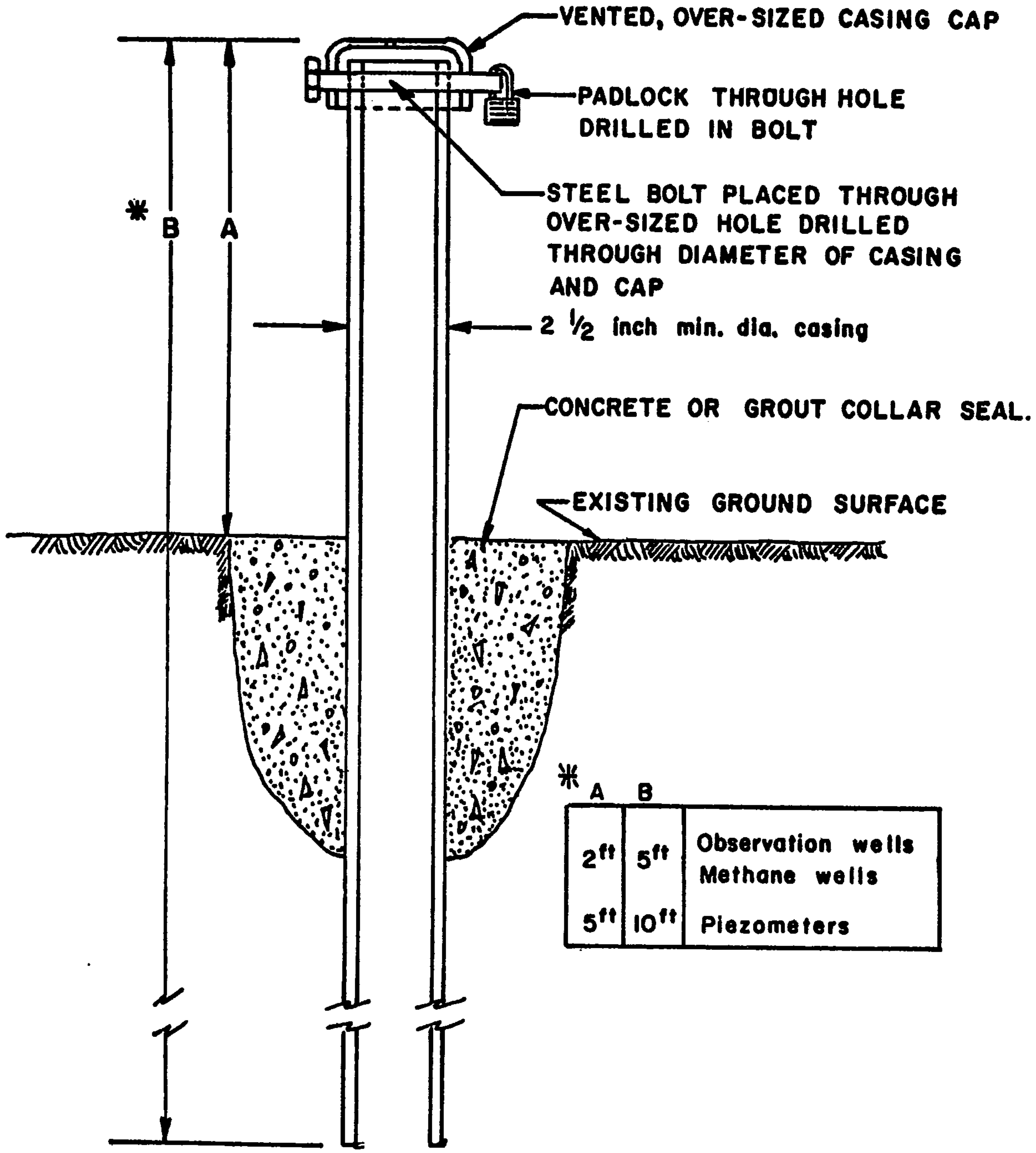


Figure B

OBSERVATION WELL
(AVERAGING TYPE)
NOT TO SCALE



* A	B	
2 ft	5 ft	Observation wells Methane wells
5 ft	10 ft	Piezometers

Figure C

LOCKABLE PROTECTIVE RISER PIPE DETAIL

NOT TO SCALE

3. Tubing, polyethylene, coaxial cable or P.V.C. tubing and couplings
4. Couplings, P.V.C. tubing (preferably compression type)
5. Unit, readout
6. Tank, surge (standard 10 liter (2.5 gal) pressurized garden insecticide sprayer)
7. Casing, steel, drilling or E-Rod size (used for Geonor vibrating wire and hydraulic M-206 piezometer installations)
8. Bentonite, pellet-form (sealed borehole piezometer installations)
9. Sand, Ottawa, or clean uniform concrete-type (sealed borehole piezometer installations)
10. Hammer, tamping and cable
11. Ruler
12. Tape, cloth
13. Friction or plastic tape
14. Tool, threading (P.V.C. tubing, if required)
15. Weight, sash, 3 ± kilogram (6 ± lb) (or equivalent), approximately 30 cm (12 in)
16. Sand, washed, concrete (backfilling casing)
17. Gatebox (Buffalo Roadway Box), with bolt-attached cover (optional)
18. Mortar, cement

Procedures

There are basically two general types of piezometer installations: those installed in completed boreholes and those driven, or pushed into undisturbed ground. The latter piezometers are most applicable to installations in cohesive soils. Details of each procedure are listed. A Piezometer Installation Report, should be completed after installation of each piezometer.

Piezometers Installed in Completed Boreholes (Permanent Casing Left in Place)

1. Drive casing of the required diameter to the approximate depth at which the bottom of the piezometer sensor will rest. The lowermost 3m (10 ft.) of casing shall not have any external couplings, and the casing is to be open-ended at the bottom. The casing may be advanced by any means, except over the final twenty feet of penetration. It shall then be advanced in 1.5m (5 ft.) increments, and the casing must be washed out after each 1.5m (5 ft.) advance. The casing shall be kept filled with water at all times; no washing below the casing will be permitted.
2. Drive a large diameter split-spoon or thin-walled tube 30cm (12 in.) below the bottom-level depth of the piezometer sensor. Preserve the sample obtained.
3. When the boring is at its final depth and in a clean condition, the tamping hammer is lowered to the bottom of the boring. Friction tape is then tightly wrapped around the tamping cable marking the exact top of the casing bumping coupling. This gives the total boring length. Exact depth should be checked using cloth or steel measuring tape.
4. Remove the tamping hammer.
5. Pour enough saturated Ottawa sand into the casing to equal 30cm (12 in.), and wait until it settles. The quantity of sand required should be computed beforehand.
6. Lower the tamping hammer into the boring and rest it on the sand. The tape mark should be above the bumping coupling a distance equal to the height of sand in the casing.
7. Remove the tamping hammer. If more sand is needed to complete the filter it is then added and the same procedure followed (a minimum of 30cm (12 in.) of sand is recommended below the piezometer sensing element).
8. Connect the leads (in one continuous unspliced length, to extend 3m (10 ft.) above the ground surface) to the piezometer sensor. The entire assembled unit including the sensor should be checked for leaks and all connections tightened and taped. Electrical piezometers are immersed in water except for the read out plug and checked for electrical continuity. Electrical piezometers should then be field calibrated by lowering sensor into hole, stopping and reading at various intervals 1.5–3m (5–10 ft.) spacing. The casing should be kept full of water at all times. Allow electrical sensor to equilibrate to water temperature before starting field calibration. For pneumatic piezometers, all leads and connections are immersed, inspected and pressurized to detect any leaks. Connect a weight to the assembly so that the weight hangs below the piezometer sensor. After the sensor and leads are assembled and tested, slowly lower the unit to the bottom of the open hole; the leads should be marked to insure that the piezometer is placed to the correct depth.
9. Pull the casing up, if required, so that the bottom of the casing is slightly below the top of the sensor, and at the same time, pour a measured volume of Ottawa sand into the casing so that the sand fills the space around the piezometer tip to approximately .75m/2.5 ft above the bottom of the casing. Maintain tension on the leads and do not permit any vertical movement of the piezometer tip.
10. Apply 20 blows to the top of the sand layer

with a 15cm (6 in.) drop of the hammer per blow.

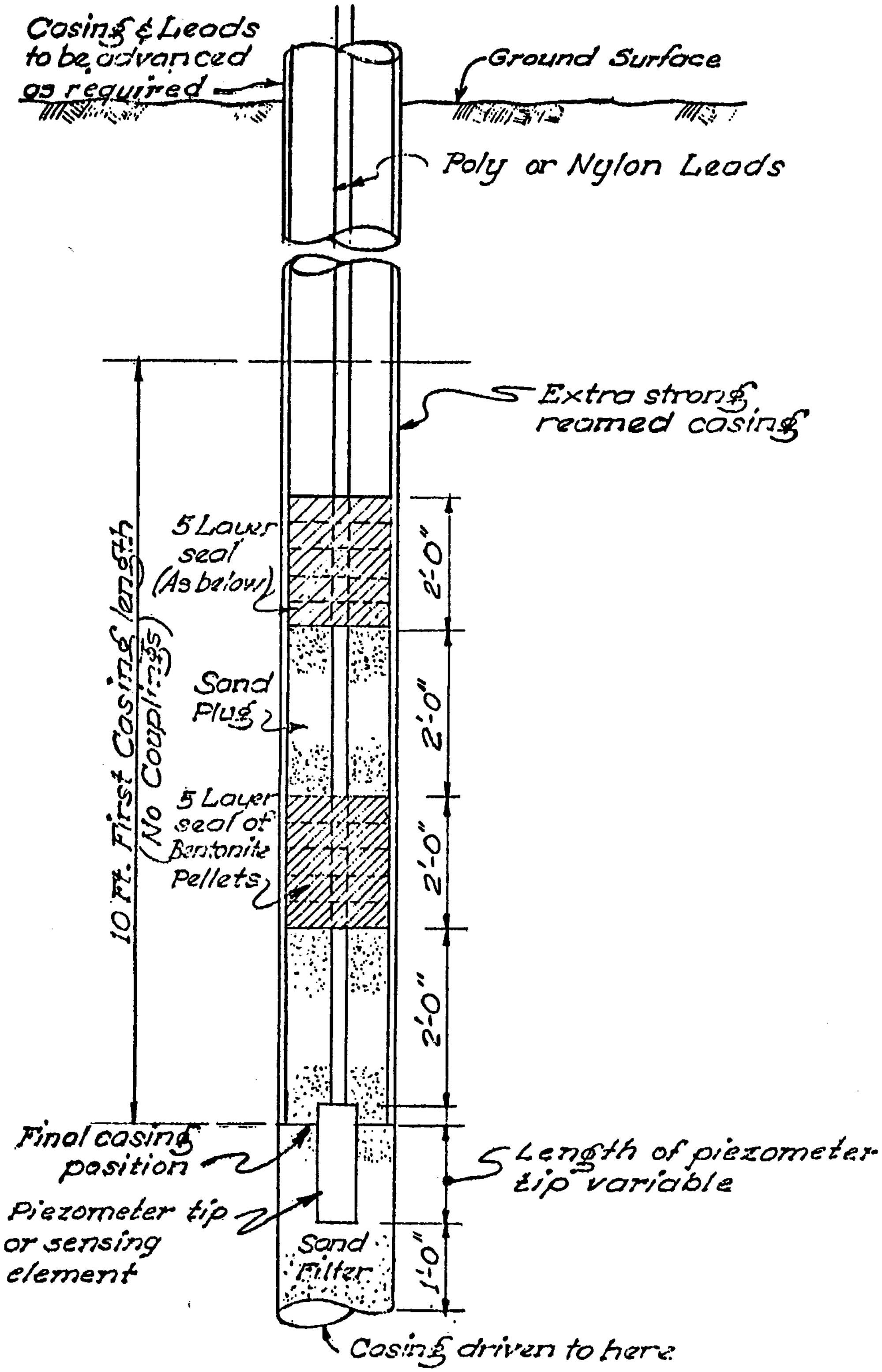
11. As shown on Fig. A, form a bentonite seal of five layers of bentonite pellets, each 7.5cm (3 in.) thick layer placed and compacted while maintaining a constant tension on the tubing. Drop bentonite balls individually until the desired seal thickness is obtained.
12. Slip a tamping hammer, weighing approximately 11.5 kilograms (25 lbs), over the plastic tubing and, keeping tension on the tubing, apply approximately 20 blows to the sand layer with a 15cm (6 in.) drop of the hammer per blow.
13. Repeat this procedure until a .6m (2 ft.) seal is formed. Whenever the tamper does not move freely, it should be immediately withdrawn and cleaned.
14. Pour clean sand into the casing to form a .3m (1 ft.) layer over the bentonite seal and compact with 20 blows of the hammer, with a free fall of 15cm (6 in.) per blow. Repeat the above process until at least a .6m (2 ft.) sand plug is formed.
15. Repeat step 12, forming another bentonite seal.
16. Fill casing to top with sand.
17. Install protective cap or gatebox.
18. Flush piezometer with clear water.

Note: During the entire procedure of placing sand, bentonite and/or tamping, a tension should be kept on the piezometer leads. Care must be exercised to insure that the leads remain vertical and that kinking or breaking of the lead does not occur. Small diameter, rigid P.V.C. may be utilized in place of flexible P.V.C. to simplify the installation procedure.

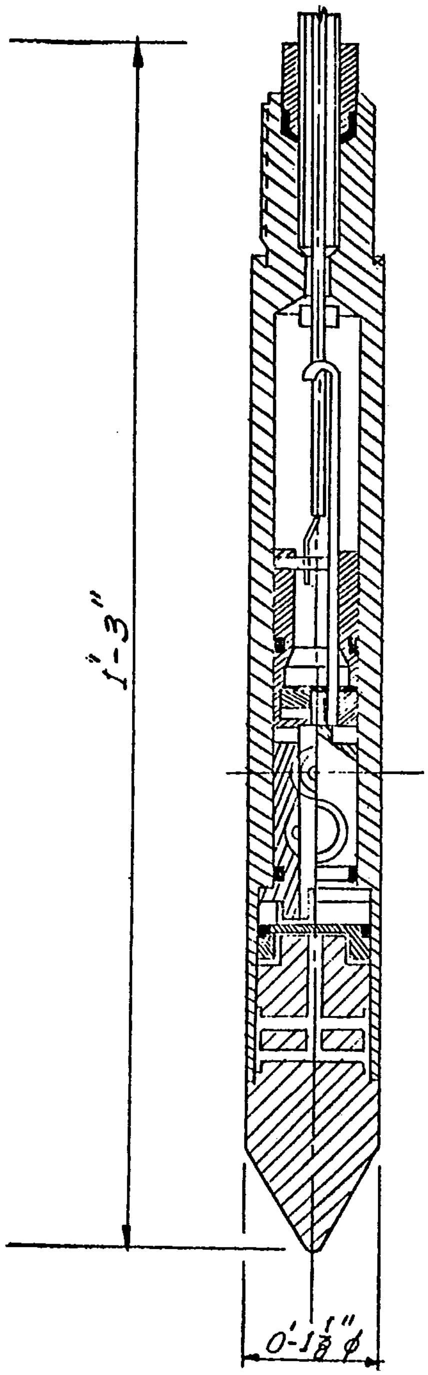
Piezometers Installed in Completed Boreholes (Casing Removed). The following section outlines the normal piezometer installation procedure. These procedures may be modified to adapt to field conditions.

1. After completing all soil and rock testing and sampling, wash or flush the borehole until the wash water is clean.
2. Lower the tamping hammer to the bottom of the hole; record the depth and mark the depth on the cable using friction tape.
3. Pour sufficient Ottawa sand into the hole to fill about .3m (1 ft.) of the borehole.
4. Lower the hammer and rest the bottom of the hammer on the sand; record the depth; raise the hammer.
5. Assemble the piezometer tip and polyethylene tubing providing about ten feet of excess tubing; measure the length of tubing to the bottom of the piezometer unit.
6. Tie the sash weight to the tubing so that the weight hangs below the piezometer unit; use a wire or nylon cord bridle.
7. Connect the open end of the polyethylene tubing to the flushing tank; lower the piezometer tip until the weight rests on the top of the sand.
8. Leaving the tubing connected to the surge tank, lower the piezometer tip until the weight rests on the top of the sand. *Hold the polyethylene tubing to prevent twisting or kinking during remaining operations.*
9. Pour sufficient Ottawa sand around the tip until the sand is about .6m (2 ft.) above the top of the piezometer. Permanently disconnect the surge tank and check the height of the sand using the tamping hammer and the reference mark on the cable.
10. When the sand is at the proper level, tamp the sand with 20, 15 cm (6 in.) blows of the hammer.
11. Place a 15 cm (6 in.) layer of dry bentonite pellets; predetermine the required volume.
12. Place a 7.5 cm (3 in.) layer of concrete sand over the bentonite.
13. Tamp the sand with 20, 15 cm (6 in.) blows of the hammer.
14. Repeat the procedure to construct a minimum seal length of .6m (2 ft.). Record the depth to the top of the seal.
15. Fill the hole with approximately .6 m (2 ft.) of concrete sand.
16. Tamp the sand as described in step 10 and then construct another .6m (2 ft.) seal.
17. Backfill with concrete sand and construct bentonite seals at 3m (10 ft.) intervals until the hole is filled to the level of the next piezometer installation. (If required)
18. Complete the installation of any shallow piezometers following the procedures in steps 3 through 16. Remove casing as required.
19. Backfill with concrete sand and construct bentonite seals at 3m (10 ft.) intervals until the hole is filled to the level of the bottom of the observation well installation. Remove casing as backfilling proceeds.
20. Place a Buffalo Roadway Box or other protective casing flush with existing ground surface and motor into place if required.

Note: Small diameter rigid P.V.C. may be utilized in place of flexible P.V.C. to simplify the installation procedure.



PIEZOMETER INSTALLATION DETAILS
(Not to scale)



GEONOR VIBRATING WIRE PIEZOMETER
NO SCALE

Piezometers Installed by "Insertion" into Cohesive Soil

1. Only those piezometer tips which are especially constructed and recommended by the manufacturer to be installed in this manner shall be inserted into the ground by the procedures described herein.
2. Leads shall be attached to the piezometer tip and, if polyethylene, the leads may be spliced together with compression type fittings. Casing or pipe shall be attached to the piezometer tip and extend for the full length of installation. The leads shall be threaded through the casing. The bottom 3 m (10 ft.) of casing shall not contain any external couplings, and the connection from the piezometer tip to the casing should be a smooth flush connection without any external projections.
3. Attach at least 3 m (10 ft.) of casing to the piezometer tip and push into the ground with a constant steady motion without twisting. Add additional lengths of leads and casing as required and advance to the final tip elevation.
4. On deep installations or where granular soils overlie the clay and it is difficult, if not impossible, to push the tip to the required elevation, an open hole may be made to within 3 m (10 ft.) of the final tip elevation using any adequate procedure. The assembled tip and casing shall then be lowered into the hole and pushed with a constant steady motion without twisting to the final required elevation.
5. If the piezometer tip is hydraulic, the tip and leads should be filled with water, flushed and checked for leaks prior to installation. Also, a slight water head should be applied by means of a surge tank to the tip throughout the installation process to prevent the tip from closing. No pressure, however, should be applied during the final few feet of penetration.

EXPLORATORY PROBES
HAND PROBES

Purpose

Hand probes are made to obtain reconnaissance information in swamp areas, concerning the thickness and lateral extent of soft, compressible cohesive soils. Small-diameter steel rods are pushed by hand to refusal and/or underlying inorganic soil.

Equipment

1. Rods, probe, small-diameter, flush-coupled, sectional (length depends on anticipated conditions)
2. Wrenches, pipe
3. Jack (optional)
4. Tape, cloth
5. Stakes and flagging
6. Rubber gloves and boots
7. Ruler
8. Map, topographic
9. Notebook, field
10. Crew, minimum of two field personnel

Procedure

1. Locate probe sites on available survey control
2. Assemble the probes in appropriate, portable lengths. Generally, do not exceed 5 m (16.5 ft.) sections.
3. Push the probe, by hand, into soil to refusal and/or underlying inorganic soils.
4. When individual samples of organic soil are desired, a 2.5 cm (1 in.) O.D. retractable soil sampler may be attached to the end of the probe rods.
5. Record the depth of penetration and the nature (if determined) of the underlying inorganic soils. Determine soil thickness to the nearest 15 cm (6 in.). Record the depth of standing water, if any.
6. It may be necessary on occasion to obtain several probes in the same location to determine if the refusal encountered is a buried stump or boulder. Be alert for this possibility.
7. Remove the probe from the ground, using pipe wrenches, as soon as possible to avoid buildup of soil friction.
8. Occasionally, due to cohesiveness of the soil and/or depth of the probes, it is necessary to remove probe rods by jacking, utilizing a small portable hydraulic jack.
9. Locate each probe on the plan of subsurface explorations; indicate probe depth. If probe sites are to be located during a later survey, drive a stake at each location suitably marked.
10. Complete Test Probe Summary form.

AIR PERCUSSION PROBES

Purpose

Air-operated percussion drilling may be employed to supplement more conventional subsurface exploration techniques to obtain additional information on the depth of relatively shallow bedrock. The track-mounted varieties can be employed in areas of relatively difficult access, or where very dense, "boney," sands overlie bedrock. This equipment may be employed in conjunction with an acoustical listening device to more accurately define the subsoil and rock conditions.

Equipment

1. Drill rig, air-track percussion, and operator
2. Compressor, pneumatic
3. Drill rods
4. Drill bits (available in several sizes)
5. Ruler
6. Field book

Optional Acoustical Equipment

1. Indicator, electronic noise level (Geomonitor)
2. Probe, amplification, and extension cord (*fully* charged)
3. Casing, p.v.c. (minimum 4 cm (1.5 in.) I.D.)
4. Earphones
5. Stop watch
6. Water container

Procedure

1. Connect the selected percussion bit to the drill rods and commence vertical drilling in approximately 3 m (10 ft.) increments.
2. Record the rate of penetration of the drill stem (per 30 cm (12 in.)) under a constant drill feed pressure. Observe and record the character of the drill cuttings as they come to the surface.
3. Add additional lengths of drill rod as required.
4. The penetration rate will be quite rapid and variable through overburden materials. The penetration rate will be considerably slower and quite constant in massive boulders or bedrock.
5. Penetrate a minimum of 3 m (10 ft.) into what is believed to be bedrock to ascertain if it is a massive boulder. This depth may be increased if larger boulders are anticipated.
6. The driller's expertise is invaluable in determining the type of materials being penetrated.
7. Record all data on Test Probe Report form.

Optional Procedure Using Acoustical Equipment

1. Drill a listening hole into bedrock following the procedures outlined above. Insure that the listening hole terminates at a minimum depth of 3 m (10 ft.) into bedrock.
2. Place minimum 4 cm (1.5 in.) I.D. casing (if required) to prevent collapse of the listening hole.
3. Fill casing and hole with water.
4. Lower the amplification probe into the listening hole to approximately 30 cm (12 in.) above the bottom of the borehole.
5. Connect the probe extension to the noise-level amplifier (Geomonitor).
6. Connect earphones to noise-level amplifier. Refer to Fig. A for sketch of test set up.
7. Commence additional percussion probes with air track equipment in the vicinity of the listening device. Percussion equipment should be a minimum distance of 6 m (20 ft.) from the probes. Maximum distances will vary depending on soil and rock conditions.
8. Record rate of drill penetration 30 cm (12 in.) and decibel readings from the noise level indicator.
9. Observe and record the character of the drill cuttings as they come to the surface.
10. When bedrock is encountered, a major difference in penetration rate and noise level will be noted. Terminate exploration at this point and move to new location. As the distance from the drill rig and listening hole increases, the noise level will decrease. Drill new listening holes as required to be able to insure reliable readings.
11. Record all data on Test Probe Report form.

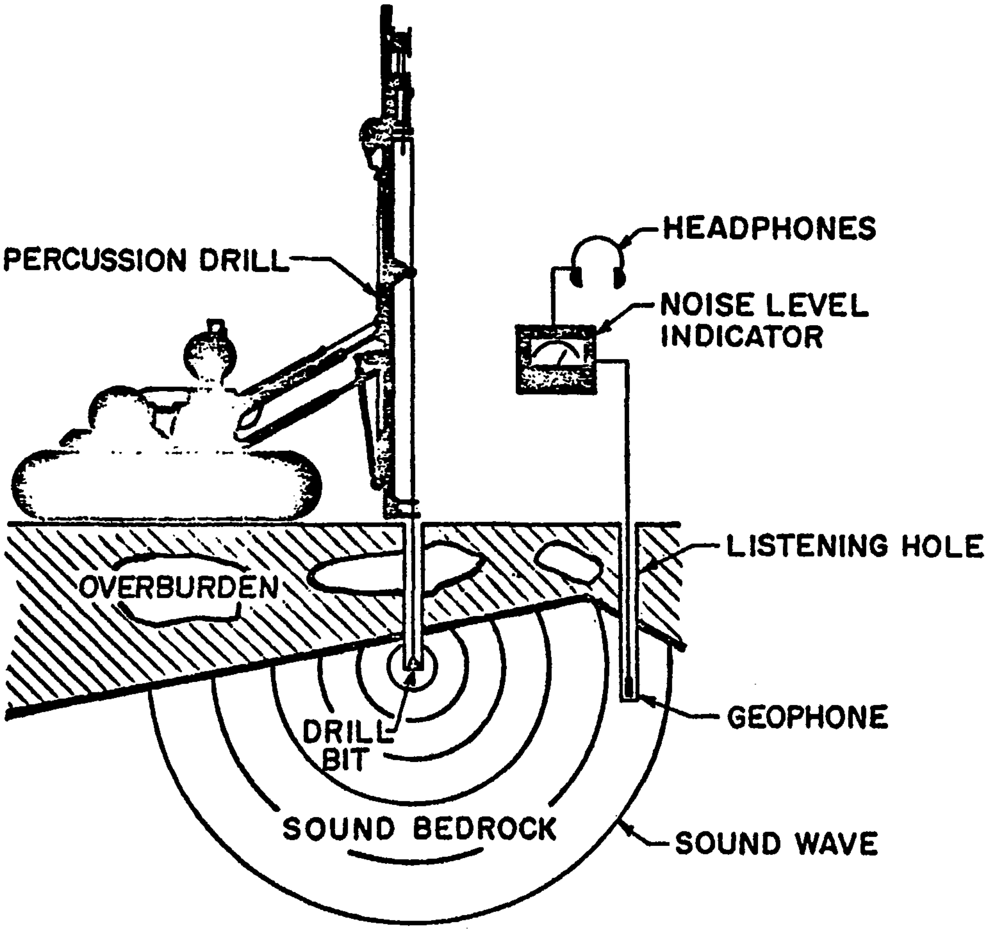
EXPLORATORY TEST PITS

Purpose

Test pits and trenches may be excavated by hand or by conventional earth-excavating equipment to provide detailed examination of near-surface geological conditions. The technique is utilized for such purposes as determining the presence of faulting, geologic contacts, preliminary slope stability estimates, and recovery of bulk samples for laboratory testing. Information to be obtained and depth required will dictate the type of equipment and procedures to be employed.

Equipment

1. Excavator, machine-type (size and type dependent on depth and accessibility)



AFTER LUNDSTRÖM AND STENBERG (1965)

COMPONENTS USED FOR
ACOUSTIC SOUNDING TECHNIQUE

2. Shovel, hand
3. Tape, cloth
4. Ruler
5. Bags, sample with labels
6. Stakes and flagging
7. Pen, indelible, waterproof
8. Site plans and forms
9. Pump, dewatering (optional)
10. Sheeting and bracing (optional)
11. Barricades (if open overnight)
12. Compaction equipment (if required for backfilling purposes).

Procedure

1. Locate the excavation to the nearest 30 cm (12 in.). Determine the presence or absence of any underground utilities before commencing the excavation. This may require preliminary investigations in association with various utility companies.
2. If the excavations are associated studies for a design structure, make all attempts to keep the excavations outside of the proposed foundation or limit the excavation as much as possible to avoid disturbance of foundation bearing materials.
3. Excavate the pit in a series of increments, examining closely the soil types and thickness of each stratum. The excavated soil shall not be placed closer than 60 cm (2 ft.) from the edge of the trench/pit.
4. Do not enter an unsupported pit that is excavated greater than a depth of 1.5 m (5 ft.)

below existing ground surface or any pit displaying evidence of instability such as headwall cracking or slumping. Installation of shoring will be in accordance with construction safety orders of the appropriate state and U.S. Occupational Safety and Health Act.

5. Record the size, quantity and type of all boulders and fill materials excavated from the pit.
6. Obtain representative samples as required. Note the presence or absence of groundwater or surface waters entering the pit; record the approximate rate of flow. If possible, leave the pit open for several hours to insure groundwater stabilization; relatively impervious soils may require several days before flow stabilizes.
7. Record the nature and character of any bedrock, including attitudes of bedding and discontinuities, as well as relative orientation (a supplemental sketch may be of value).
8. Record voids, stability and density of the materials encountered, and obstructions to excavation.
9. If the pit/trench is to remain open for longer than the day of excavation, a barrier will be constructed to bar inadvertant entrance of persons or domestic animals.
10. Backfilling of trenches should be bucket-tamped and rolled by equipment.

THIN-WALLED OPEN DRIVE SAMPLING

Purpose

Thin-walled, open drive samples, referred to as "Shelby tubes," provide samples of cohesive or granular material for laboratory classification and limited testing purposes.

Equipment

1. Thin-wall tube (seamless steel or brass, diameter 5–12.5 cm (2–5 in.))
2. Assembly, head
3. Screws, head cap
4. Wrench, Allen
5. Rods, drill
6. Bit, washing or chopping
7. Drill rig
8. Ruler, six-foot
9. Crayon, yellow keel

Procedure

1. Advance the borehole to the required depth and wash thoroughly with a conventional washing bit, taking care not to disturb the material to be sampled.
2. Examine the thin-wall tube to determine that it is free of rust, dents or scratches. The cutting edge should be bevelled and drawn-in, approximately .0156 mm (1/64 in.) less than the outside diameter of the tube.
3. Pass a clean rag through the tube to remove accumulations of foreign matter.
4. Attach the thin-wall tube to the head assembly and drill rods.
5. Lower the sampler assembly to the required depth and press or drive the sampler 61 cm (2 ft.) into the soil. If the sampler has to be driven, record the method and number of blows, per 15 cm (6 in.) of penetration.
6. To insure good recovery, leave the assembly in the borehole for 10 to 15 minutes, to allow buildup of skin friction within the thin-wall tube. Then rotate entire assembly 1 or 2 revolutions to shear off sample from soil below.
7. Withdraw assembly from the borehole and disassemble.
8. Remove any disturbed material from the tube ends and measure the recovery.
9. Seal, mark and store the tube as described under Section 6.9, Sample Preservation and Shipment.

MECHANICAL STATIONARY PISTON SAMPLING

Purpose

Stationary piston samples provide undisturbed samples of potentially sensitive soils for detailed laboratory testing.

Undisturbed samples may be obtained in granular soils. The procedure outlined below (cohesive soil) is identical with the exception of using a heavy drilling "mud" to insure recovery and to maintain the sides of the uncased borehole. Extreme care must be exercised during handling of the sample and it *must be* examined in a vertical position at all times.

Equipment

1. Thin-wall tube (seamless steel or brass, diameter 5–12.5 cm (2–5 in.))

2. Assembly, piston, rod and head, including cone and spring
3. Rods, actuating
4. Assembly, pull-down ("Christmas Tree") (optional)
5. Locks, coupling
6. Pliers, "Vise-grip" type
7. Screws, head cap
8. Wrench, Allen
9. Auger, shielded, jet-type, clean-out
10. Drill rig
11. Ruler
12. Crayon, yellow keel

Procedure

1. Advance the borehole to required depth by washing with shielded, jet-type, clean-out auger. Turn off the wash water and rotate the auger by hand the last 7.5 or 10 cm (3 or 4 in.) to the required depth to clean out disturbed sludge. When sampling in very soft plastic soils, or loose granular soils, the casing should be kept full of water or drilling mud at all times.
2. Examine the thin-wall tube to determine that it is free of rusts, dents or scratches and that the cutting edge has been bevelled and drawn-in approximately .0156 mm (1/64 in.) less than the outside diameter of the tube.
3. Maintain casing full of water to insure good recovery and to minimize sample disturbance during all phases of testing.
4. Pass a clean rag through the tube to remove accumulations of foreign matter.
5. Insure all vents in the piston head assembly are clean, and the leather packing around the piston head is soft and pliable and that the piston fits tightly within the tube.
6. Assemble piston sampler making sure that the piston is flush with the cutting edge of the tube, and is secured against downward travel by the cone and spring assembly.
7. Connect required length of drill rods with actuating rods inside. Actuating rods should extend above the top of the drill rods. Attach sampler assembly to inner actuating rods and outer drill rods. Lock actuating rods, by means of lock coupling, to drill rods and mark actuating rods so that any movement of the piston may be detected as the assembly is lowered into the borehole.
8. Lower the sampler assembly to the required depth. Secure the actuating rods to the casing by means of the rod brace and clamp assembly. Connect pulleys and gin lines to the block and fall on the derrick. The "Christmas Tree" Assembly may not be required for pressing the tube assembly if other types of equipment are available (example: large hydraulically operated drill rig).
9. Mark the drill rods at the point where sample penetration should stop (typically 61 cm (24 in.)).
10. Press sampler into soil at a uniform rate on the order of 30 cm (12 in.) per second without rotation.
11. Observe the casing closely during the pressing operation to insure that the casing is not coming up. If this should occur, questionable test results may be obtained, and the occurrence should be noted. (If a large hydraulic drill rig is used as the power source, this will not be a problem or concern).
12. To insure good recovery, leave the assembly in the borehole for 10 to 15 minutes to allow buildup of skin friction. Then rotate 1 or 2 revolutions to shear off sample from soil below.
13. Disassemble "Christmas Tree" (if applicable) and remove sample from borehole with care.
14. Unscrew sampler head assembly, release the vacuum and remove the piston, taking care not to disturb the material. Clean any disturbed material from the tube and measure the recovery.
15. Seal and mark the tube as described under Section 6.9, Sampling Preservation and Shipment.

HYDRAULIC PISTON SAMPLING

Purpose

Hydraulic piston samples are obtained in uncased boreholes to provide undisturbed samples of potentially sensitive soils for detailed laboratory testing.

The hydraulically operated piston assembly, because of its compact and portable nature, makes it an ideal tool in swamps and areas of difficult access where larger conventional drilling equipment are inaccessible. There is a disadvantage, however; its ability to penetrate only relatively soft, cohesive materials.

Equipment

1. Assembly, hydraulic piston (complete unit such as Osterburg)

2. Tube, thin-wall (seamless steel or brass)
3. Auger, jet-type, clean-out, shielded
4. Rods, drill (AW or larger)
5. Device, rod, locking
6. Tank, compressed air, and pressure valve (p.s.i.) (Depending on type of unit, water or drilling mud, may be used to activate the hydraulic assembly.)
7. Air hose
8. Drill rig
9. Ruler
10. Crayon, yellow keel
11. Bentonite (for drilling mud)

Procedure

1. Advance the borehole to the required depth by washing with a shielded-jet, clean-out auger. The hydraulic sampler may also be employed in advancing the borehole. The borehole may have to be stabilized with drilling mud to maintain its integrity.
2. Examine the thin-wall tube to determine that it is free of rust, dents or scratches and that the cutting edge has been bevelled and drawn-in approximately .0156 mm (1/64 in.) less than the outside diameter of the tube.
3. Pass a clean rag through the tube to clean out accumulations of foreign matter.
4. Inspect the hydraulic assembly to insure that it is clean and the piston head packers are soft and pliable and that the piston fits tightly within the tube.
5. Connect the thin-wall tube to the hydraulic piston assembly, insuring that the piston is flush with the bottom of the thin-wall tube.
6. Connect the required amount of heavy drill rods and lower the assembly to the required sampling depth.
7. Lock the unit to the drilling tower or casing to prevent further downward movement of the assembly.
8. Mark the drill rods at the point where sample penetration should stop (61 cm (24 in.)).
9. Connect the reinforced pressure hose to the top of the drill rods and to the compressed air or fluid power source.
10. Open the pressure valve which will activate the assembly, and advance the thin-wall tube into the soil. The assembly will automatically stop at the completion of the 61 cm (24 in.) press. Shut off pressure valve.
11. To insure good recovery, leave the assembly in the borehole for 10 to 15 minutes to allow build-up of skin friction within the thin-wall tube. Then rotate entire assembly 1 or 2 revolutions to shear off sample from soil below.
12. Withdraw assembly from the borehole and disassemble. It may be necessary to cut a small slot near the top of the tube with a hacksaw to release the pressure build-up to facilitate removal of the piston from the thin-wall tube.
13. Clean out any disturbed material from the tube and measure the recovery.
14. Seal and mark the tube as described under Section 6.9, Sample Preservation and Shipment.

DENISON SAMPLING

Purpose

The Denison Sampler is designed to recover relatively undisturbed, thin-wall tube samples, in partially-cemented sediments (such as glacial tills, sands and gravels, hard clays, weathered and soft bedrock). The sampler consists of a double-tube core barrel with a non-rotating inner thin-wall sampler tube. In general, permeable and course granular soils cannot be sampled with the Denison Sampler unless the soil at the sampling depth is in a frozen state.

Equipment

1. Sampler, Denison (four sizes: 9–19.5 cm (3.5–7.75 in.))
2. Bit, core, various types available: Diamond (hard formations), Carbide (soft formations)
3. Tube, inner, thin-wall (four sizes: seamless steel or brass)
4. Retainer, core; split ring (hard formations); basket spring (soft formations)
5. Auger, clean-out
6. Rods, drill, NW or larger
7. Wax, microcrystalline, and melting container
8. Mud, drilling (optional, depending on formation and drilling procedures)

Procedure

1. Advance the borehole to required depth, using conventional drilling techniques.
2. Assemble Denison Sampler with inner thin-wall tube, required core bit and appropriate core retainer. The core bits are available in different pre-set lengths to protect the sample tube, and are selected based on the density of the material being penetrated.

3. Prior to sampling in cohesive soils clean out borehole using a shielded jet type clean-out auger.
4. Lower sampler to required depth by adding lengths of NW drill rods.
5. Adjust rate of advance and revolutions of the core barrel in accordance with bit type and nature of formation being sampled (driller's expertise).
6. Upon attaining the required penetration, retrieve sampler and remove inner sample tube and preserve as described in Section 6.9, Sample Preservation and Shipment.

PITCHER SAMPLING

Purpose

The Pitcher Sampler is a patented device designed to recover samples from formations that are too hard or brittle for standard sampling equipment or too soft or water-sensitive for use with core-barrel-type samplers (such as Denison). The sampler consists of a single-tube core barrel with a self-adjusting, spring-loaded, inner sample tube which can project as far as 15 cm (6 in.) ahead of a bit which cuts through the soil mass to isolate the cylinder to be sampled.

Equipment

1. Sampler, Pitcher (choice of four diameters and two lengths)
2. Bit, core (various types; most are sawtooth type)
3. Tube, inner, thin-wall (choice of four diameters and two lengths; seamless steel or brass)
4. Rods, drill, NW-size
5. Water supply

Procedure

1. Inspect the cutting bit for serviceability
2. Observe placement of the sampling tube on the core barrel in retracted position. If the tube will not retract into the barrel, replacement of leather seals is indicated.
3. Advance the borehole to required depth using conventional drilling techniques.
4. Insure that the borehole is thoroughly cleaned of cuttings and sediment.
5. Lower the sampler to required depth by adding appropriate lengths of NW drill rod.
6. Adjust the rate of advance and revolutions of the core barrel in accordance with the type of bit and nature of the formation being sampled (driller's expertise).
7. Upon attaining the required penetration, retrieve the sampler, being careful to hold loose sands within the barrel.
8. If an obstacle hinders advancement, remove the sampler and make the appropriate entry on the log. Continue to advance the boring past the obstruction.
9. Remove sample tube, drain free water, trim off excess length of tube, place "cap plugs" on either end, apply sealing tape, mark the upward end with an arrow pointing toward original ground surface, and seal over cap plug and tape by immersing each end in hot sealing wax.
10. Store the sample tubes in the vertical-upward position. Place cushioning material under and around each tube.
11. Follow the provisions of Section 6.9, Preservation and Shipment of Samples.

APPENDIX B
In Situ Borehole Testing

B.1 GENERAL

In situ borehole tests are performed to determine the various properties of soil or rock formations that are needed to conduct geotechnical analyses. In general, three classes of properties are necessary: shear strength, deformation characteristics, and permeability. Traditionally, these properties have been determined by laboratory tests on undisturbed samples of cohesive soils, reconstituted samples of cohesionless soil, or representative rock core specimens.

In certain cases it may be technically more feasible to determine the required *in situ* properties by means of *in situ* borehole testing. Improvements in apparatus, instrumentation, measurement techniques and analysis procedures have led to the increased use and acceptance in recent years of *in situ* measurement techniques.

The primary advantages of in-situ borehole testing are the ability to:

- Determine properties of soil that cannot easily be sampled.
- Avoid sample disturbance, improper stress states, and changes in physical and biological environment that may influence laboratory testing.
- Test a volume of soil or rock that, in some cases, is larger than that which can be tested in the laboratory.

The above advantages have to be considered in light of costs and the following possible limitations:

- Uncertain empirical correlations between measured quantities and actual properties may exist.
- Flow (in permeability tests) and stress direction cannot be independently varied.
- Applied principal stress directions in the field test may differ from those in real problems.

B.2 SCHEDULING

Since much of the same equipment (such as drill rigs, sample rods, etc.) may be required for the field testing, *in situ* tests should be carried out whenever practicable, concurrently with the test boring program. Generally, *in situ* field tests, such as field vane, pressuremeter, and permeability tests, are performed in the same borehole that is used for logging and identification of subsurface strata. However, it is advisable to consider possible time delays due to field testing on actual production rates for test borings. For example, pressuremeter testing generally can be more efficiently conducted in a separate borehole due to the time required to perform an individual test and the special techniques necessary to stabilize the borehole walls.

Certain other tests, such as the cone penetrometer tests, do not involve the recovery of samples or the need for an open borehole. These tests, therefore, have to be conducted at locations separate from test borings.

As stated above, it may be advantageous in terms of cost and time, to perform field tests in conjunction with the subsurface exploration program. However, in project areas where little is known about subsurface conditions, it may not be possible to evaluate the need for and types of field tests required prior to some field exploration work. In these situations, it may be desirable to conduct the subsurface exploration program in phases. The initial or preliminary phase may involve a minimum number of subsurface explorations to define the general nature of the site conditions. These initial explorations may consist of standard sampled test borings or non-sampled, *in situ* field tests, depending on local geologic conditions and practices. Additional field test requirements cna be assessed after the preliminary program is completed and be incorporated into subsequent phases of the subsurface exploration program.

B.3 TYPES OF TESTS

The purpose of performing *in situ* tests is to determine the shear strength, deformation characteristics, and/or permeability of the soil and rock. Certain types of field tests may be used as index tests; that is, tests which do not measure directly the soil or rock property of interest, but which can be empirically correlated with a given property. These tests will be referred to as correlation tests; the standard penetration test is an example. The penetration resistance (blow count) obtained from this test can be correlated to such properties as relative density, bearing capacity, and liquefaction potential.

Certain tests may be used to determine more than one parameter. For example, the cone penetrometer may be used to determine both the shear strength and the deformation parameters of a given soil deposit. Field *in situ* borehole tests will be grouped into three broad categories, as follows:

- Correlation Tests
- Strength and Deformation Tests
- Permeability Tests

Specific tests that will be discussed in each of these categories are:

1. Correlation Tests
   a) Standard Penetration Test (SPT)
   b) Dynamic Penetration Test (DPT)
2. Strength and Deformation Tests
   a) Penetrometers
      1. Cone Penetrometer Test (CPT)
      2. Piezocone Penetrometer Test (PQS)
   b) Pressuremeters (PMT)
      1. Menard Pressuremeter Test
      2. Self-Boring Pressuremeter Test
      3. Dilatometer
   c) Stress or Shear Devices
      1. Hydraulic Fracturing Test
      2. Vane Shear Test
      3. Borehole Shear Test
3. Permeability Tests
   a) Water Pressure Tests
   b) Pump Tests
   c) Hydraulic Conductivity Tests
   d) Percolation Tests

The general details of each of these tests are discussed in the following subsections. Where appropriate, specific equipment requirements and test procedures are presented in Appendix C. Where standard specifications (such as ASTM or AASHTO) exist for a given test, they are referenced in Appendix C.

B.4 CORRELATION TESTS

B.4.1 Standard Penetration Test

The Standard Penetration Test (SPT) is a widely used field test in subsurface exploration programs. Details of the SPT are discussed in Appendix C. Briefly, the test consists of driving a split-barrel sampler (split spoon) into a soil deposit with a hammer of known weight and recording the number of blows (blow count, N) required to drive the sampler 30 cm (12 in.). For uniformity, the sampler should be 51 mm (2 in.) O.D. and 35 mm (1.375 in.) I.D. The standard hammer weights 64 kg (140 lbs.) and the height of fall is 75 cm (30 in.). Except for unusual cases, these standards should be adhered to rigorously. Refer to Appendix C for the use of alternate hammer sizes and weights.

There are other penetration type tests or soundings. An advantage of the Standard Penetration Test over the other types is that the hole is cased or otherwise supported down to the depth at which the sampler is driven so that side friction along the drill rod does not influence the results. Another important advantage is that a sample is recovered which can be visually classified.

The results of this test in terms of blow count ("N" value), are used to describe the density of cohesionless soils and the consistency of cohesive soils as discussed in Appendix E. They thereby provide a method for determining the uniformity of a given soil deposit and identify changes in soil strata. The blow count has also been correlated with such engineering parameters as relative density and bearing capacity of granular soils, shear strength of cohesive soils, and liquefaction potential of fine sands.

Refer also to Section 7.6.2 for additional information regarding SPT.

B.4.2 Dynamic Penetration Tests

These tests consist of driving, by hammer impact, a probe into a soil stratum and recording the number of blows required to achieve a pre-determined amount of penetration. They are similar to the SPT except that a probe point is used instead of a split-sampler barrel and no sample is recovered. These tests are not to be confused with the penetrometer tests which are discussed in Section B.5.1, in which the penetrometer is pushed, not driven, into the soil at a constant rate.

Information from these dynamic penetration tests can be used as a relative index of granular soil density or the consistency of cohesive soil. However, specific empirical correlations such as those available with the SPT are not known or widely available. The principal use of these tests is, therefore, to provide an indication of the uniformity or consistency of a given soil stratum, either with depth in a given borehole or between probe locations. Caution must be used in interpreting the data as soil samples are not obtained. These field tests should be used to supplement, not replace, other tests such as the SPT.

These tests may also be useful in locating the bedrock surface, but should be used in conjunction with cored borings to determine if boulders may be present to influence the test results.

There are basically two types of dynamic penetration tests. In one, the cone point is attached to drill rods and soundings are made at intervals in a test hole. After each sounding, the drill hole is advanced and another sounding is made after the drill hole is cleaned out. In the other type of test, the cone point is driven continuously from the ground surface to the required depth. At the completion of the test, the drill rods are extracted, the expendable point is left in the ground.

General procedures for conducting these tests are discussed in Appendix C. Basically, the cone is advanced by driving it with a known weight which is dropped a prescribed distance. These dimensions and weights vary from organization to organization. The number of blows to advance the penetrometer in 15 cm (6 in.) intervals should be recorded. For the retractable cone, the cone is generally driven a total of 30 cm (12 in.). It is important to note, that the number of blows is influenced by the general material type. In granular soils, the number may be significantly greater for the second 15 cm (6 in.) than for the first, whereas in clays, the numbers may be about the same.

Friction penetrometers are also available and their use is similar to that of a cone penetrometer, except that they are used to measure frictional resistance. The average friction (shear) above a given point can be calculated from the pulling force and from the surface area of the penetrometer rod.

Numerous other variations in the dynamic penetration tests exist; their general operating principals and use are the same as those discussed above. They should be used with caution, bearing and in mind the limitations discussed herein.

B.5 STRENGTH AND DEFORMATION TESTS

B.5.1 Penetrometers

B.5.1.1 Cone Penetrometer Test (CPT). The Code Penetrometer Test (CPT)(is a quasi-static, sounding method used to obtain the *in situ* soil bearing capacity and side friction components of penetration resistance. Several types of penetrometers and methods are used throughout the world (Sanglerat, 1972). However, the Dutch have experimented with these techniques since the early 1930s and the CPT is often referred to as the "Dutch Cone Test". The CPT has been used in the United States since the mid 1960s. ASTM has published test procedures (D3441) for the CPT.

The CPT is most useful in permeable, coarse to fine sands where the effects of pore pressure are negligible. Under these conditions, the CPT measures drained behavior. However, the CPT measures undrained behavior when penetrating homogeneous plastic clays. Table B-1 summarizes the advantages and disadvantages of the CPT.

Briefly, the CPT consists of pushing a cone-shaped steel point into the ground with a slow, constant speed, generally 1 to 2 cm/sec (2 to 4 ft/min.). The thrust required to accomplish this, divided by the projected end area of the point gives the *cone bearing capacity (qc)*. Recent additions of the friction sleeve near the point provides a similar measurement of *local sleeve friction (fs)*. To date, very little information is available which explains the theoretical problem of a passing penetrometer tip as it moves through soil (Durgunoglu and Mitchell, 1975). Many variables, such as soil density, cementation or vertical effective stress and test equipment procedures such as shape of penetrometer tip or method of penetration,

Table B-1
Advantages and Disadvantages of Cone Penetration Test

Advantages

- Provides a continuous or near-continuous record of data.
- Data are of the *in situ* or undisturbed variety and, hence, better suited for solution of geotechnical design problems.
- Speed of test.
- Economical.

Disadvantages

- Does not retrieve a soil sample during testing. (Note: the cone tip, however, can be replaced with a soil sampler.)
- Requires time to reduce and interpret data.
- Has limited depth capability, particularly in very dense sands, cemented strata, or glacial till.
- Obstructions such as boulders generally result in termination of the test.

influence the interpretation of CPT data (Schmertmann, 1978).

Equipment necessary to conduct a CPT consists of a cone, push rods, measuring equipment, and thrust/reaction equipment, as discussed below:

- *Cone:* The penetrometers are equipped with either non-friction or friction-cones of both the mechanical or electrical types. The more common cone types are a Begemann Mechanical Friction-Cone and a Fugro (Netherlands) Electrical Cone. Mechanical penetrometers operate at incremental depths, generally 10 or 20 cm (4 or 8 in.), by extending a telescoping penetrometer tip. Electrical penetrometers permit continuous reading of both end-bearing and friction sleeve values; one at a time. The standard cone tips have a 60-degree apex angle and a projected end area of 10 cm^2 (1.55 in^2). Friction sleeve areas are generally 150 cm^2 (23.2 in^2).
- *Push Rod:* The penetrometer tip is advanced to test depths by thick-walled push rods. These rods are typically 1 m (3.28 ft) long and they are screwed together to form one continuous section as the test depth increases. The mechanical cone tip is extended during testing by an independent set of 1 m (3.28 ft.) inner rods, which fit inside the push rods.
- *Friction Reduction Rod:* The friction reduction rod, placed just above the cone penetrometer, consists of an enlarged diameter rod which bores a hole larger than the push rods, thus reducing accumulated rod friction and increasing the depth sounding capabilities.
- *Stabilizing Rod:* A stabilizing tube may be used to minimize buckling of lengths of unsupported push rods extending between the base of the thrust/reaction equipment and ground surface, if mounted on a truck body or drill rig.
- *Measurement Equipment:* The thrust required to advance the penetrometer during testing is measured at the ground surface. A hydraulic or electrical load cell or proving ring is typically used to measure the thrust required to activate mechanical penetrometers. Penetration resistance of electrical penetrometers is generally measured by force transducers attached to the cone tip and friction sleeve. The transducers are connected to a surface data recording system by an electrical cable passing through the push rods, after the inner rods are removed.
- *Thrust/Reaction System:* The thrust system advances the penetrometer and rod assembly into the ground. This system must be capable of advancing the penetrometer tip at a constant rate while the magnitude of thrust varies. Special equipment made for the CPT, has "hydraulic" thrust capabilities of 9 or 18 tonnes (10 or 20 tons). Conversion kits, such as manufactured by Hogentogler (1979) permits the CPT to be performed by the hydraulic push system available on standard drill rigs. This system, however, reduces maximum available thrust to generally less than 5 tons. Thrust system CPT equipment must be secured to a stable reaction system, such as the bed of a heavy duty truck.

As previously stated, the CPT data output consists of cone bearing capacity and local sleeve friction values. Based on empirical correlations, these soil parameters can be used as an indicator of soil type or to estimate the physical and engineering properties of soil or very soft sedimentary rock. Research by the University of Florida has attempted to use the cone bearing capacity and sleeve friction parameters to make separate predictions of end bearing and side resistance components of total pile capacity (Freed, 1973; Nottingham & Schmertmann, 1975). A comprehensive design manual prepared by Schmertmann (1978) for the FHWA provides guidelines for interpretation and use of CPT data.

As shown on Figure B-1, Schmertmann has prepared a correlation of soil type and CPT data for north-central Florida. Use of this chart requires the definition of the *friction ratio* which is a dimensionless term representing sleeve friction divided by cone bearing, and expressed in percent. CPT data have also been used as an estimate of engineering properties such as relative density, compression modules, angle of internal friction for sands, untrained shear strength, and degree of consolidation for clays.

In recent years, methods of CPT analyses have been gaining in acceptance in geotechnical engineering. The method for estimating settlement in sand incorporates a strain-influence factor and values of Young's modules as established by cone bearing capacity. In addition, the method accounts for the depth of measurement as well as settlement due to secondary creep.

In view of the extensive use of the Standard Penetration Test (SPT) in the United States, several attempts have been made to correlate SPT blow count values (N-values) with the cone bearing capacity (q_c). Schmertmann has provided both theoretical explanations and empirical ratios, the latter shown as Table B-2. Recently, Schmertmann (1979) has used stress wave dynamics to explain the q_c/N ratios measured in complementary site CPT probes and standard SPT borings.

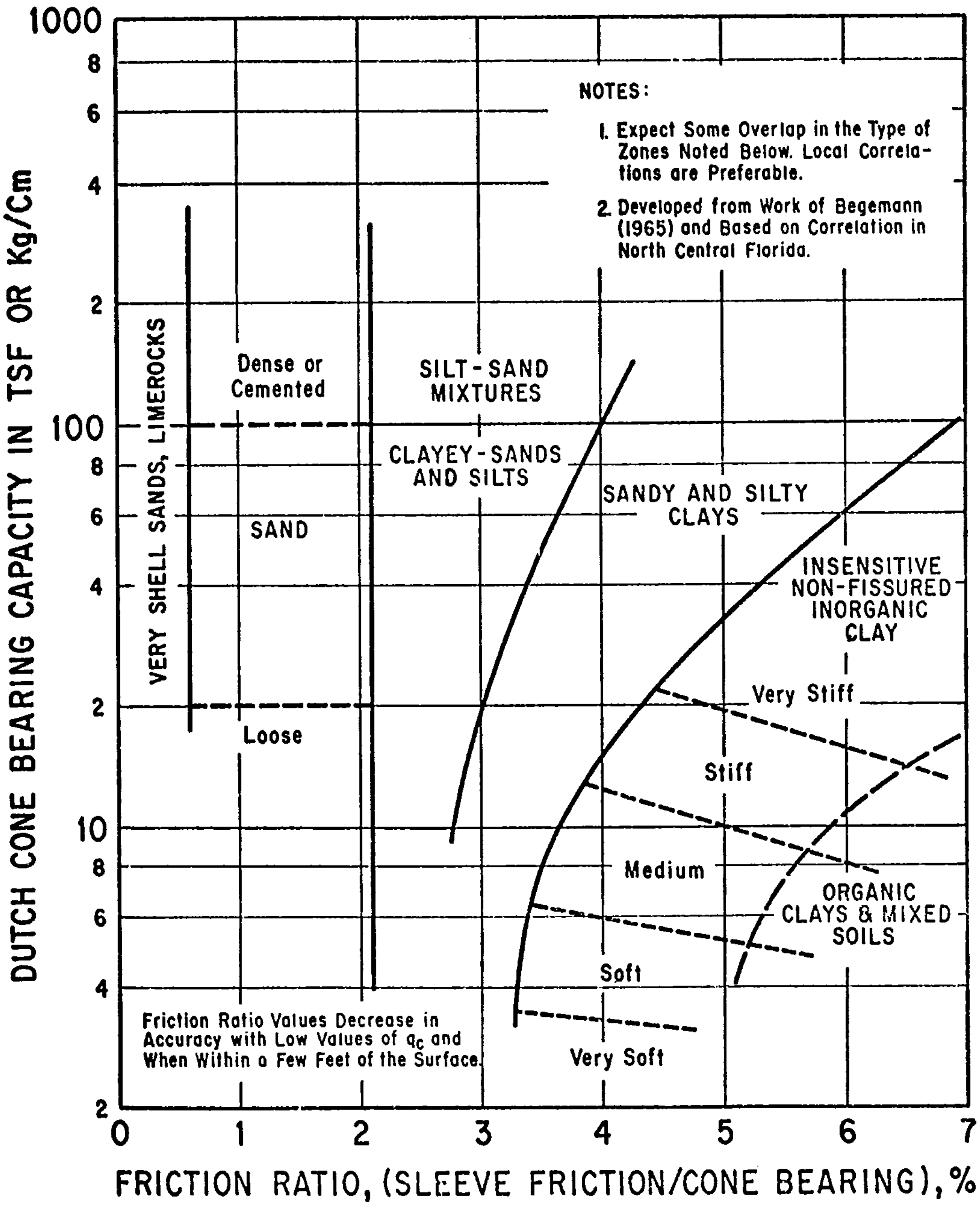


Figure B–1. Guide for estimating soil type from Dutch Friction-Cone Ratio (Begemann Mechanical Trip)

B.5.1.2. Piezocone Penetrometer Test (PQS). The Piezocone Penetrometer Test recently developed by Geotechniques International, Inc. (Piezocone), and similar equipment developed by the U.S. Army Engineer Waterways Experiment Station (PQS Probe), combine the functions of an electric friction cone and a piezometer probe. These modified penetrometers are capable of simultaneously measuring cone resistance, pore water pressure and skin friction during penetrometer advancement (Baligh et al., 1981; Franklin & Cooper, 1981). Penetration resistance is measured electrically as the axial load on the point and the shearing force on the friction sleeve. The pore pressure is measured through a porous filter element near the tip of the cone (Figure B-2).

The piezocone penetrometer contains a porous, stainless steel tip which is hydraulically connected to a pressure transducer for measuring the pore pressure. The force required to push the cone is measured by a load cell located behind the porous stone. The friction

Table B-2
Typical Ratios for qc(kg/cm^2)/N(SPT blows/ft)

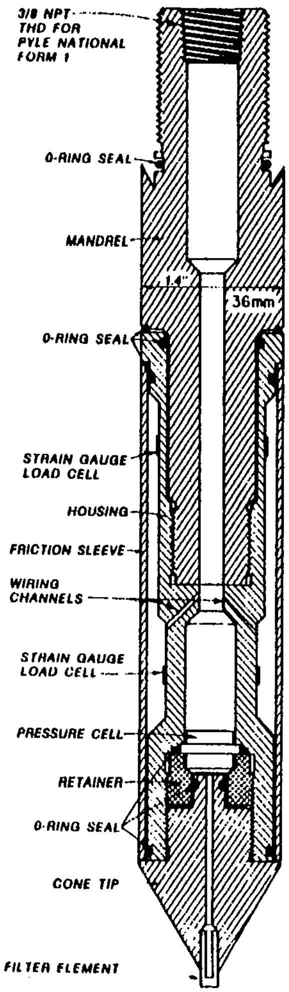
Type soil	Fugro* tip	Delft* mechanical tips
sand and gravel mixtures	5	6
sand	5	4
sandy silts	4	3
clay-silt-sand mixtures	2	2
insensitive clays	1	1.5
sensitive clays	ratios can become very high as N approaches zero	

* Both devices were developed in The Netherlands
Ref. Schmertmann, 1978, p. 20.

sleeve consists of a freely rotating, hollow cylinder which is equipped with a load cell for measuring friction force. The depth penetration is recorded as an electric signal. All data are displayed on a strip chart recorder and can also be recorded on magnetic tape for subsequent computer processing (Baligh et al., 1981).

The piezocone penetrometer is equipped with a protection device to eliminate overloading the load cell measuring cone resistance and complete saturation of the internal mechanism and porous filter of the probe is essential before obtaining field measurements.

Results indicate that the modified penetrometer-

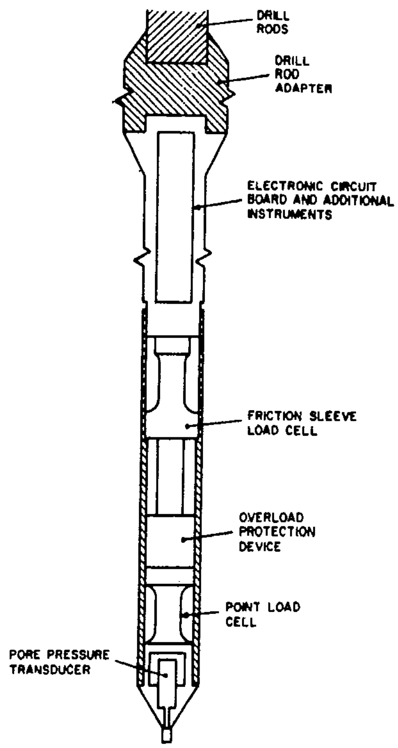


PQS Probe

U.S. Army Engineers

Waterways Experiment Station

(from Franklin & Cooper,1981)



Piezocone Penetrometer

Geotechniques International, Iac.

(from Baligh, et.al. 1981)

Figure B–2.

piezometer probe provides repeatable and reliable measurements and has proved to be extremely useful in the determination of soil stratification and identification. In addition, pore pressure dissipation measurements, when penetration stops, can be used to estimate consolidation and/or permeability characteristics of soils (Baligh et al., 1981).

B.5.2 Pressuremeters

In the Pressuremeter Tests, a cylindrical device is expanded radially against the sides of a borehole under increasing increments of pressure. The test results, expressed as pressure versus volumetric or radial strain, are then used in foundation design and engineering in both empirical and theoretical procedures. Since 1954, Louis Menard has promoted the development of the pressuremeter, with most of the work having been done in France (Baguelin, and others, 1978). Within the past 5 to 10 years, the pressuremeter has found increasing use in the United States.

Schmertmann (1975) lists the three following advantages of the pressuremeters as compared to other *in situ* tests.

- The test models the axisymmetric expansion of an infinite cylindrical cavity—which is a problem with well developed elastic and elastic plastic solutions, and with such theories apparently well suited to further development to better match the behavior of real soils.
- A properly conducted test may permit an estimate of the *in situ* horizontal stress.
- Data on the stress-strain properties of the soil, in addition to strength data, may be derived from the test, but the properties are applicable only to the direction perpendicular to the axis of the expanding cavity.

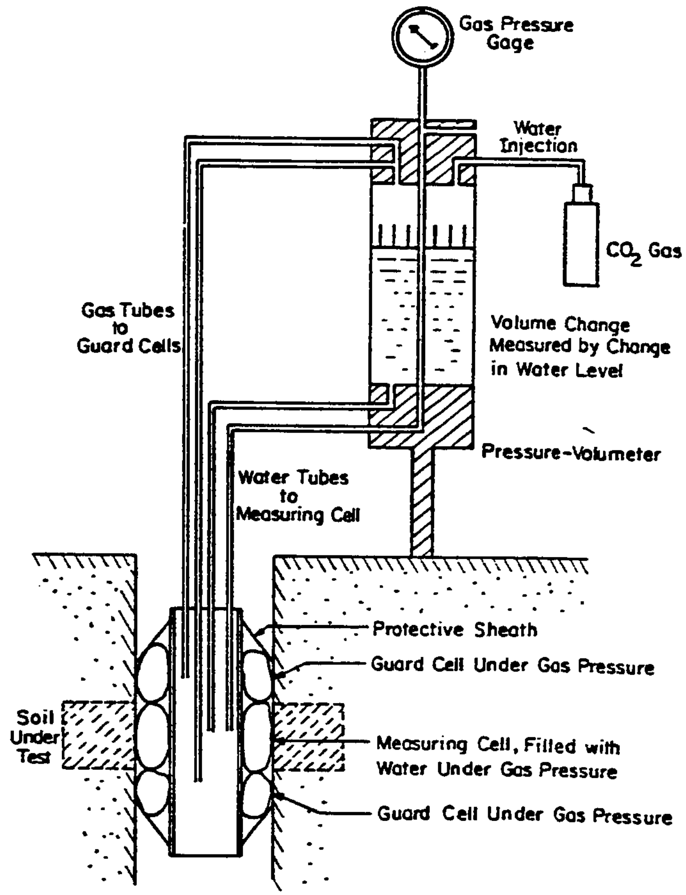
The pressuremeter is most useful in soils in which "undisturbed" sampling is difficult, such as soft silts, sensitive clays, interbedded or layered sands, silts and clays, and brittle soils such as glacial till or weathered rock. A disadvantage of the test is the uncertainty of soil drainage conditions in finer grained soils around the expanding cavity.

Two types of pressuremeters are in use today: the Menard type and the Self-Boring type. With the Menard pressuremeter, a cylindrical device is lowered down a pre-drilled borehole to the desired depth and expanded against the sides of the borehole. The recently developed Self-Boring pressuremeter drills its own hole and eliminates much of the borehole disturbance associated with inserting the Menard pressuremeter into a borehole. The Menard and the Cambridge (United Kingdom) Self-Boring pressuremeters are shown in Figure B-3. Additional types of Self-Boring pressuremeters, which are also identified as Dilatometers, include the French "Autoforeuse" (Baguelin et al., 1972 and 1978) and the Italian "Flat Dilatometer" (Marchetti, 1980). Discussion will deal with pressuremeters of the Menard design because they are most common in engineering practice today; Self-Boring pressurementers are still in the research and development stage.

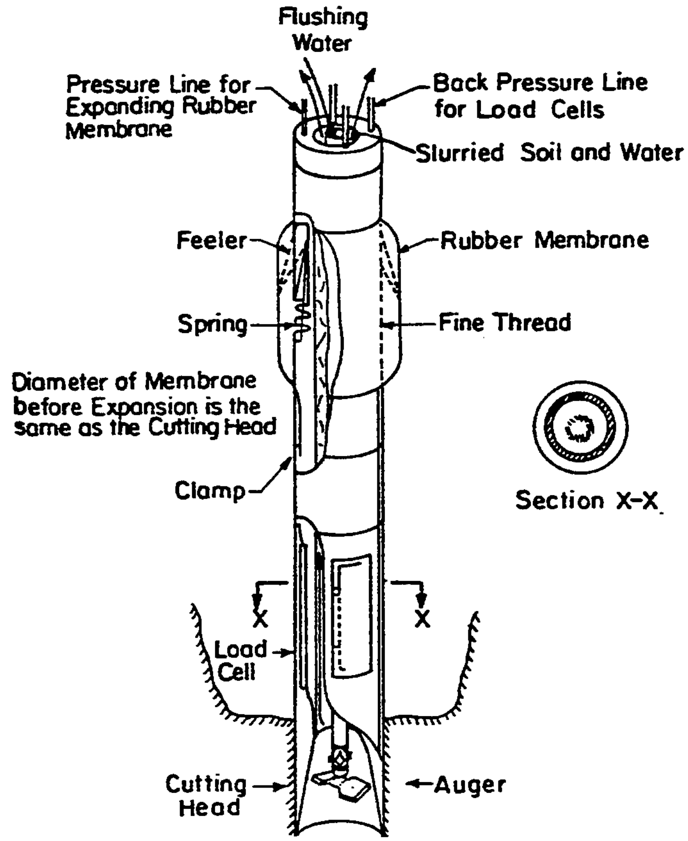
Pressuremeter apparatus consists of three components; a cylindrical probe, the control unit, and the tubing which connects the two. The cylindrical probe generally contains three expandable cells, a center measuring cell, and guard cells above and below it to isolate the measuring cell from probe and bore-hole and effects, thus maintaining plane strain conditions in the soil. The three cells are commonly surrounded by protective sheaths of polyurethane or metal strips. A recently developed pressuremeter by Oyo, eliminates the guard cell requirements and combines all the instrumentation into one component. The control unit is located on the ground surface near the borehole and is used to control pressure in the probe and measure volume change of the measuring cell. Water is generally used in the measuring cell. Volume change is monitored by reading the water level in a graduated sight tube called the "volummeter." Pressure is usually supplied by compressed gas applied to the fluid of the measuring cell and to the guard cells. There are pressuremeters in use which use spring-loaded feeler gauges to measure borehole expansion at the center of the probe (Mitchell and Gardner, 1975). Tubing between the control unit and the probe is usually coaxial to facilitate use of both water and gas pressures.

B.5.2.1 Menard Pressuremeter. The following general procedure is followed for the Menard-type pressuremeter test. The pressuremeter test using Menard-type apparatus is performed in a pre-drilled borehole. Drilling procedures should minimize disturbance to the sides of the borehole at desired test levels. The French use hand-auger procedures whenever possible, with drilling mud to keep the borehole open (Schmertmann, 1975). Other drilling procedures using mechanical equipment are commonly used in the U.S. In soft clays, thin-walled tube samplers are often used to prepare the borehole. The ratio of borehole diameter to probe diameter should be only slightly greater than 1.0 to ensure a snug fit and minimize spurious test results (Kastman, 1980).

During the test, pressure is applied via the compressed gas to the cells of the probe. Pressure is held



Menard Pressuremeter Apparatus (from Mitchell, et al, 1978)



Cambridge Self-Boring Pressuremeter Device (from Wroth and Hughes, 1973)

Figure B–3. Diagramatic views of Menard-type and Self-Boring Pressuremeters.

constant for 60 seconds at each desired pressure, and volume readings made and recorded 15, 30, and 60 seconds after the pressure is attained. The pressure may then be increased or decreased to the next desired level and the process repeated. Unloading and reloading cycles may be included in the test program. At least five approximately equal pressure increments should be used in a test; 8 to 12 are desirable and as many as 12 to 16 may be used.

Test data are plotted as the increase in measuring cell volume from the initial "as inserted into the borehole" condition versus pressure which may be adjusted for elevation and probe-inertial effects. Also plotted versus pressure is the creep volume or the change in volume which occurs between the 30 and 60-second volume readings at each pressure level. A plot of typical pressuremeter test data (not corrected for inertia or elevation) is presented in Figure B-4.

Also indicated on Figure B-4 are the three phases of the pressuremeter test curve; recompression of disturbed soil, elastic, and plastic. The pressure at which the test enters the elastic phase has been called the lateral earth pressure at rest. It also corresponds to the pressure at which the volume change on the creep curve becomes constant. However, due to borehole disturbance, it often bears no relation to the insitu lateral (horizontal) earth pressure (Baguelin, and others, 1978). This pressure should, therefore, be called the *pseudo, at-rest pressure,* p_o. The creep pressure, p_f, occurs at the end of the elastic phase of the pressuremeter curve; it is also the pressure at which creep volume begins increasing.

The limit pressure, p_1, theoretically is the pressure at which shear failure of the soil occurs. Menard suggests that for practical purposes, this would occur when the borehole has been expanded to twice its initial volume. However, due to limitations of the pressuremeter device, it is generally not physically possible to attain the real limit pressure. Therefore, the test is run well into the plastic range and the pressuremeter test curve extrapolated to the limit pressure. Where the plastic phase of the curve is limited and extrapolation difficult, p_1 is sometimes taken as twice p_f, however, judgement must be exercised.

A pressuremeter deformation modulus is then derived using the following equation (Menard, 1975):

$$E_{PMT} = 2\,(1+\mu)\left(V_c + \frac{v_f + v_o}{2}\right)\left(\frac{p_f - p_o}{v_f - v_0}\right)$$

where, E_{PMT} = pressurement modulus

μ = Poisson's ratio, varies between 0.33 and 0.50

V_c = initial measuring cell volume of the "at rest" probe.

p_o, v_o and p_f, v_f = "pseudo"-at-rest and creep pressures, and corresponding volumes, respectively, from the pressuremeter test curve.

Where an unloading sequence has been used in the test, an unloading pressuremeter modules, E_{PMT} can be calculated using the linear portion of the unloading pressuremeter curve.

Several factors may affect test parameters. Borehole disturbance can cause a decrease of as much as 50 percent in E_{PMT} (Hartman and Schmertmann, 1975).

B.5.2.2 Self-Boring Pressuremeter. In soft clays, use of the Self-Boring pressuremeter may be the most effective way to minimize soil disturbance effects on pressuremeter parameters. The limit pressure may be affected by the ratio of pressuremeter probe length to diameter depending on the assumptions made in test data interpretation (Laier, and others, 1975).

In general, it is difficult to determine strength and deformation parameters commonly used in geothechnical engineering from results of pressuremeter tests. The undrained shear strength of cohesive soil may be calculated using the semiempirical relation:

$$S_u = \frac{p_l - p_h}{N}$$

where,

p_l = limit pressure

p_h = actual *in situ* lateral (horizontal) earth pressures.

N = a factor generally taken as 5.5

Schmertmann (1975) cautions that the calculated S_u is very sensitive to evaluation of *in situ* lateral pressure and maintains that a 20 percent under-estimate error in p_h can produce a 40 percent error (overestimate) in S_u. Procedures have also been developed whereby the complete stress-strain curve for saturated clay under undrained conditions may be derived based on pressuremeter test results (Ladanyi, 1972; Palmer, 1972; Baguelin, and others, 1972; Amar, and others, 1975). However, the procedures are subject to the same difficulties in proper evaluation of p_h.

In cohesionless soil, procedures do exist whereby the effective angle of internal friction can be estimated (Schmertmann, 1975 and Baguelin et al., 1978). However, these procedures require several assumptions (as indicated by Schmertmann and Baguelin) that must be carefully evaluated.

As previously mentioned, *in situ* lateral pressures cannot be accurately measured by the pressuremeter test due to borehole disturbance. Even the minor amount of disturbance caused by the self-boring pressuremeter can have a substantial effect on the measured p_o value.

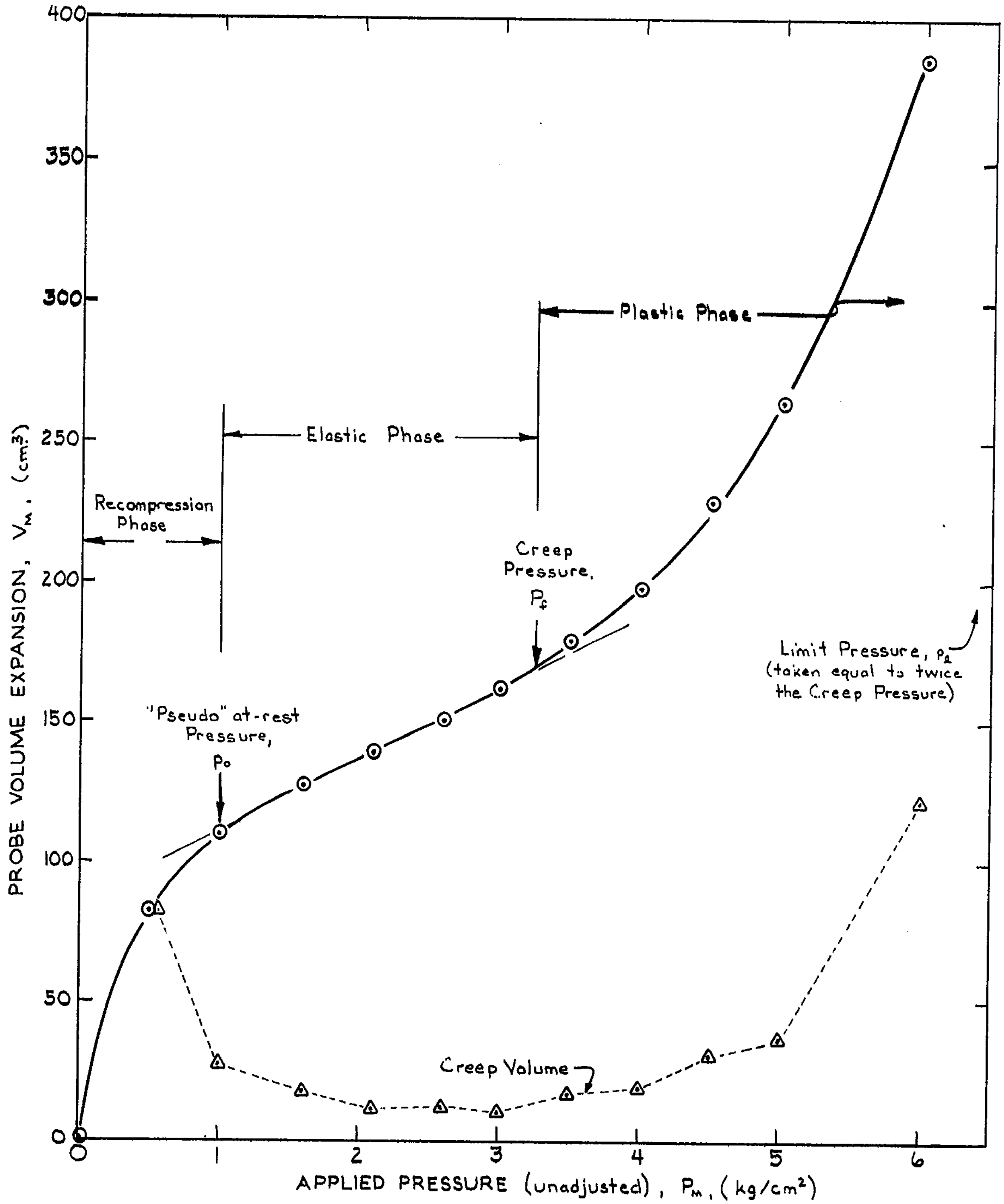


Figure B–4. Plot of typical pressuremeter test data. (Haley & Aldrich, Inc.)

Various procedures have been proposed for evaluating the vertical modulus of deformation of soil, E, using pressuremeter test results. The agreement between E values determined from pressuremeter test results and from other more traditional procedures, such as laboratory tests, *in situ* plate bearing test, and/or full scale footing tests, has often been poor (Baguelin, and others, 1978). Values of the rheological factor, "a", proposed by Centre d'Etudes Menard are presented in Table B-3; soil deformation modulus, E, may be determined by dividing E_{PMT} by "a."

It is generally held that the pressuremeter does not measure consolidation parameters such as m_v or c_v. However, work in Japan has indicated that there may be a relationship between the creep pressure and the preconsolidation pressure of clay (Mori and Tajima, 1964).

Foundation engineering using pressuremeter test results generally uses relationships developed specifically for the pressuremeter test. The bearing capacity and settlement of spread footings and pile foundations can be estimated using such special equations (Baguelin et al., 1978).

The pressuremeter has also been used to estimate the capacity of horizontally loaded piles with varying amounts of success. Experimental work is underway investigating the use of the pressuremeter to estimate soil anchor capacity and earth pressures in retaining structure. The pressuremeter has also been used in fill control work (Kastman, 1980).

B.5.3 Stress or Shear Devices

B.5.3.1 Hydraulic Fracturing (Hydrofracturing). Hydraulic fracturing was initially employed as a technique to improve oil well production by rupturing the adjacent borehole strata. A recent application of this technique is to measure *in situ* stress conditions at a variety of borehole depths. The technique consists of sealing-off a section of the borehole at the desired depth by means of inflatable rubber packers. The packers are then hydraulically pressurized until the surrounding rock ruptures in tension (Haimson, 1977). The pressure which creates the initial fracture is recorded and then additional pressure is applied which is required to keep the fracture open and extend it away from the borehole wall. The recorded pressures are used to calculate the magnitudes of the *in situ* stresses.

After the pressure is released, an impression

Table B-3

Soil Type	Peat		Clay		Silt		Sand		Sand and gravel	
	E_m/p_1^*	α	E_m/p_1^*	α	E_m/p_1^*	α	E_m/p_1^*	α	E_m/p_1^*	α
Over-consolidated			>16	1	>14	2/3	>12	1/2	>10	1/3
Normally consolidated		1	9–16	2/3	8–14	1/2	7–12	1/3	6–10	1/4
Weathered and/or remoulded			7–9	1/2		1/2		1/3		1/4

Rock	Extremely fractured	Other	Slightly fractured or extremely weathered
	α = 1/3	α = 1/2	α = 2/3

Rheological factor, α, values for various soils (from Baguelin, et al., 1978).

L/B	1 Circle	1 Square	2	3	5	20
λ_d	1	1.12	1.53	1.78	2.14	2.65
λ_c	1	1.10	1.20	1.30	1.40	1.50

Foundation shape factors, λ_c and λ_d (from Baguelin, et al., 1978).

packer is used to obtain an oriented imprint of the fractured portion of the borehole showing the inclination and azimuth of the hydraulic fracture. Using the recorded fracturing pressures and the fracture impression, the principal stresses and their directions can be calculated (Haimson, 1977).

The technique of hydraulic fracturing has also been applied to soils. This procedure involves the installation of a conventional piezometer which is back-pressured to fracture the soil. This method has been used successfully to measure lateral stress and tensile strength of fine-grained soils up to a depth of 125 ft. (Bjerrum, L. et al., 1972).

B.5.3.2 Vane Shear Test. The field vane shear test attempts to directly measure the *in situ* undrained shear strength of fine-grained, cohesive soils. Specific details of the test are presented and are summarized in ASTM (D2573). Briefly, the test consists of advancing a four-bladed vane to a desired soil depth and measuring the applied torque as the vane is rotated at a constant rate. Shearing resistance is considered to be mobilized on a cylindrical failure surface corresponding to the top, bottom and sides of the vane assembly (Figure B-5). The preferred vane shape is a rectangular four-bladed vane with a height/diameter ratio of 2.

Several models of vane shear equipment are available. The simplest equipment and field procedure utilizes a conventional torque wrench and the vane is either pressed into the natural, undisturbed soil, or it may be used in standard, cased test borings. Only gross shear strength information can be obtained with this method due to the difficulty in maintaining a constant rate of vane rotation. A major improvement in this technique utilizes a precision-type torque head assembly, such as the "Acker" model, which is used in cased boreholes (Figure B-6). This procedure is capable of maintaining a constant rate of rotation and a high precision force gauge is monitored for each degree of vane revolution. A totally self-contained, portable vane shear unit, which is capable of providing its own cased hole, in addition to constant rotation and accurate pressure monitoring, is the "Geonor", SGI vane borer (Figure B-7).

The untrained shear strength can be calculated

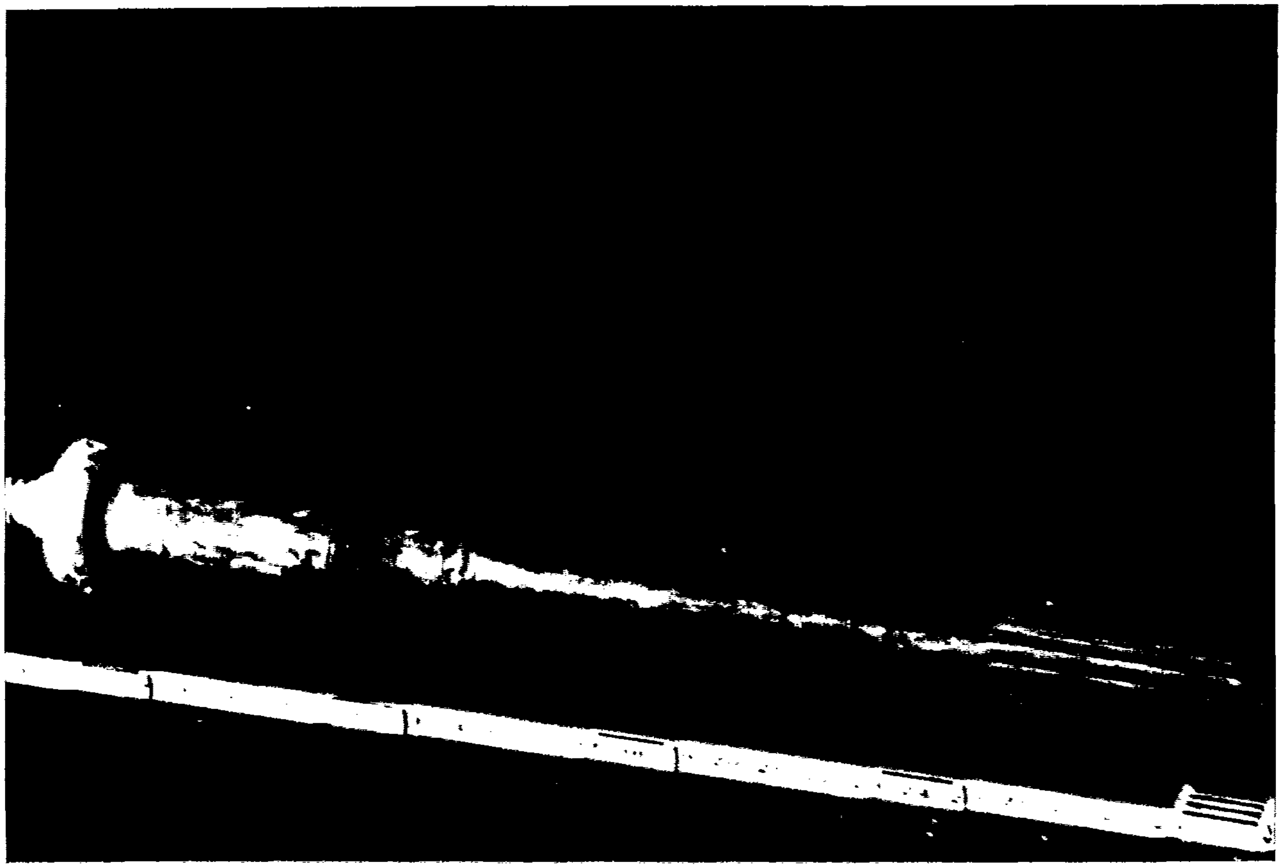


Figure B–5. Vane shear device (Haley & Aldrich, Inc.)

from the measured torque, provided that the shear strengths on the horizontal and vertical planes are assumed equal, by the following:

$$S_{uv} = \frac{2\,T}{\pi\, D^3\,(H/D + a/2)}$$

where

S_{uv} = undrained shear strength
T = maximum applied torque
H = vane height
D = vane diameter
a = factor which is a function of the assumed shear distribution along the top and bottom of the failure cylinder
a = 0.66 if uniform shear is assumed
a = 0.50 if triangular distribution is assumed (i.e. shear strength mobilized is proportional to strain)
a = 0.60 if parabolic distribution is assumed.

The vane shear test actually measures a weighted average of the shear strength on vertical and horizontal planes. It is possible to determine the horizontal and vertical shear strength for either plane by performing the test in similar soil conditions using vanes of different shapes or height/diameter ratios. It has been found that, in general, the ratio of horizontal/vertical shear strength is less than unity. Therefore, an accurately determined value of S_{uv} may be used as a conservative estimate of the shear strength along the vertical plane.

Until recently, the vane shear test was considered to be a very reliable way of measuring the undrained shear strength of soft to medium clays. A number of cases have been encountered in which the use of the vane shear strength leads to unconservative results in undrained stability analysis (Bjerrum, 1972; Pilot, 1972). A correction procedure was developed (Bjerrum) which attributes the difference in field behavior to strain rate effects and relates such effects to a soil plasticity index, Ip. The true undrained shear strength (S_{uv}) is approximately related to the measured shear strength (S_u) as follows:

$$(S_u)_{field} = (S_u)_{vane} \cdot \mu$$

The correction factor, μ, is related to the plasticity index, Ip, as shown in Figure B-8 (Ladd, 1975).

LaRochelle, and others, (1974) and Ladd (1975) have noted that the use of the Bjerrum correction factor may yield occasional unconservative results. The procedure is subject to the additional uncertainty in the determination of Ip and S_{uv}.

The field vane has the advantages of being relatively easy to use and inexpensive and in providing almost continuous undrained shear strength data. However, the vane test is subject to uncertain effects due to such factors as:

- Rotation of principal planes during shear
- Dimensions of vane and failure cylinder
- Rod friction
- Rate of rotation
- Disturbance during insertion

Because the field vane imposes a stress system during shear that is unlike any failure mode encountered in practice, many engineers recommend treating the field vane as a strength index test. That is, field vane strengths for each soil type should be correlated with the results of other tests and observed failures with these soil types in order to establish a calibration factor. It has been found that the field vane greatly over-estimates the undrained strength of many highly plastic clays, especially if they contain roots, shells, sand lenses, and varves.

As a strength index test, the field vane can provide an excellent relative measure of the variation of strength of a given deposit varies with depth and location. Also, once the test has been calibrated, it yields a simple and convenient method of obtaining the undrained shear strength.

B.5.3.3 Borehole Shear Test (BST). The Borehole Shear Test provides a procedure with which *in situ* soil shear strength can be measured in the field. The shear strength of the soil is determined in the borehole by pulling up on the borehole shear device while it applies pressure against the sides of the uncased borehole. By repeating the test at increasing horizontal pressures, a plot of maximum shear stress versus normal stress can be developed. Shear strength parameters c and ϕ are determined by drawing the Mohr-Coulomb failure envelope.

Advantages of the Borehole Shear Test (Mitchell et al., 1978):

- The test is relatively quick; shear strength parameters can be determined in less than an hour in some soils.
- Erroneous shear values are usually apparent and tests may be re-run as necessary.
- Tests may be located in specific strata as they are usually performed in completed, logged boreholes.

Some of the disadvantages of the BST are uncertainty about drainage conditions in the zones being sheared, and the unsuitability of some soils to stage testing. Interference by the ends of the shear heads on soil not under normal stress may add to the apparent shear force resistance (Schmertmann, 1975).

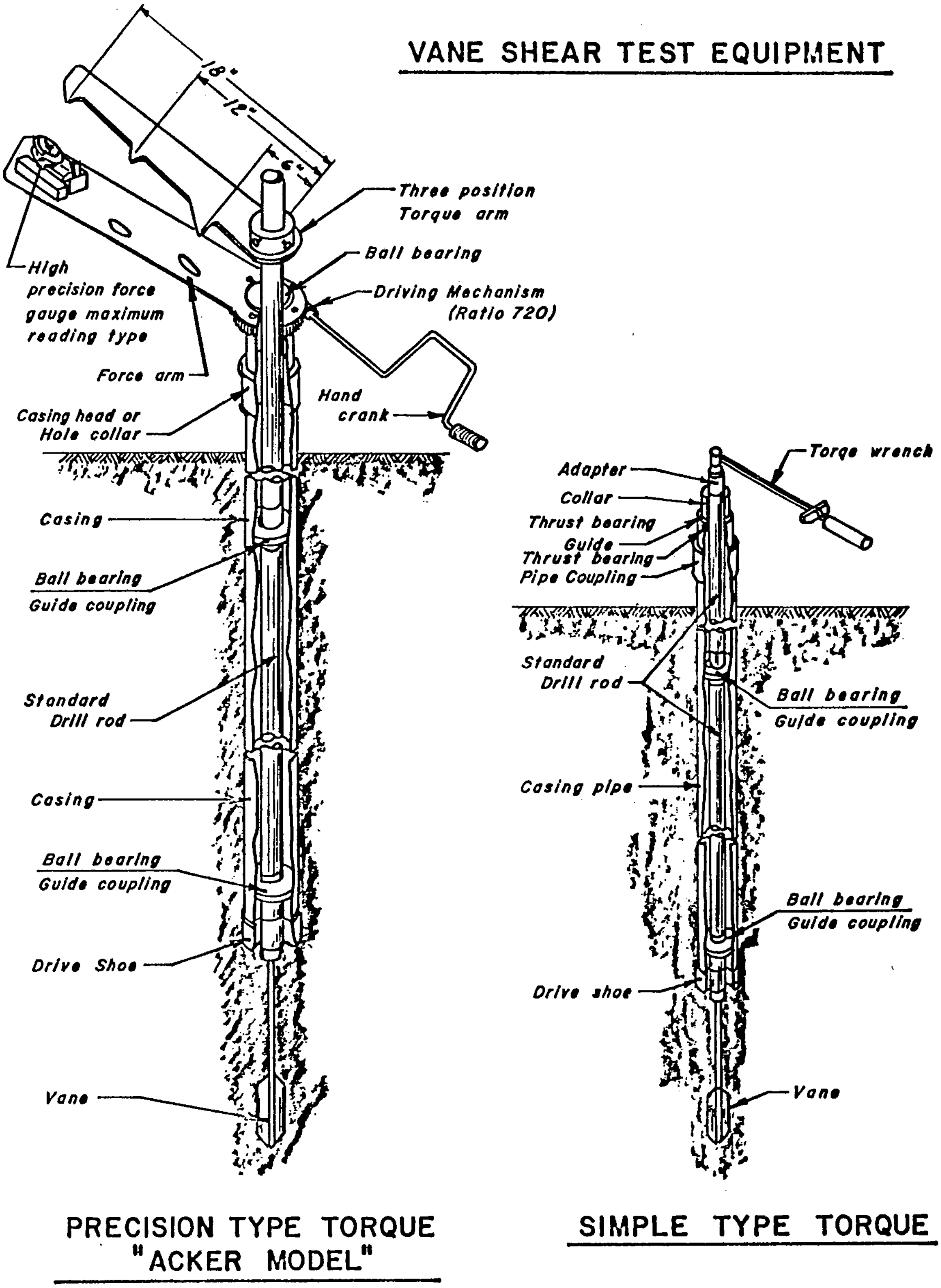


Figure B–6. Vane Shear Test Equipment. (Acker Drill Co.)

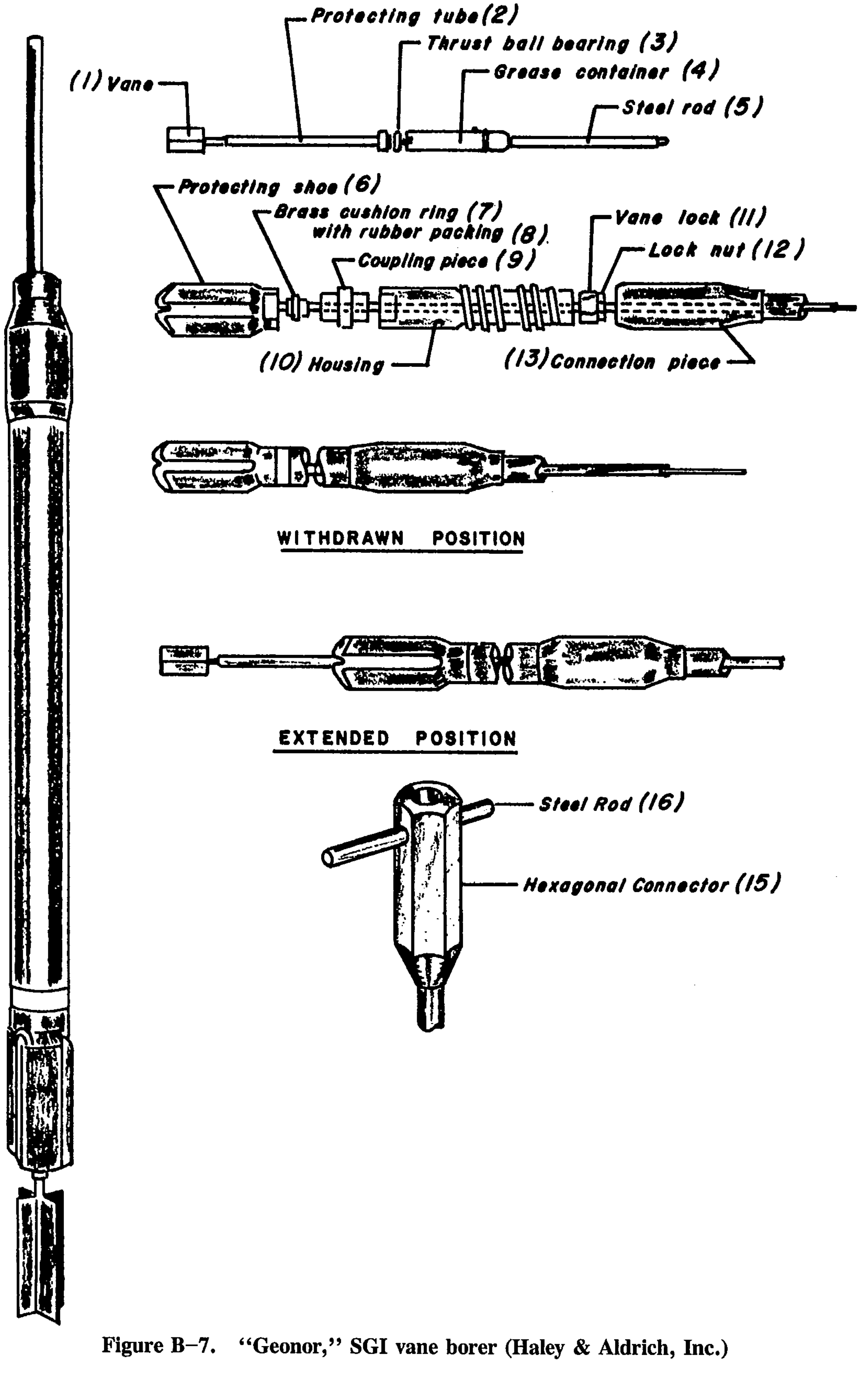


Figure B–7. "Geonor," SGI vane borer (Haley & Aldrich, Inc.)

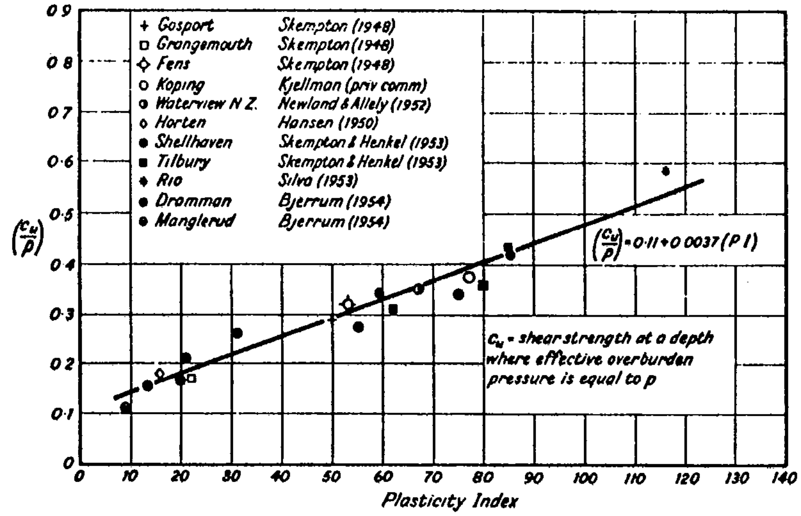


Figure B–8. The relationship between (c_u/p) and Plasticity Index (PI) for normally consolidated clays. (Ladd, 1975)

The main components of the Borehole Shear Device are: the shear head, the pulling device, and the console (Wineland, 1975). Refer to Figure B-9 for schematics of the major components.

In the shear head, curved, ridged shear plates are forced against the sides of the borehole by gas pressure (usually nitrogen) in the pistons; the pressure is controlled at the console. The knife edges at the top and bottom of the shear heads minimize end resistance. The shear head is connected to the pulling device by steel rods.

The hand operated pulling device rests on a plate at ground surface, centered above the borehole. A worm gear and screw device is used for pulling the shear head, via the rods. Shear force is measured by pressure in two hydraulic cylinders.

The console component includes bottled gas used to press the shear heads into the borehole sides, a pressure regulator, and pressure gauge. Pressurized gas is delivered to the shear head by plastic tubing.

A general description of the performance of the Borehole Shear Test follows. Refer to Appendix C for further details. To conduct a BST, a cased borehole is advanced to the depth of the test. Final hole preparation is made by advancing a 76 mm (3-in.) thin-walled tube sampler into the borehole bottom. The test is performed in the hole thus created. The shear head is lowered down the borehole by the rods. At the proper location, the plates are expanded against the borehole sides under a controlled pressure. A ten minute wait is usually allowed for consolidation of the soil beneath the shear plates at the initial pressure.

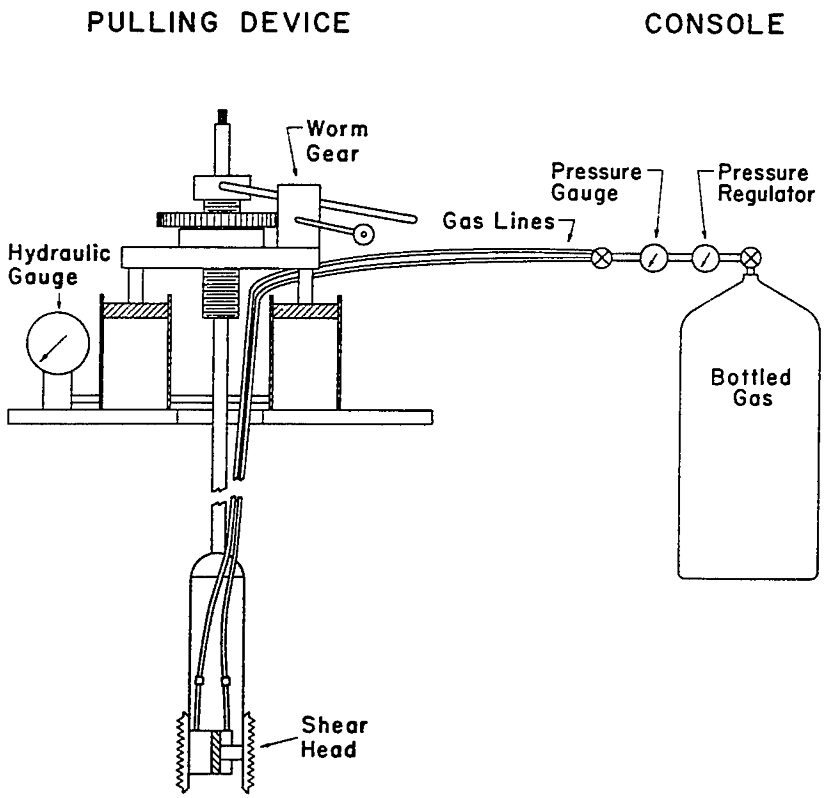
After consolidation, the pulling device is used to pull the shear head up at a constant rate of 0.05 mm (0.002 in.) per second. Pulling is continued until maximum shear force is reached. Normal and shear stresses can be calculated knowing the area of the shear plates. They are plotted as one point of the Mohr envelope.

The pressure of the shear plates against the borehole sides is then increased, a consolidation period allowed, generally five minutes, and the shear process repeated. As many test stages as desired may be performed (or until the shear heads have been fully expanded). Five points on the Mohr envelope are usually sufficient to establish the required shear strength parameters.

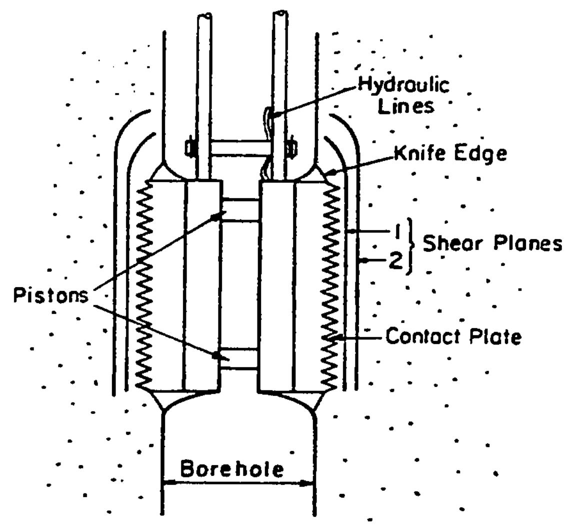
In some low permeability soils, clays in particular, longer consolidation periods may be required. Inconsistent data may be checked by rotating the shear head 90 degrees and rerunning several phases of the first test.

An example of Borehole Shear Test data sheet and data with plotted results is presented in Figure B-10. Normal and shear stresses may be determined in the field. The Mohr envelope was drawn through the origin and through the steeper data points. The drop in shear stress at higher normal stress is often attributed to development of planes of reduced shear strength and/or full expansion of the shear head such that the pressure indicated on the pressure gauge is not actually applied to the shear plates.

Close agreement between shear strength parameters determined in the Borehole Shear Test and other



Major components (from Wineland, 1975)



Details of Shear Head (after Mitchell, et al., 1978)

Figure B–9. Schematic of Borehole Shear Device.

field tests and laboratory triaxial tests have been reported by several investigators (Lambrechts and Rixner, 1981, and Nickel, 1975). The BST appears most useful in relatively free draining soils, such as sands and some silts, where there is little worry about the development of untrained conditions during shear.

B.6 PERMEABILITY TESTS

B.6.1 Water Pressure Tests

Water Pressure Tests are conducted *in situ*, within pre-drilled test borings, to measure the permeability of a specific stratigraphic zone, usually bedrock. Pressure testing helps to locate zones of leakage and measures the capacity of such zones for transmitting water. Pressure test data are used in appraising conditions that may exist in bedrock at the level of a proposed tunnel or foundation; and are useful in estimating grouting and dewatering requirements for construction purposes.

The Water Pressure Test assembly consists of expandable packers, either mechanical (Figure B-11) or pneumatic (Figure B-12), which can isolate specific borehole sections or strata. Double pneumatic packer assemblies with a 1.5 m (5 ft.) spacing between the packers are the more commonly used; however, the spacing between the packers can be adjusted to accommodate the specific situation. In addition, only one packer may be required to isolate major sections of the borehole.

The time rate of water flow into the isolated test section, at a selected gauge pressure, is recorded for 5 to 30 mins., depending on the volume of water which enters the test section. This procedural step is usually repeated several times at increasingly higher pressures, but not exceeding a computed maximum allowable gauge pressure, which could hydraulically fracture the strata being tested. After the zone of interest is tested, the packer(s) may be deflated and the assembly moved and reset in different zones for additional tests.

The coefficient of permeability (k) calculated from the test results of the various borehole permeability tests gives a gross indication of the overall mass permeability. A qualitative description of rock mass permeability based on water pressure test results is given below:

Rock Mass Permeability (cm/sec)	
Less than 1×10^{-6}	Very low (Equivalent to clay)
1×10^{-5} to 1×10^{-6}	Low (Equivalent to silt)
1×10^{-3} to 1×10^{-5}	Medium (Equivalent to fine sand)
1×10^{-2} to 1×10^{-3}	High (Equivalent to sand)
More than 1×10^{-2}	Very high (Equivalent to clean sand or gravel)

It should be noted that water inflow into a rock excavation will probably be greater than predicted by these measurements. Most water flow in rock occurs along individual discontinuities and a single discontinuity could provide the majority of water flow to an excavation. It is highly unlikely that water pressure testing would locate all or even most of the worst flow conditions. Hence, the average flow computation should be multiplied by some factor such as 2.0 or greater in order to obtain a reasonable and conservative estimate of water inflow. Observation of nearby excavations or cut slopes, or careful inspection of large diameter explorations will help in selecting an appropriate multiplication factor.

B.6.2 Pump Test

Pump tests are a reliable means of measuring the hydraulic conductivity of a water-bearing material because the material is not disturbed as in laboratory tests (See Section 10), and because a sizeable area of the material is tested over a relatively long period of time compared to the other field permeability tests. Pump tests provide data that are used to calculate transmissivity and the storage coefficient of the water-bearing materials in question. These two measures of the material's ability to transmit and store water are essential for further quantitative evaluation of the material's hydrologic responses to various changes in conditions that may be caused by construction of some transportation structure. For example they are necessary to calculate required well spacing and pumping rates to dewater an area for construction of a deep foundation or a tunnel. They are also necessary to calculate the area over which water levels may be lowered by excavation of a road cut. If desired, the value of the hydraulic conductivity of the material may be found by dividing transmissivity by the saturated thickness of the water-bearing material.

Pump tests require a pumping well plus at least one other observation well. Several observation wells at varying distances from the pumping well are preferred. Most often it is necessary to have several observation wells, and sometimes more than one pumping well, depending upon the area and the type and complexity of the material being tested. Many areas have multiple water-bearing zones, and their degree of interconnection must be determined because there may be anything from virtually no water transfer between zones to sufficient transfer that they are practically one zone. Such situations usually require compound or cluster observation wells to provide water

IOWA BORE HOLE SHEAR TEST

Project: Keene - Rte. 9 Bridge Boring No: B-203 Test No. 3

Location STA 184 + 27 ; Right 59' Date 18 July 1978

Depth 26.5' below ground surface Horizon ____ Tested by ____

Description ____

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}		
1	12	2.5	6	1.2		
2	20	4.8	14	3.4		
3	30	7.4	19	4.8		
4	40	10.1	22	5.5		
5	50	12.9	6	1.2		

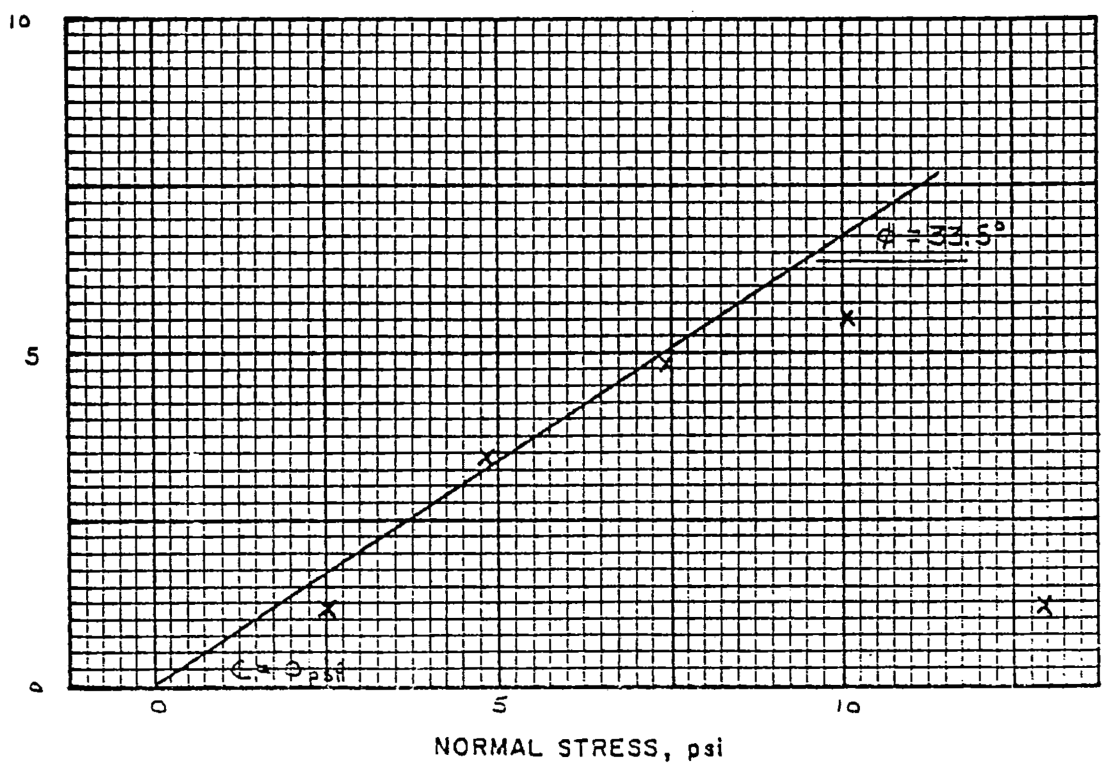


Figure B–10. Example Borehole Shear Test Data Sheet and Mohr Envelope Plot.

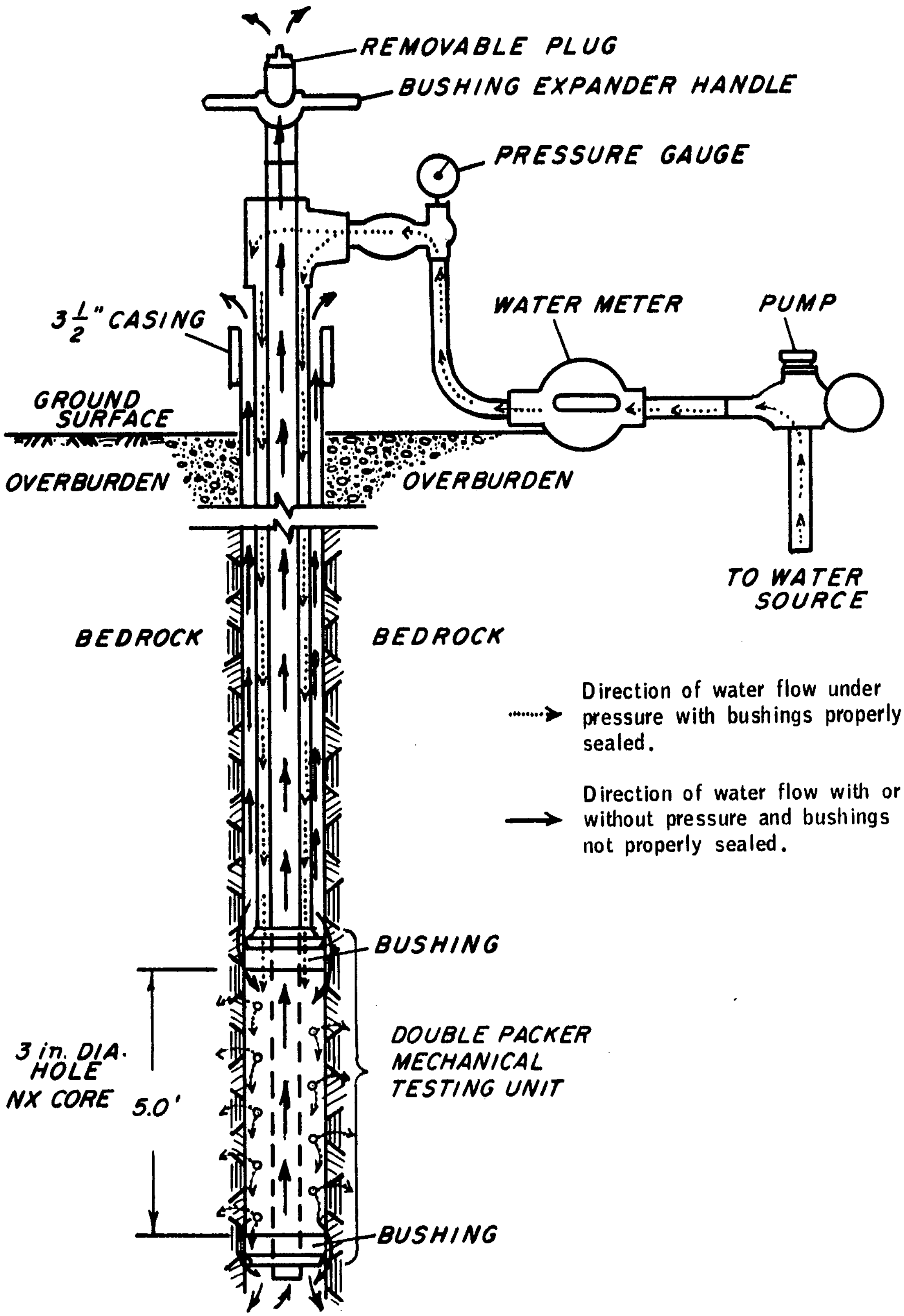


Figure B–11. **Water Pressure Test Equipment. Mechanical Packers. (Haley & Aldrich, Inc.)**

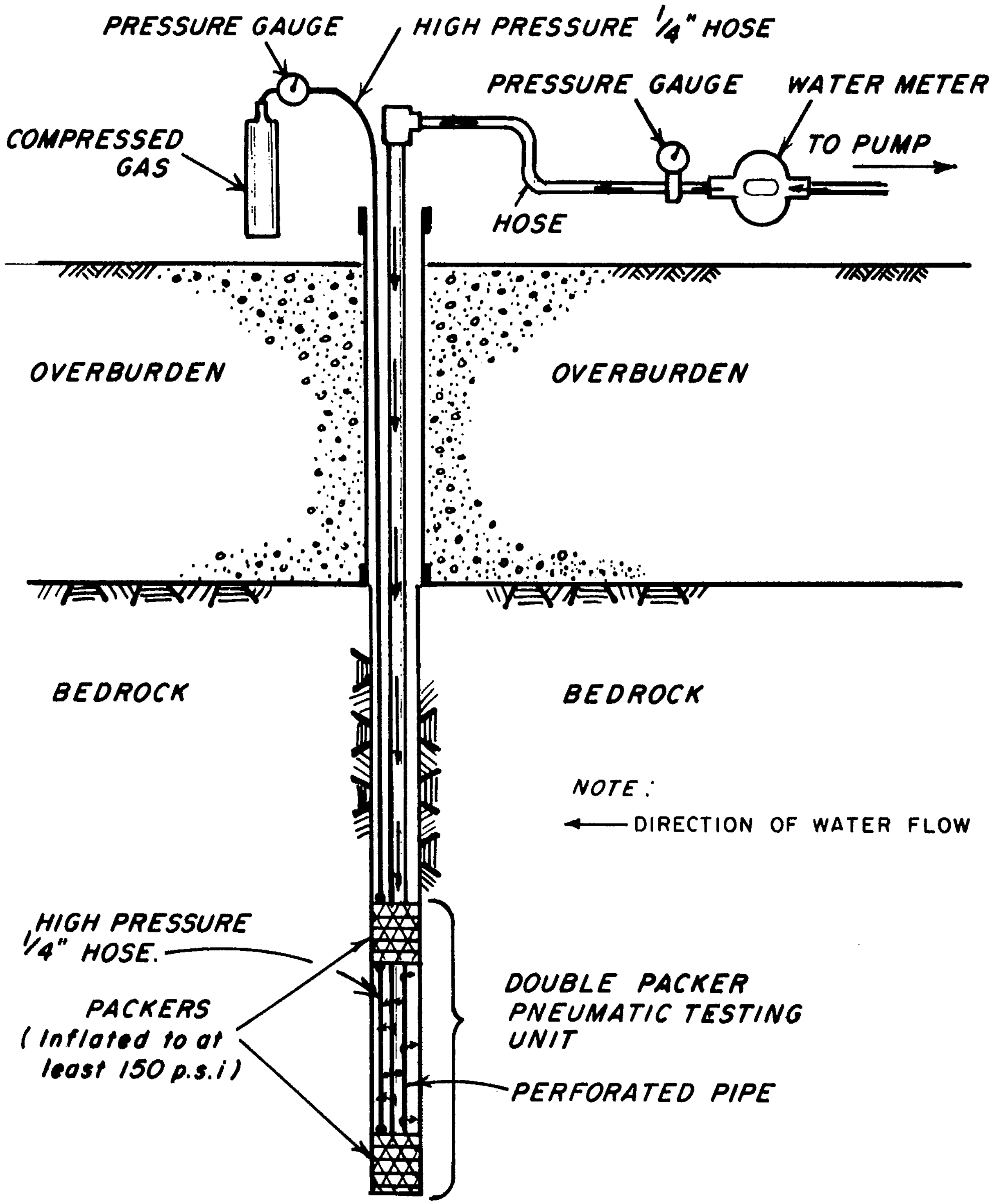


Figure B–12. Water Pressure Test Equipment. Pneumatic Packers.

levels at multiple depths at one point. Thus, it becomes apparent that pump tests are an expensive method of hydraulic conductivity measurement. Also, pump tests can be used only in saturated materials that will yield fairly large amounts of water to a well. For these reasons, pump tests will not be appropriate or economical for every project. Pump tests are most appropriate on projects that require large excavations into high-yield aquifers. Such projects would include tunnels, deep foundations, and possibly very large road cuts, where the ability to carry out the construction, the safety of the workers, and the effects on adjacent structures or aquifers due to dewatering a large area are items of great concern.

When conducting a pump test, it is important to test the water-bearing material under essentially natural conditions. Therefore, it is necessary to convey the pumped water to a place where it can be discharged without seeping back in the material to provide abnormal recharge that will give false (too high) water level readings in one or more of the observation wells. It is necessary as well to be sure that pumping wells and observation wells are not providing a conduit to allow seepage from overlying materials to move unnaturally into the zone being tested. Such seepage is controlled by placing well casing and sealing it at the top of the zone to be tested. Sealing may be accomplished by driving the casing into the top of the water-bearing material or into an overlying impermeable material if present. Another method is to drill a larger hole to the top of the water-bearing zone, center the casing in this hole, and fill the annular space between the casing and the outer hole with an impermeable material. This method provides an adequate seal, especially if the entire annular space is filled with cement grout. However, it is not always practical or necessary to place the grout to more than a few feet above the top of the water-bearing zone. It may be sufficient even to use a swelling clay such as bentonite in place of the grout, especially for the observation wells. In either case, a smaller hole is then drilled inside the casing to the bottom of the water-bearing zone, and the well completed for pumping.

B.6.3 Hydraulic Conductivity Tests

Aside from the pump tests discussed in the preceding section, there are a number of other means of determining the hydraulic conductivity of water-bearing materials. One is to remove water from the material being tested, usually by means of a well. Another is to add water to the material, either in the field by means of a well, or in the laboratory on a sample of the material removed from the field.

The water-removal methods can be applied only in the saturated zone, and in materials of sufficient hydraulic conductivity to yield water to the well at a reasonable rate. The water-addition methods can be used in the field or laboratory on materials that are unsaturated as well as saturated, and on materials of rather low hydraulic conductivity. Whether testing is done in the field or in the laboratory, there are two types of water-addition methods that can be used. One is called the falling head test and the other is called the constant head test. None of these methods require more than one well. That is, all required measurements can be made in one well without the need for additional observation wells as in a pump test.

Among the several field falling-head methods, which are based on the rate of decay of an excess head imposed in a borehole, a method described by Hvorslev (1951) is considered quite useful. Requirements are that the borehole opening be of a definite size and shape, and that the hole remain open, which may require the placing of a screen to prevent the collapse of the hole while allowing water to enter the hole. The borehole is filled with water and the water level is measured at frequent time intervals for a period of about 15 minutes or until the water level drops about one foot. The hydraulic conductivity of the material around the borehole at the test depth can be calculated at one's convenience using the procedure described by Hvorslev (1951). The calculation is related to the configuration of the borehole opening used, the rate of fall of the water level, and the static water level in the well prior to testing.

Constant-head methods are based on the rate at which water must be added to the hole to maintain an excess head in the borehole. This type of test may be required in highly permeable materials because the water level will fall too rapidly for the series of measurements required in the falling head tests. Procedure for calculating hydraulic conductivity have been developed for this method as well and a good procedure is presented by Hvorslev (1951).

A method of testing that in a test boring or well is called the rising head test can be conducted. It involves the removal of a measured volume of water from a well by means of a bailer or pump and measuring the residual drawdown some time after the completion of any number of bailing cycles. The method is described in more detail and the rather simple method of calculating the transmissivity is shown in Ferris, and others (1962). The water level prior to the start of bailing must be measured in order to determine the residual drawdown, which is the difference between the "static" level (the water level prior to bailing) and the recovered water level measured after bailing has ceased. Also, in order to calculate the hydraulic conductivity, the thickness of the water-bearing zone must be known, since transmissivity is equal to the hydraulic conductivity multiplied by the thickness of the water-bearing material. However, if transmissivity is known it is not always necessary to determine the hydraulic conductivity per se. A somewhat similar, but more elaborate, test in which the hydraulic conductivity is calculated directly by a relatively complicated procedure, is called the auger-hole method. This method is described in *Drainage of Agricultural Land* (U.S. Soil Conservation Service, 1973).

Attempts have been made to estimate hydraulic conductivity by timing the movement of a tracer from one observation well to another. Various materials have been used as tracers, including dyes, soluble salts, radioactive substances, and biological agents. These methods frequently produce unsatisfactory results due to dilution and reactions with soil and rock materials. Radioactive tracers, and sometimes biological agents, also tend to produce adverse emotional reactions from the public. Tracers are most reliable only over short distances, requiring a much closer spacing of observation wells than would otherwise be needed, adding to the cost of the project. Tracers become more effective as more is known about the water-bearing material and the groundwater flow system, and so are best used as confirmation of previously calculated flow directions and rates.

B.6.4 Percolation Tests

Percolation tests measure the movement of water into and through the unsaturated zone. The movement of water from the surface into the soil or rock is called infiltration. Percolation refers to the movement of water through the subsurface materials.

Infiltration is measured by two basic types of systems, or infiltrometers. They are the sprinkler type and the flooding type. Sprinkler infiltrometers attempt to simulate rainfall and the dynamic action of rain-drop impact. Infiltration rates determined by sprinkler methods generally yield infiltration rates about one-half as great as the rates obtained by flooding methods. Therefore, the method used should match the situation for which the infiltration data is to be used. For example, highway drainage is often discharged over the land rather than directly to a stream, and a flooding-type infiltrometer would be the most appropriate to evaluate the resultant runoff. The use of flooding type infiltrometers is generally much simpler than sprinklers, and so they are used more often. A common flooding method is the double-ring infiltrometer. These methods essentially measure vertical flow of water expressed in inches per hour. Infiltration rates apply only to the soil and vegetative conditions prevailing where the test is made and should be extended to areas with different conditions only with great care. A different approach to determining infiltration averaged over larger areas is to use hydrography analysis of runoff from the watershed of interest. Discussion of all of these methods and more specific references can be found in Gray (1970) and U.S. Environmental Protection Agency (1977). Infiltration data are useful primarily in runoff calculations for drainage designs.

Percolation tests are required by most states for establishing the size and acceptability of septic tank drainage fields for on-site sewage disposal. Some methods are described in U.S. Environmental Protection Agency (1977). These tests should not be used for geotechnical design purposes as they are known to give excessive and unrealistically high infiltration rates. They are mentioned here because on-site sewage disposal may be employed in conjunction with highway rest areas, maintenance and office buildings, and other similar structures. Appropriate state and local agencies should be consulted for details of percolation test requirements in the jurisdiction of a specific project.

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APPENDIX C
In Situ Testing Procedures

A selected summary of field testing procedures required to determine various soil and rock properties, and the associated forms to record the data, are included within Appendix B. This summary is not intended to be all-inclusive of the many field tests currently in existence. It is intended to discuss only those basic tests and procedures which are in common use. A detailed discussion of the very complex methods which are not commonly in use is beyond the scope of this Manual.

The various field procedures are included only for general guideline purposes, realizing that specific operational details are dependent upon local practice, available equipment, and subsurface conditions. Standard and acceptable procedures in one part of the country may be prohibited or not available in other areas.

Various field data collection forms are also included with the applicable procedure for general information purposes. These forms are presented only as a guide to the type of information required. Specific field forms will also vary between organizations and alternate formats which record the pertinent information may be used.

IN SITU BOREHOLE TESTING PROCEDURES

STANDARD PENETRATION TEST (SPT)

(ASTM DESIGNATION D-1586, AASHTO DESIGNATION T-206)

1. The sampler spoon consists of a 5 cm (2 in.) O.C. by 3.5 cm (1³/₈ in.) I.D., 46 cm (18 in.) minimum length, heat treated, case-hardened, steel head, split-spoon and shoe assembly. Splitspoon or split-tube samplers are the most generally accepted method for obtaining representative, disturbed samples of the selected stratum.

   The head is vented to prevent pressure buildup during sampling and must be kept clean. A steel ball watercheck is located in the head to prevent downward water pressure from acting on the sample. Removal of the watercheck frequently causes sample loss.
2. The drive rods which connect the split-spoon to the drive head should have a stiffness equal or greater than that of the A-rod. In order to maintain only minimal rod deflection, on exceptionally deep holes it may be preferable to employ N-rods. The size of the drive rods must be kept constant throughout a specific exploration program, as the energy absorbed will vary with the size and weight of the rod employed.
3. The drive head consists of a guide rod to give the drop hammer free fall in order to strike the anvil attached to the lower end of the assembly. The rod must be at least 1.1 m (3.5 ft.) in length to insure the correct hammer drop.
4. The drop hammer used in determining SPT resistance must weigh 64 kg (140 lbs.) and

have a 5.15 cm (2.5 in.) diameter hole through the center, for passage of the drive head guide rod.

5. The hammer is raised with a rope (gin line) activated by the drill rig cathead; no more than two turns of the rope should be allowed on the cathead. A 76 cm (30 in.) hammer is mandatory for proper SPT determination. Extreme care must be exercised to produce consistent results.

   Automatic trip hammers are commercially available which insure the 76 cm (30 in.), free-fall drop. When preservation of the soil structure is critical (such as in liquefaction studies), the automatic trip hammer should be employed.
6. Attach the split-spoon to the drill rods and lower the assembly to the bottom of the hole. Measure the drill rod stickup to determine if "heave" or "blow-up" of the stratum has occurred. Note any penetration of the sampler into the stratum under the weight of the rods. The 64 kg (140 lb.) hammer is raised 76 cm (30 in.) above the drivehead anvil and then allowed to free fall and strike the anvil. This procedure is repeated until the sampler has penetrated 45 cm (18 in.) into the stratum at the bottom of the hole.
7. The number of blows of the hammer required for each 15 cm (6 in.) penetration is counted and recorded. A penetration rate of 100 blows per 30 cm (foot) is normally considered "refusal"; however, this criterion may be varied depending upon the desired information. The penetration resistance (N) is determined by adding the second and third 15 cm (6 in.) resistance blow counts together. When other sizes and types of sampling and drive equipment are employed, Figs. C-1 and C-2 may be used in converting the obtained blow count to the accepted SPT value.
8. The sampler is then withdrawn from the borehole, preferably by pulling on the rope. If the sampler is difficult to remove from the stratum, it may be necessary to remove it by hitting the drive head upward with short, light hammer strokes. Remove the sampler from the bottom of the borehole slowly to minimize disturbance. Keep the casing full of water during the removal operation.
9. Careful measurement of all drilling tools, samplers and casing must be exercised during all phases of the test boring operations, to insure maximum quality and recovery of the sample.
10. The split-spoon is opened and carefully examined, noting all soil characteristics, color seams, disturbance, etc. A representative sample is selected and preserved in a screw-top, glass jar and properly labeled. In the event that more than one soil type is encountered in the sample split-spoon, each soil type should be preserved in a separate jar. Refer to Section 6.9, Sample Preservation and Shipment.

ROCK QUALITY DESIGNATION (RQD)

Figure C-3 illustrates the basic approach for obtaining the Rock Quality Designation (RQD) of rock core. In general RQD is defined as the total length of recovered core pieces greater than 102 mm (4.0 in.) in length expressed as a percent of the core drilled. Since RQD is supposed to be an "objective" measure of rock quality, it would be ideal if the procedure was, in fact, this easy. Unfortunately, several factors such as those listed below must be properly evaluated in order for RQD to provide reliable results.

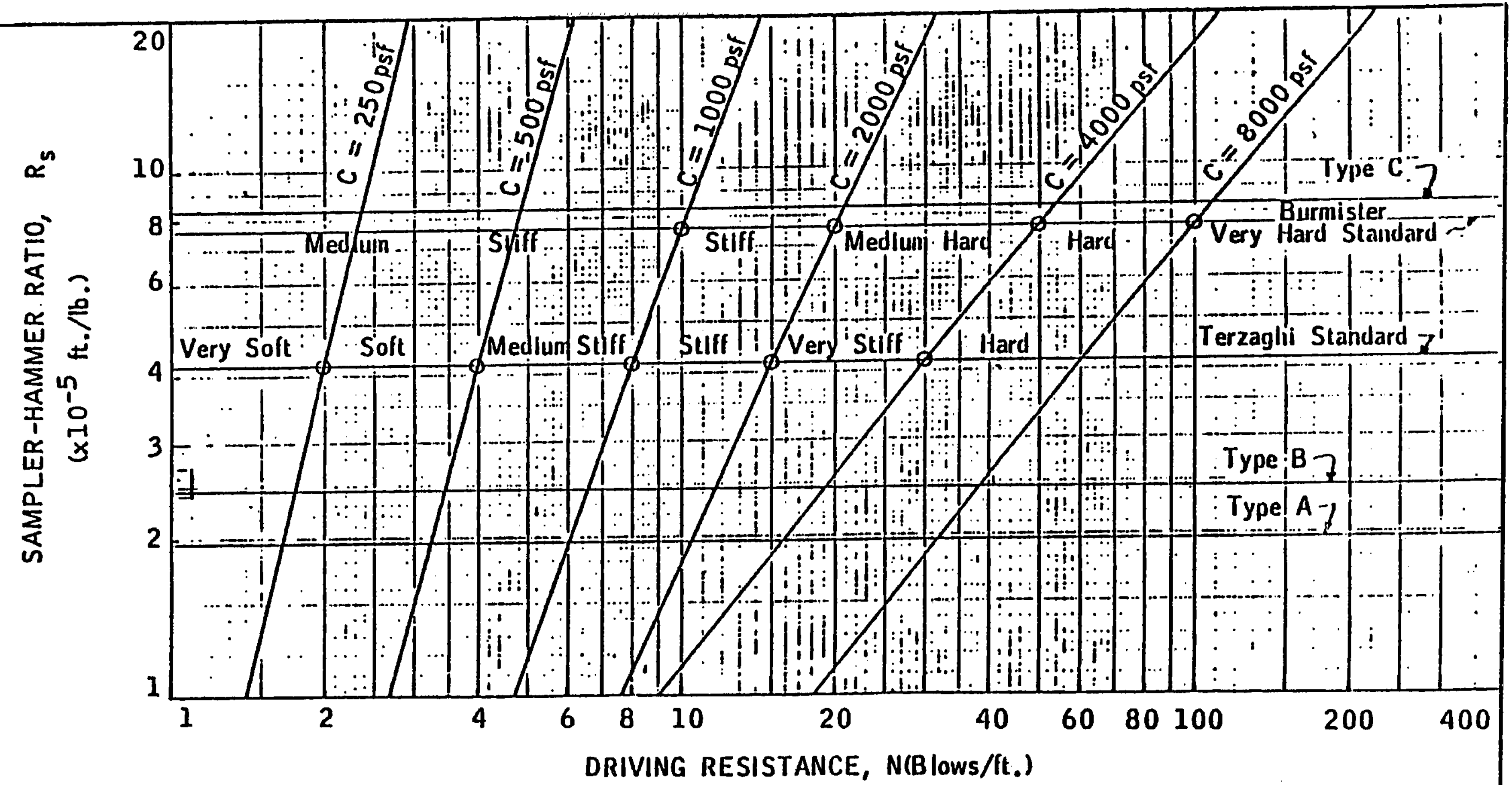
Core Barrel Size and Type: RQD is most frequently calculated for NQ size core or larger obtained with double-tube core barrels. Smaller diameter cores and single-tube core barrels tend to adversely affect the quality of the core, thereby artificially degrading the RQD. Thus, RQD should not be used with core barrels smaller than NQ.

Weathering: Rock assigned a weathering classification of *severe* or *very severe* should not be included in the determination of RQD, regardless of length.

Core Recovery: RQD measurements intrinsically assume that coring is well done and that core recovery is at or near 100%. As core recovery varies from 0–100% explanatory notes may be required in order to describe the reason for the variation and the effect on RQD.

Variation in Core Run Length: RQD is most frequently determined per core run. If the runs vary greatly in length, RQD can also vary without significant changes in core quality. For instance, if 15 cm (6 in.) of bad rock is recovered in a 60 cm (2.0 ft.) core run, the RQD would be 75 percent. If the core run was extended to 1.5 m (5 ft.) without encountering additional poor rock, the RQD would be 90 percent. In general, RQD should be based on consistent 1.5 m (5 ft.) or 3 m (10 ft.) core runs. Variations in core run lengths should be noted.

Drilling Fractures: Only natural fractures such as joints or shear lanes should be considered when calculating the RQD. Fractures due to drilling and handling must be discounted.

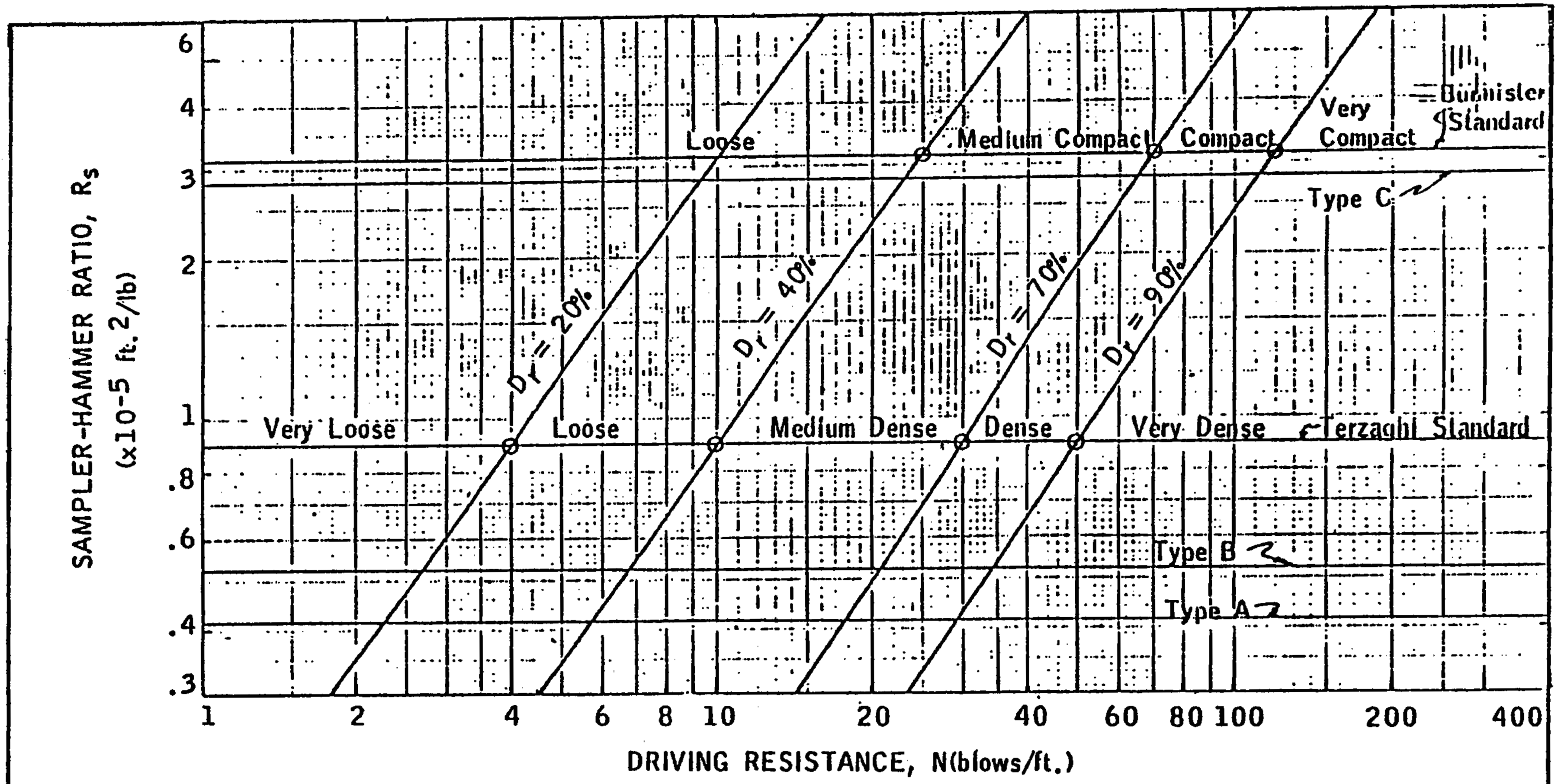


W = Weight of Hammer (lb.)
H = Height of Drop (in.)
D_o = Outside Diameter Sampler Shoe (in.)
D_i = Inside Diameter Sampler Shoe (in.)
R_s = Sampler-Hammer Ratio = $\frac{D_o^2 - D_i^2}{WH(12)}$ (ft./lb.)
(Cohesive Soils)
C = Shearing Strength (psf)

SAMPLER TYPE	W	H	D_o	D_i	R_s
TERZAGHI STANDARD	140	30	2.000	1.375	4.18×10^{-5}
BURMISTER STANDARD	250	20	3.625	2.930	7.59×10^{-5}
TYPE A	300	30	2.000	1.375	1.95×10^{-5}
TYPE B	300	24	2.000	1.375	2.44×10^{-5}
TYPE C	140	24	3.000	2.375	8.33×10^{-5}

SAMPLER DRIVING RESISTANCE
VS. SHEAR STRENGTH
COHESIVE SOILS

Figure C-1



W = Weight of Hammer (lb.)

H = Height of Drop (in.)

D_o = Outside Diameter Sampler Shoe (in.)

D_i = Inside Diameter Sampler Shoe (in.)

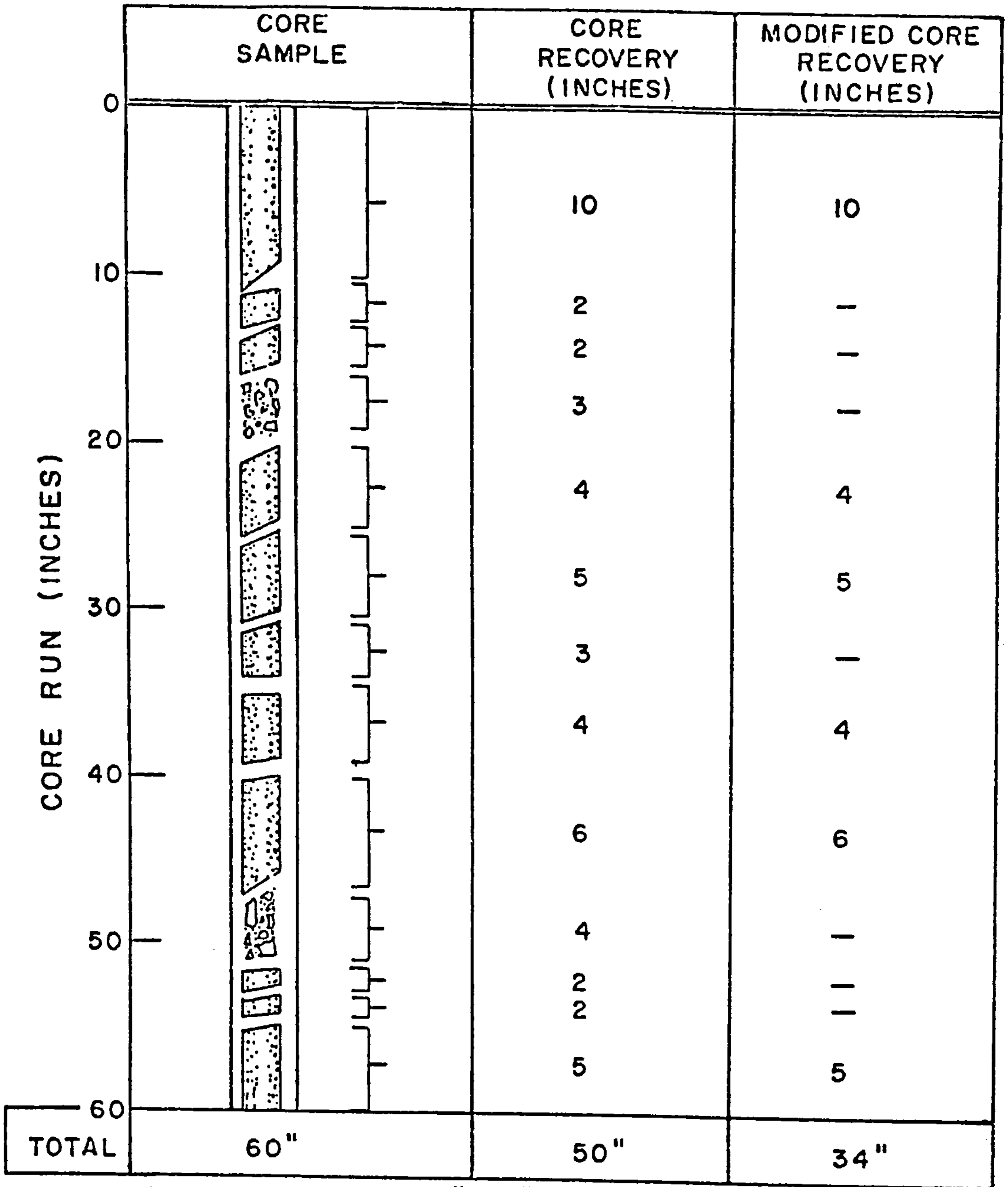
R_s = Sampler-Hammer Ratio = $\frac{D_o^3 - D_i^3}{WH(144)}$ (ft.^{2}/lb.) (Cohesionless sands and silts)

D_r = Relative Density (%)

SAMPLER TYPE	W	H	D_o	D_i	R_s
TERZAGHI STANDARD	140	30	2.000	1.375	0.893×10^{-5}
BURMISTER STANDARD	250	20	3.625	2.930	3.12×10^{-5}
TYPE A	300	30	2.000	1.375	0.417×10^{-5}
TYPE B	300	24	2.000	1.375	0.521×10^{-5}
TYPE C	140	24	3.000	2.375	2.81×10^{-5}

Figure C-2

SAMPLER DRIVING RESISTANCE VS. COMPACTNESS COHESIONLESS SANDS AND SILTS



CORE RECOVERY = 50"/60" = 84 % RQD = 34"/60" = 57 %

RQD (Rock Quality Designation)	Description of Rock Quality
0 – 25	V. Poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

Figure C-3
MODIFIED CORE RECOVERY AS AN INDEX OF ROCK QUALITY

RQD can be useful as a measure of overall rock quality and as a guide to engineering judgement. It is beneficial to know, for instance, if a tunnel will be excavated in poor quality rock or excellent quality rock. Difficulty has been encountered, however, in trying to correlate RQD with specific rock behavioral characteristics without knowledge of other important rock properties such as continuity. RQD should be considered only as an approximate measure of overall rock quality.

DYNAMIC PENETROMETER TESTS

Recoverable Type

The equipment consists of a penetrometer cone (solid, conical point), drill rods to advance the boring, anvil, and a hammer having a regulated fall. It is to be noted that dimensions and weights vary among the state organizations. The most common hammer used is the standard 64 kg (140 lb.) with 76 cm (30 in.) drop, and the most common cone diameter is 5 cm (2 in.). Another combination used is the 77 kg (170 lb.) hammer with a 60 cm (24 in.) regulated drop on a 7.5 cm (3 in.) diameter cone.

Procedure

1. The bottom of the boring is cleaned out, and the cone threaded to the end of the drill rod is lowered to the bottom of the hole. Then the anvil is attached to the top of the drill rod with the hammer placed in position on top of the anvil.
2. The penetrometer cone is seated at the bottom of the hole by driving it a sufficient distance to penetrate any disturbed material.
3. The depth to the tip of the cone is recorded and reference marks are made on the drill rod at 15 cm (6 in.) increments.
4. The actual test consists of driving the cone with the chosen hammer dropped its regulated distance. As in the SPT, this regulation should preferably be accomplished mechanically by an automatic tripping mechanism, and the height of fall should be checked during driving.
5. In relatively soft materials, the number of blows required to drive the cone at 15 cm (6 in.) increments for a total of 30 cm (one foot) should be recorded.

Expendable Type

The equipment consists of an expendable penetrometer point, standard drill rod to advance the test, anvil, and a regulated drop hammer. The penetrometer point must be larger than the drill rod in order to prevent skin friction on the drill rod. The most common size penetrometer point is 5.1 cm (2.3 in.) in diameter used with 4 cm (1.6 in.) diameter drill rod (Standard A-rod). The hammers used in this test are usually heavier than the one used in the SPT. Two in use are the 136 kg (300 lb.) with the 76 cm (30 in.) drop and the 159 kg (350 lb.) with the 46 cm (18 in.) drop.

Procedure

1. The expendable penetrometer point is attached to the drill rod through a sleeve arrangement, and the anvil and hammer are placed in position on top of the drill rod. Reference marks are then made at every 30 cm (foot) or as needed.
2. The actual test consists of continuously driving the penetrometer point with the regulated drop hammer. In relatively soft materials, the number of blows required per 30 cm (foot) of penetration should be recorded. In hard materials, including rock, the number of blows should be recorded for each 2.5 cm (inch) of penetration.
3. Another version of the driven probe test involves the use of the "California Penetrometer." This is a cone penetrometer consisting of a 4 cm (1.6 in.) diameter rod with a 5.1 cm (2.3 in.) diameter expendable cone tip. This penetrometer is driven by a compressed air powered sheet pile hammer. The rate of penetration is recorded in seconds per 30 cm (foot) until refusal is attained or until the driving rate equals 180 seconds per 30 cm (foot) for the cone.

STATIC CONE PENETROMETER TESTS

ASTM Test Designation D3441-86 provides standards and procedures for completing the Cone Penetrometer Test (CPT).

Mechanical Cone

1. Set the thrust machine to the required test speed, generally between 1 to 2 cm/sec (2 to 4 ft/min.).
2. Prior to inserting the penetrometer into the ground, check that the assembled penetrometer tip and first push rod section are in straight alignment, the point and friction sleeve moves freely with respect to each other, and the inner rod moves freely within the push rod.

3. If testing in cohesive materials, it is advisable to use a friction reducer section, located directly above the penetrometer. This special rod has a diameter larger than the push rods and minimizes frictional resistance on the push rods when advancing to test depths.
4. Locate the thrust machine over the sounding location. If required, advance a borehole to the first test depth using conventional test boring techniques.
5. Attach the penetrometer to the length of push rods required for the first test depth.
6. Lower the penetrometer/rod assembly into the borehole.
7. Tests are typically completed at 20 cm (8 in.) intervals, but the interval between tests may be decreased to 10 cm (4 in) to obtain more detailed information.
8. Prior to each test increment, verify that the inner rod extends approximately 5 cm (2 in) above the top of the push rod. This condition indicates the penetrometer is fully collapsed and prepared for test.
9. Move the control level on the thrust machine to the test position and advance the inner rod at the preset test speed, such that the penetrometer tip is fully extended. Record the gauge reading, indicating the cone resistance.
10. If the cone has a friction sleeve, continue to advance the inner rods to engage the sleeve. Record the second gauge reading, indicating the combined resistance on the point and friction sleeve (Fig. C-4).
11. Move the control lever on the thrust machine to collapse the penetrometer tip and advance to the next test depth; repeat steps 9 and 10. Add additional rods as required.
12. At the completion of the tests, remove the loading head assembly and attach the rod pulling device to the thrust machine and remove penetrometer/rod assembly from the ground.
13. Complete the cone penetration report (Figure C-5).

Electrical Cone

1. Set the thrust machine to the required speed, generally between 1 to 2 cm/sec (2 to 4 ft/min).
2. Attach the penetrometer tip to the push rod sections (without inner rods). String the electric cable from the tip through the push rods and connect to the surface data recording system. (Note: It is preferred to prestring the cable through all push rods required for testing, so that at the completion of each 1 m (3.3 ft.) thrust, the data recording system does not have to be tampered with.)
3. If testing in cohesive soils, use a friction reducer section to minimize total friction resistance on push rods.
4. Locate the cone penetrometer thrust rig over the sounding location. If required, advance a borehole to the first test depth using conventional test boring techniques.
5. Lower the penetrometer/rod assembly into the borehole.
6. Advance the penetrometer tip at the preset test speed a distance of approximately 1 m (3.3 ft.). Simultaneously, record the continuous output of cone bearing and local sleeve friction.
7. Add additional push rods and repeat step 6.
8. At the completion of the tests, remove the penetrometer/rod assembly from the ground, using the rod pulling device.
9. Complete the cone penetration report (Figure C-5).

PRESSUREMETER TEST (MENARD TYPE)

The pressuremeter test consists of an expansion of a cylindrical cavity formed in the ground in order to measure the relationship between pressure and deformation for the soil. The pressuremeter test is extremely sensitive to borehole disturbance and all drilling procedures should minimize disturbance of the soil strata to be tested. It is recommended that experienced personnel supervise all pressuremeter field work.

Procedure

1. *Borehole Advancement:* The following procedures my be used as necessary to maintain borehole stability and minimize borewall disturbance:
   a) Wash boring techniques using flush-joint, steel casing may be driven to within 60 cm (2 ft.) of the zone to be tested. Uncased wash boring techniques require a weighted drilling mud to maintain borewall stability.
   b) Machine operated, continuous flight augers may be used in preparing the test borehole. The rate of auger advance and withdrawal must be very slow in order to minimize borewall disturbance.
   c) Large diameter, thin-wall tube samplers

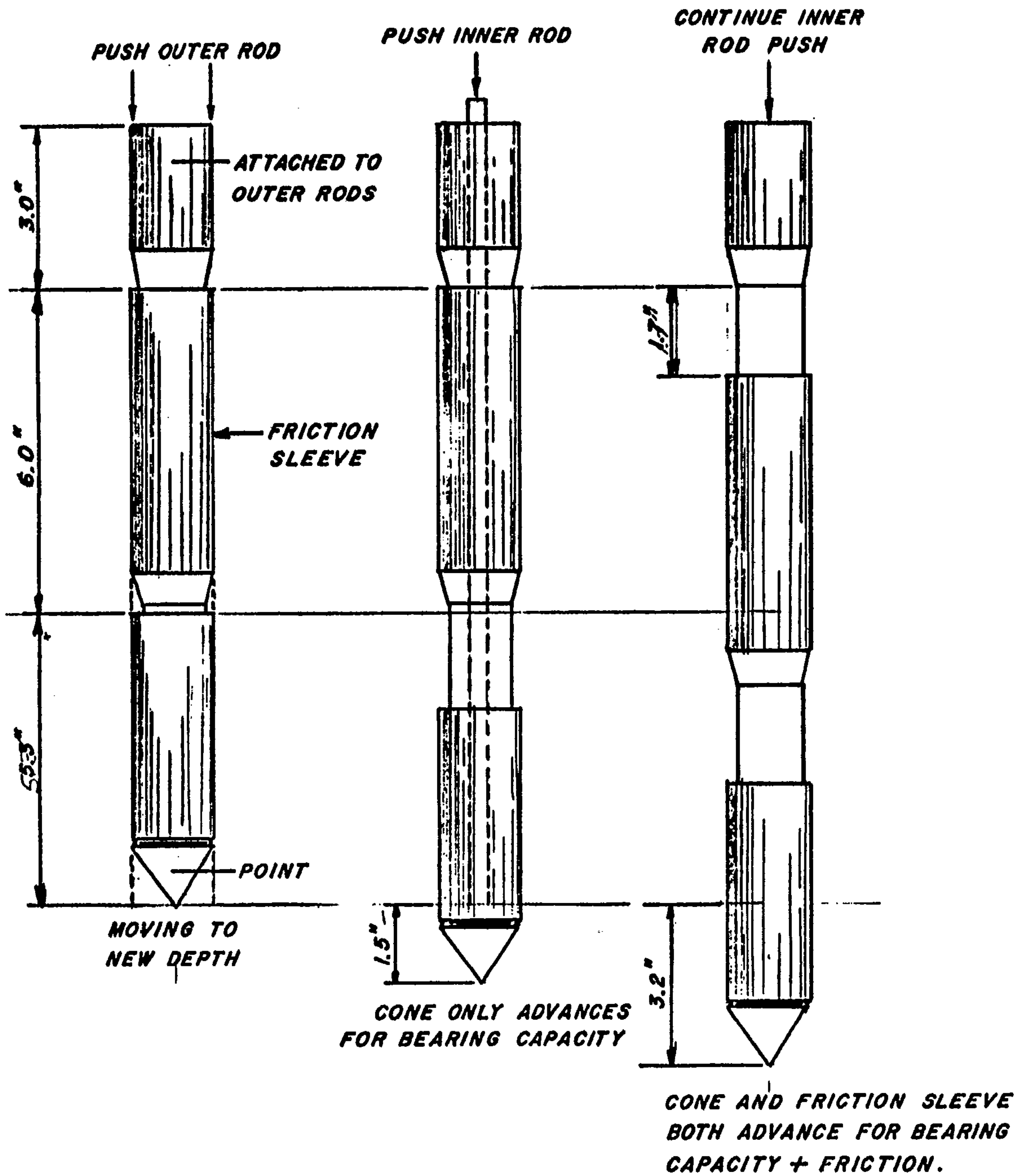


Figure C-4

MECHANICAL FRICTION CONE
(DEVELOPED BY BEGEMANN)

DUTCH CONE REPORT

HOLE NO. B101

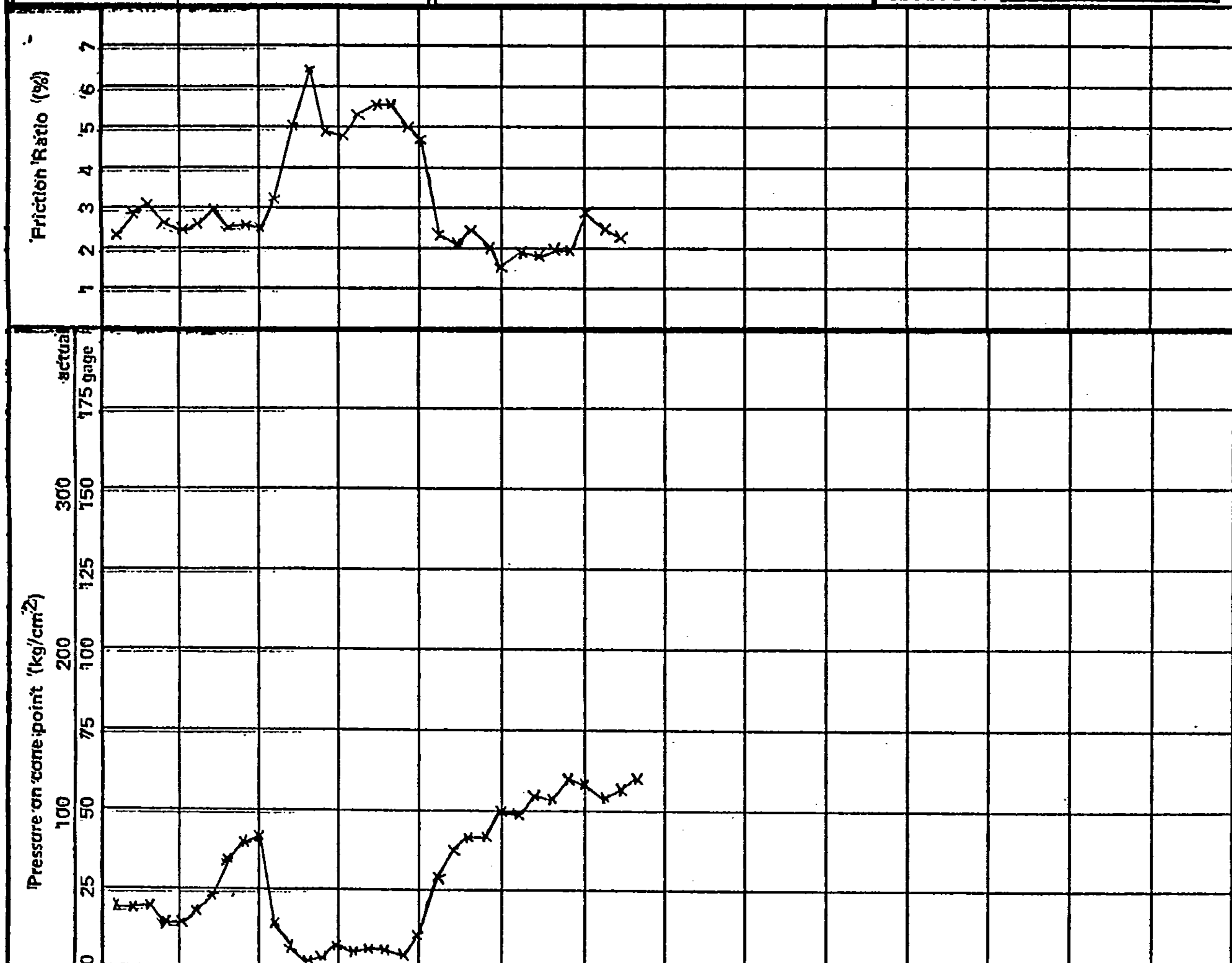
PROJECT: Proposed Finmark Building, Cambridge
CLIENT: Smith & Cullen, Boston
CONTRACTOR: XYZ Drilling & Boring Co., Boston

FILE NO. 679201
SHEET NO. 1 of 1
LOCATION: See Plan
ELEVATION: 16.0 MSL
DATE START: 6 Oct. '76
DATE FINISH: 8 Oct. '76
DRILLER: J. Brown
INSPECTOR:
GEOLOGIST: F. Smith

GROUNDWATER DATE	TIME	DEPTH TO: WATER	BOTTOM OF HOLE
7 Oct.	-	6.0	3.0'
9 Oct.	24 hr.	5.8	2.5'

ADDITIONAL INFORMATION

CONE TYPE Mechanical MECHANICAL ELECTRICAL
FRICTION SLEEVE Yes (Yes No)
RATE OF ADVANCE 2 cm/sec.



FIELD DATA

Gage Point = G	Point + Sleeve G + ΔG	ΔG Up 20cm	6.6ΔG = F	F/G % = Friction Ratio
20	—	7	46	2.3
18	25	8	53	2.9
19	27	8.5	56	3.0
15	23.5	6	39.5	2.6
14	20	5	33	2.4
18	23	7	46	2.6
23	30	10	66	2.9
35	45	13	86	2.5
40	53	16	105.5	2.6
42	58	16	105.5	2.5
14	30	7	46	3.3
6	13	4.5	29.5	5.0
3	7.5	3	19.5	6.5
4	7	3	19.5	4.9
7	10	5	33	4.7
5	10	4	26	5.3
6	10	5	33	5.5
6	11	5	33	5.5
4	9	3	19.5	4.9
10	13	7	46	4.6
30	37	10	66	2.2
37	47	11	52.5	2.1
41	52	15	99	2.4
42	57	12	79	1.9
50	62	12	79	1.6
48	60	14	92.5	1.9
55	69	15	99	1.8
54	69	16.5	109	2.0
60	76.5	17	112	1.9
57	74	25	165	2.9
55	80	20	132	2.4
57	77	20	132	2.3
60	80			

Depth Meters: 1 2 3 4 5 6 7

Figure C-5

are generally used to prepare the test area below the bottom of the casing. Tube samplers should be pressed, whenever possible and not driven into the undisturbed test zone. The tube sampler should not be twisted during removal.

d) Rotary core drilling techniques, utilizing drilling mud, are generally used for advancing the borehole into weathered or sound rock.

e) Extreme care must be exercised at all time for any borehole advancement method in order to minimize borehole disturbance in the desired test zone.

2. *Equipment*
The pressuremeter consists of three parts as illustrated in Figure C-6: the probe, the control unit, and the tubing:
   a) *The probe*
   Water is used to pressurize the cavity and to measure the resulting volume change. It is contained in a flexible, impervious rubber bladder which fills the cavity and defines its length. In Figure C-6 it is called the 'measuring cell.' To make sure that the cavity expands as it should, the measuring cell is flanked top and bottom by guard cells which are also rubber bladders and which are inflated by gas to the same pressure as the measuring cell. The inflated guard cells effectively seal off the borehole and prevent the measuring cell membrane from expanding into the void of the hole.
   b) *The control unit*
   The control unit is located at a convenient spot on the ground surface close to the hole, and its function is to control and monitor the expansion of the probe. It does this by applying a given pressure on command to the probe and then measuring the volume change of the measuring cell. The pressure source is a bottle of compressed gas; the flow of water to the measuring cell is monitored using a graduated cylinder which is called the volumeter.
   c) *The tubing*
   Tubing is required between the control unit and the probe to allow water and gas to be sent from one to the other.
3. *Testing*
The test is simply a process of simultaneously applying gas and water pressure at the volumeter through the tubing to the probe. When a desired pressure is reached, it is held constant for one minute and volume readings observed in the sight tube are recorded fifteen, thirty and sixty seconds after the pressure level is attained. Another increment of pressure is added at a steady rate of 1.5 kg/cm^2 (21 p.s.i.) per minute, and held constant at the next desired level, and fifteen, thirty and sixty second volume readings are again recorded at this new pressure. Between eight and fourteen pressure increments are used before advancing the test hole to the next deeper test level.

Field plots of "creep" volume should be made during the test; the creep volume is the volume change which occurs between the thirty and sixty second readings. The "creep" volume is generally low and fairly constant during the elastic or linear portion of the test. Unloading and reloading sequences may be performed, however, unloading should not be done until at least four points on the linear (elastic) portion of the volume vs. pressure plot have been attained.

A sample of the data form is presented as Figure C-7.

BOREHOLE SHEAR TEST (Iowa Type)

The borehole shear test employs a pair of corrugated plates attached to an expandable shear head. The plates are activated by gas pressure and pressed against the walls of a borehole. A constant pressure is applied and the shear plates are pulled upward.

Procedure

1. Prepare a smooth, flat and firm working area around the borehole which will accommodate the pulling device.
2. Advance a borehole to a point just above the required test depth by drilling or augering. Advance the borehole to the required test location by either careful hand augering, or pressing a 76 cm (3 in.) thin-walled tube sampler or driving a 76 mm (3 in.)-O.D. split-spoon sampler. The test will be performed in the resulting hole. Location of test must be at least 30 cm (1 ft.) below the bottom of the borehole casing, if casing is used.
3. Lower the shear head to the desired test depth, with the necessary length of pulling rods attached.
4. Place the base plate and attached mechanism over the rods extending from the borehole

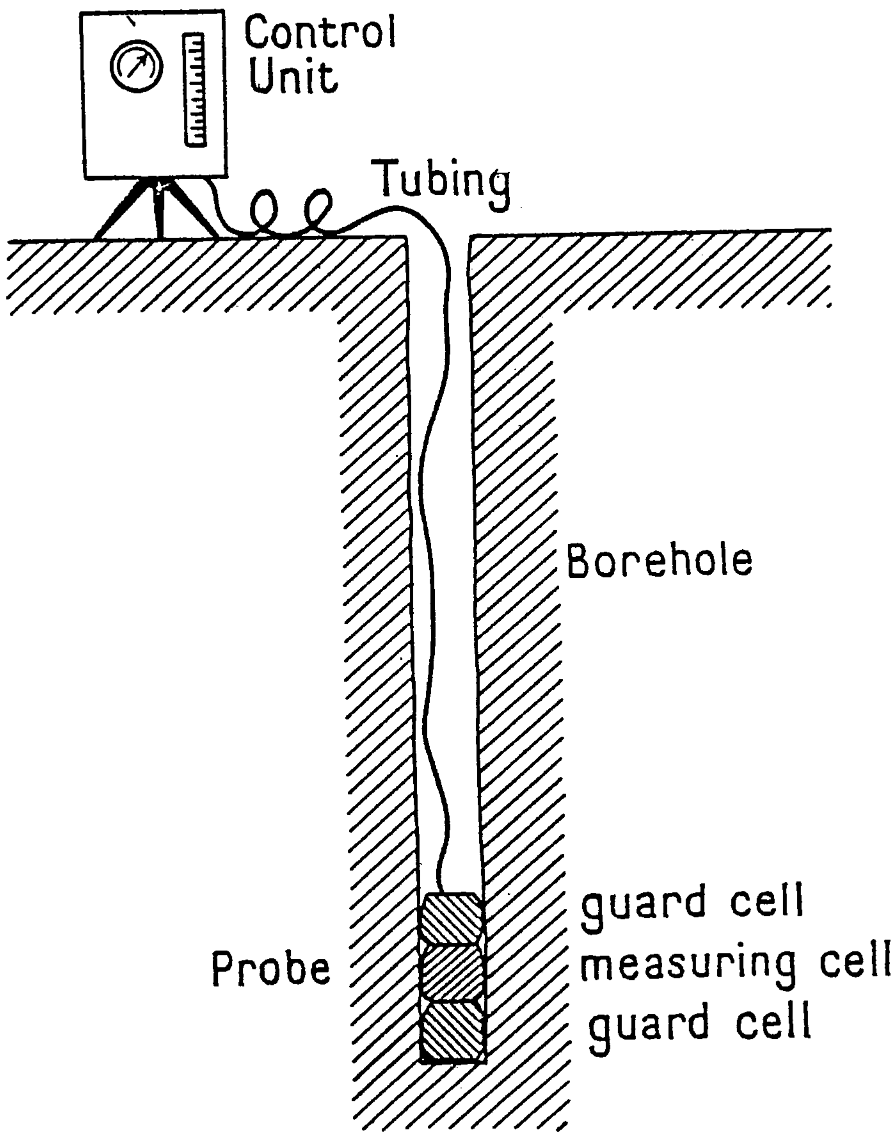


Figure C-6: Basic principles of the pressuremeter.

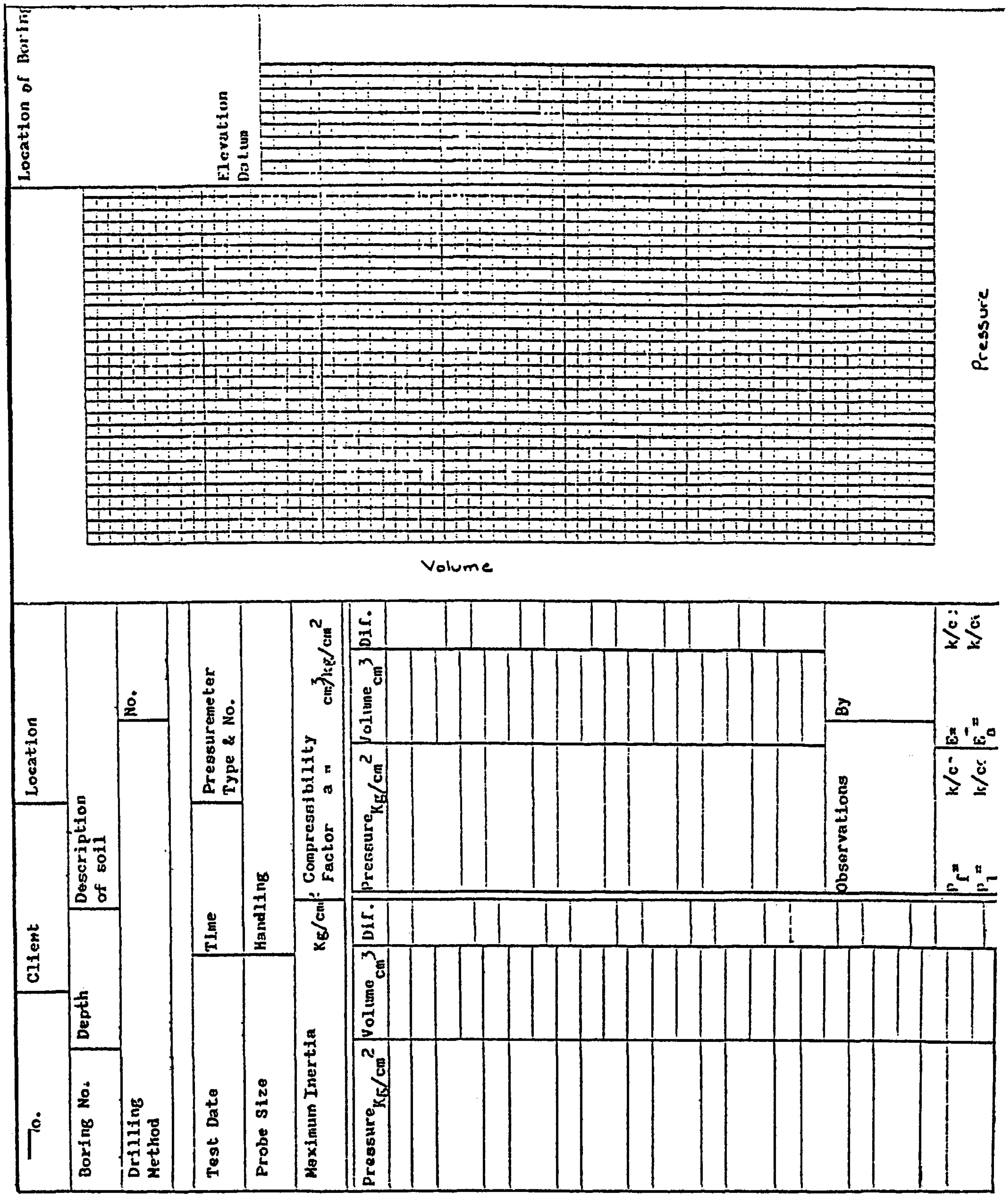

Location of Boring
Elevation
Datum
Pressure
Volume
No.
Client
Location
Boring No.
Depth
Description of soil
No.
Drilling Method
Test Date
Time
Pressuremeter Type & No.
Probe Size
Handling
Maximum Inertia Kg/cm²
Compressibility Factor a = cm³/kg/cm²
Pressure Kg/cm²
Volume cm³
Dif.
Observations
By

Figure C-7

and set the base plate on firm leveled ground. The pulling unit must be perpendicular to the axis of the borehole.

5. Complete assembly of the pulling unit. Make final adjustment of depth of shear head. Check for free movement of the pulling device. Clamp pulling rods in pulling device.
6. Connect the compressed gas lines from the shear head to the console.
7. Begin the test by applying the first desired normal pressure to the shear head using the regulator valve on the console.
8. Allow at least 10 minutes for initial consolidation of soil.
9. After the consolidation period, perform the shear test as follows:
   a) Turn the crank on the pulling device to pull the shear head upwards at a rate of 0.05 mm (0.002 in.) per second.
   b) At intervals of 30 seconds, record the pressure shown on the hydraulic gauge on the pulling device.
   c) Continue pulling the shear head at a constant rate until either an observable shear occurs, as evidenced by a drop in hydraulic gauge readings, or the pressure remains constant for at least 5 minutes.
   d) After the maximum shear force has been attained, unload the pulling rods by turning the crank in the opposite direction until the hydraulic gauge indicates 6.895 $\times$ 10^3 N/m^2 (1 p.s.i.) or less.
10. Increase the shear head normal pressure to the next desired level and allow the soil to consolidate for at least 5 minutes.
11. Repeat steps 9 and 10 for each normal pressure. Generally at least 5 normal pressure levels are performed per location to allow ready interpretation of the plotted area.
12. After completion of all tests at a given depth, turn the crank so as to reduce the pressure indicated on the hydraulic gauge to less than 6.895 $\times$ 10^5 N/m^2 (1 p.s.i.). Relieve the normal pressure on the shear head and remove the pulling device and shear head.
13. Advance the borehole to the depth at which the next test is required.
14. Plot the individual failure points (such as points 1-5 or Fig. B-9) and construct the Mohr envelope to obtain the angle of internal friction (0) and cohesion (c), if present.

A sample data form is presented as Figure C-8.

WATER PRESSURE TEST

Water pressure tests are performed *in situ*, within test borings, to measure the permeability of a soil or rock mass. Pressure testing helps to locate zones of leakage, measuring the capacity of such zones for transmitting water, and useful in estimating grouting and dewatering requirements for construction purposes.

Equipment

1. Pump, water (non-centrifugal preferred)
2. Meter, water (measures flow to 0.1 gallon)
3. Gauge, pressure (PSI; calibrated)
4. Packer system (pneumatic or mechanical)
5. Tank, surge (optional, depending on pump type)
6. Drill rig
7. Miscellaneous pipe and fittings
8. Valve, bypass (for regulating water pressures)
9. Hose, air, reinforced
10. Tank, nitrogen or oxygen, with pressure regulator and two gauges
11. Time piece
12. Ruler
13. Forms

Mechanical units are not as preferable, in terms of pressure sealing, as are pneumatic types. A surge tank is required only when centrifugal pumps are used to provide a constant and steady flow of water.

Procedure

1. Drill NX-size boring to required depth; surge to remove drilling debris.
2. Determine static water level prior to installation of packer.
3. Determine internal friction of fully assembled equipment, on the ground surface, by passing water through system at a rate of 38-76 litres/min (10-20 g.p.m.); record pressure gauge readings.
4. Determine Maximum Allowable Gauge Pressure (MGP) according t the formula below. In order to avoid hydrofracturing (loosening) the rock mass. *Do not* exceed MGP during testing.

   MGP = Maximum Gauge Pressure = ZK
   where
   Z = Depth (feet) from top of upper packer to ground surface
   K = 0.75 (constant)

IOWA BORE HOLE SHEAR TEST

Project: Keene - Rte. 9 Bridge Boring No: B-203 Test No. 3

Location STA 184+27; Right 59' Date 18 July 1978

Depth 26.5' below ground surface Horizon ______ Tested by ______

Description ______

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}		
1	12	2.5	6	1.2		
2	20	4.8	14	3.4		
3	30	7.4	19	4.8		
4	40	10.1	22	5.5		
5	50	12.9	6	1.2		

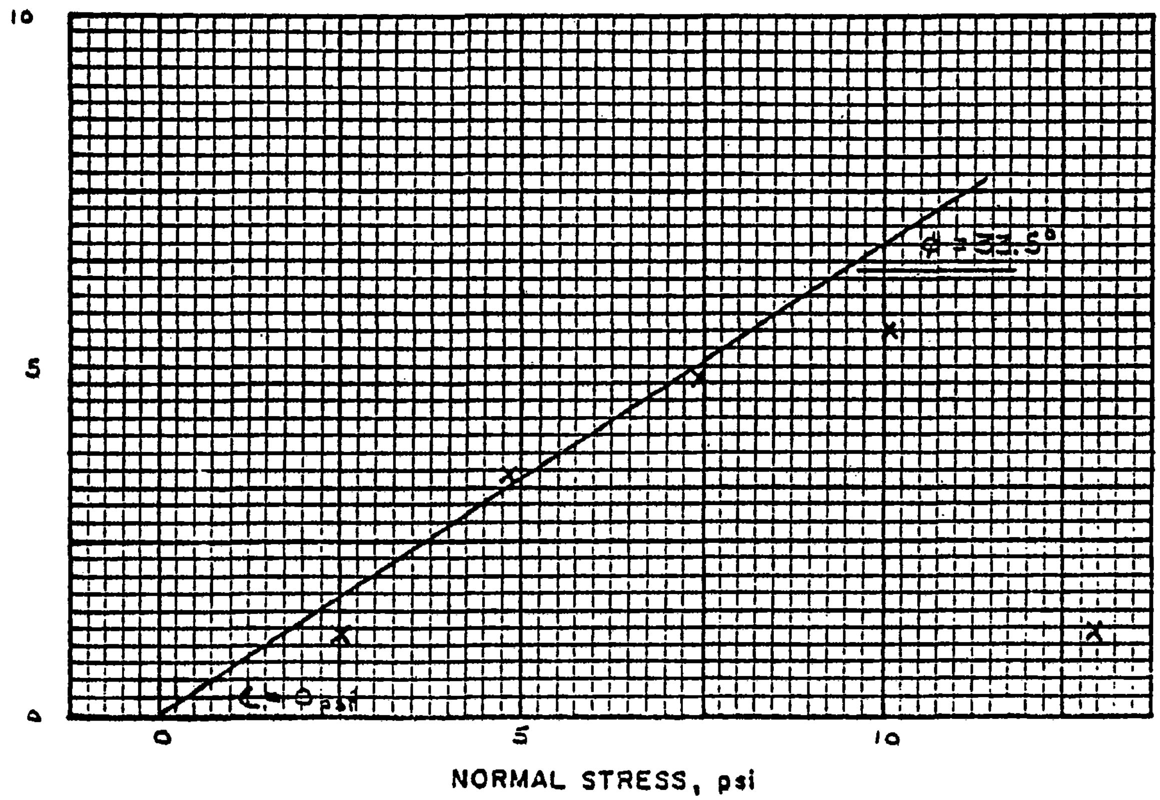


Figure C-8. Example Borehole Shear Test Data Sheet and Mohr Envelope Plot.

HALEY & ALDRICH, INC.. CAMBRIDGE, MASSACHUSETTS	ROCK PRESSURE TEST			HOLE NO. S1	TEST NO. 2
PROJECT: EXAMPLE				FILE NO. 1000	
CLIENT:				SHEET NO. 1 of 2	
CONTRACTOR: C. L. GUILD DRILLING & BORING CO.				LOCATION: See Plan	
	PACKER SYSTEM	SURGE CHAMBER	PRESSURE GAUGE	ELEVATION: 50.3	
				DATE START: 2-10-70	
TYPE	Pneumatic	Mini Trol	Ascheraft 100	DATE FINISH: 2-10-70	
MANUFACTURER	Damco Div. Ingersol Rand	Amtrol, Inc.	Dresser	DRILLER: Joe Neal	
				INSPECTOR: J. Infascelli	
MODEL	H2750 Double Packer	500	$3\frac{1}{2}$", 1010	GEOLOGIST: R. McClary	

HOLE SIZE: O.D. 2-15/16" - NXM
COMPUTED MAX GAUGE PRESS: 30 P.S.I.
COMPUTED INTERNAL FRICTION: 0

ROCK TYPE: Gray medium-fine grained GRANITE
RECOVERY (%) 78 - 86
R Q D (%) 30 - 35

DEPTHS: (All Distances Measured From Ground Surface In Feet)

TO TOP OF ROCK 14
TO TOP LOWER PACKER 58.5
TO BOTTOM OF BORING 70
TO BOTTOM UPPER PACKER 51.9
TO WATER TABLE 20.2
LENGTH OF TEST SECTION 6.6

NOTE: HEIGHT OF PRESSURE GAUGE ABOVE GROUND LEVEL = 1.6'

TIME	ELAPSED TIME (MIN)	PACKER PRESSURE (PSI)	GAUGE PRESSURE (PSI)	METER READING (GALS)	VOLUME OF FLOW (GALS/MIN)	REMARKS
1123	0:00	150	5	258.3		
	0:30			260.8		
	1:00			262.3		
	1:30			262.5		
	2:00			262.7		
1125	0:00		10	263.7		
	0:30			265.5		
	1:00			266.5		
	1:30			267.1		
	2:00			267.5		
1127	0:00		15	268.9		
	0:30			269.5		
	1:00			271.4		
	1:30			272.5		
	2:00			273.0		
1129	0:00		20	275.0		
	0:30			276.1		
	1:00			277.1		
	1:30			280.7		
	2:00			282.3		
1131	0:00		25	284.5		
	0:30			285.7		
	[illegible]			[illegible]7.5		

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Figure C-9

ROCK PRESSURE TEST

Page 2 Of 2

Project EXAMPLE Boring No. S1 Test No. 2

TIME	ELAPSED TIME (MIN)	PACKER PRESSURE (PSI)	GAUGE PRESSURE (PSI)	METER READING (GALS)	VOLUME OF FLOW (GALS/MIN)	REMARKS
	1:30	150	25	288.1		
	2:00			289.2		
1133	0:00		30	291.6		
	0:30			294.8		
	1:00			296.9		
	1:30			299.0		
	2:00			301.5		
	2:30			304.6		
	3:00			305.7		
	3:30			306.1		
	4:00			306.5		306.6 Gal.
	4:30			306.5		289.2 Gal.
	5:00			306.6		17.4 Gal.
	5:30			306.6		
	6:00			306.6		
	6:30			306.5		$\frac{17.4 \text{ Gal.}}{6 \text{ Min.}} = 2.9$ GPM
	7:00			306.6		
1140						

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HALEY & ALDRICH, INC.
H&I, 1r 27, rev. 0

Figure C-10

6. Lower test apparatus to specified test depth, inflate both packers to at least 1.02×10^6 N/M^2 (150 p.s.i.). Double packers are usually spaced at 1.5 M (5 ft.), but spacing can be varied to meet specific test requirements.
7. Before starting test, record the following:
   a) Test number
   b) Test section
   c) Hole size
   d) Height of pressure gauge above ground surface
   e) Ground surface elevation
   f) Depths to rock surface, groundwater, bottom of boring, bottom of upper packer and top of lower packer.
8. Pump water into the system. Attain and hold pressure at 1/3 and 2/3 of the MGP for 3 minutes, taking flow readings every thirty seconds. Attain and hold MGP for 10 minutes taking flow readings every minute.
9. If leakage of water, from the packed section, into the surrounding rock is so great that the MGP cannot be reached, run the pump at its full capacity with the by-pass valve closed; record the amount of water pumped into the test section, at 30 second intervals, with associated pressure readings.
10. Upon completion of test, deflate the packers and move to the next test depth. Complete data sheets, Figures C-9 and C-10.

Rock Mass Permeability is computed from records made on the test forms and evaluated as follows*:

Very Low, equivalent to clay	Less than 1×10^{-7}cm/sec
Low, equivalent to silt	1×10^{-5} to 1×10^{-7}cm/sec
Medium, equivalent to fine sand	1×10^{-4} to 1×10^{-5}cm/sec
High, equivalent to sand	1×10^{-2} to 1×10^{-4}cm/sec
Very High, equivalent to clean sand or gravel	More than 1×10^{-2}cm/sec

* From a scheme developed by G. S. Brierley, Haley & Aldrich, Inc., 1976.

APPENDIX D
Laboratory Testing Procedures—Soils and Rock

This summary of test procedures, discussed in Section 9 of the body of this Manual, references standard methods of AASHTO and ASTM. Where there are no standards, commonly accepted procedures are referenced.

This summary is not intended to be all inclusive of the variety of tests currently in use in all parts of the country. It is intended to reference only those basic tests which are in common use. A detailed discussion of complex methods which are not in common use is beyond the scope of this Manual. Deviations from standard methods or acceptable procedures may be necessary on occasion dependent on local practice, subsurface conditions and project requirements and peculiarities of test procedures or equipment.

Test data sheets are not included with the referenced procedures because specific forms vary considerably among organizations. Laboratories should adopt whatever data sheets are most suited to their experience, practices and equipment.

Abbreviations as follows, are used when referring to standards or methods. The complete citations for these standards or methods are shown in "References for Appendix D."

T—AASHTO
D—ASTM
STP—ASTM Special Technical Publication
EM—Engineer Manual, EM 1110-2-1906, "Laboratory Soils Testing" (U.S. Army Corps of Engineers)
L—Lambe, "Soil Testing for Engineers"
H—Hvorslev, "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes"
WES—Waterways Experiment Station, (U.S. Army Corps of Engineers)
B and H—Bishop and Henkel, "The Measurement of Soil Properties in the Triaxial Test"
FHWA—Federal Highway Administration (U.S. Department of Transportation)
ASCE—American Society of Civil Engineers, "Earthquake Engineering and Soil Dynamics"
S—Shore Scleroscope Bulletins, S-34, S-35
AFWL—Air Force Weapons Laboratory, AFWL TR-65-116, "Engineering Classification and Index Properties for Intact Rock"
F—Forney, Inc., Concrete Test Hammer, Type L

Procedures

Sample Handling

H—Chapter 6, Disturbance of Soil Samples, Chapter 16, Preservation and Handling of Samples
EM—Introduction, Sample Handling and Storage, Selection and Preparation of Test Specimens
L—Chapter 1, Introduction

Unified Soil Classification System

WES—Technical Memorandum No. 3-357, The Unified Soil Classification System
D2487—Classification of Soils for Engineering Purposes
D2488—Description of Soils (Visual–Manual Procedure)

Moisture Content

D2216—Moisture Content of Soil, Laboratory Determination

Grain Size Analysis

T88—Particle Size Analysis of Soils
D422—Particle-Size Analysis of Soils
L—Chapter IV, Grain Size Analysis

Atterberg Limits

Liquid Limit
T89—Determining the Liquid Limit of Soils
D423—Liquid Limit of Soils
L—Chapter III, Atterberg Limits and Indices.

Plastic Limit and Plasticity Index
T90—Determining the Plastic Limit and Plasticity Index of Soils
D424—Plastic Limit and Plasticity Index of Soils
L—Chapter III, Atterberg Limits and Indices.

Shrinkage Limit
T92—Determining the Shrinkage Factors of Soils
D427—Shrinkage Factors of Soils
L—Chapter III, Atterberg Limits and Indices.

Specific Gravity

T100—Specific Gravity of Soils
D854—Specific Gravity of Soils
L—Chapter II, Specific Gravity Test.

Direct Shear

T236—Direct Shear of Soils Under Consolidated Drained Conditions
D3080—Direct Shear Tests of Soils Under Consolidated Drained Conditions
L—Chapter X, Direct Shear Test on Cohesionless Soil
L—Chapter XIV, Direct Shear Test on Cohesive Soil
EM—Appendix IX, Drained Direct Shear Test

Unconfined Compression Test

T208—Unconfined Compressive Strength of Cohesive Soil
D2166—Unconfined Compressive Strength of Cohesive Soil
L—Chapter XII, Unconfined Compression Test
EM—Appendix XI, Unconfined Compression Test

Triaxial Compression Test

T234—Strength Parameters of Soils by Triaxial Compression
D2850—Unconsolidated, Undrained Strength of Cohesive Soils in Triaxial Compression
B and H—The Measurement of Soil Properties in the Triaxial Test
EM—Appendix X, Triaxial Compression Tests

Consolidation Test

T216—One-Dimensional Consolidation Properties of Soils
D2435—One-Dimensional Consolidation Properties of Soils
EM—Appendix XIII, Consolidation Test
L—Chapter IX, Consolidation Test

Constant Head Permeability Test

T215—Permeability of Granular Soils (Constant Head)
D2434—Permeability of Granular Soils (Constant Head)
EM—Appendix VII, Permeability Tests

Falling Head Permeability Test

EM—Appendix VII, Permeability Tests
L—Chapter VI, Permeability Tests

Soil Suction Test

FHWA—Research Report FHWA-RD-79-51, Soil Suction Test Procedure Using Thermocouple Psychrometers.
EM—Appendix VIII A, Swell and Swell Pressure Tests

Moisture and In-Place Density

T191—Density of Soil In-Place by the Sand-Cone Method
T205—Density of Soil In-Place by the Rubber-Balloon Method
D1556—Density of Soil In-Place by the Sand Cone Method
D2167—Density of Soil In-Place by the Rubber-Balloon Method
D2937—Density of Soil In-Place by the Drive-Cylinder Method

Compaction

T99—Moisture-Density Relations of Soils Using a 5.5 lb. Rammer and a 12-in. Drop
T189—Moisture Density Relations of Soils Using a 10-lb. Rammer and an 18-in. Drop
D698—Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb. Rammer and 12-in. Drop
D1557—Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb. Rammer and 18-in. Drop
D2168—Calibration of Mechanical Laboratory Soil Compactors
D2049—Relative Density of Cohesionless Soils

Dynamic Properties

STP—654, Dynamic Geotechnical Testing
EM—Appendix X A, Cyclic Triaxial Test
ASCE—Proceedings, Specialty Conference on Earthquake Engineering and Soil Dynamics.

Rock Tests

D448—Scleroscope Hardness Testing of Metallic Materials
S—Shore Scleroscope Bulletins S-34, S-35
AFWL—Engineering Classification and Index Properties for Intact Rock.
F—Forney, Inc., Concrete Test Hammer, Type L, Operating Instructions
D2938—Unconfined Compressive Strength of Intact Rock Core Specimens

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APPENDIX E
Materials Classification

One of the basic objectives of geotechnical investigations is to define the nature, characteristics, properties, thickness and lateral extent of bedrock and overlying soils within a given project area. The process of soil formation (Fig. E-1) are influenced mainly by parent rock lithology, climate, topography, time, and geologic history. In order to define and describe the nature of the subsurface conditions, it is necessary to adopt standard classification systems for both the soil and the underlying rock.

The purpose of any soil classification system is to place soils having the same characteristics into a given category. Each category should describe the soil type in terms of grain size and plasticity, and at the same time, provide some qualitative idea of the range of engineering properties or characteristics of a given soil type (Fig. E-2).

The soil type within the given system must be described in terms easily understood and readily recognized by drilling foremen, field and laboratory technicians, geologists and geotechnical engineers. Simple and basic visual, field and laboratory techniques must be available to assist in classifying the various soil types, including borderline categories.

To be of most value, a classification system should be standardized and be applicable for all regions of North America as well as other areas. From the various classification systems that exist, the two classification systems most widely used in the engineering field are the AASHTO Soil Classification system and the Unified Soil Classification system. These two systems will be described in this section.

Some agencies such as the South Dakota DOT, have found that soil classification data may be useful after completion of a project and have developed computer retrieval systems (Crawford and Thomas, 1973). Since most soil materials formed or deposited under similar conditions are usually grouped into similar units or receive comparable names, the range of values for quantifiable soil properties are often reasonably narrow. A computer-based retrieval system can be easily programmed to perform statistical calculations and plots as well as to retrieve soil data from locations near a proposed project. The South Dakota DOT found that the soils are best categorized by location, depth, particle size gradation characteristics, liquid limit, plasticity index, maximum dry density, optimum moisture content and Munsell color.

E.1 AASHTO Soil Classification System

The American Association of State Highway and Transportation Officials (AASHTO) system of soil classification–AASHTO Designation: M 145-(AASHTO, Part I, 1986)–is based upon the observed field performance of subgrade soils under highway pavements. The original system was developed by the U.S. Bureau of Public Roads about 1928, and has been revised several times over the years to its present form. This system is widely used by highway engineers in the United States and other parts of the world.

In this system, soils having approximately the same general load-carrying capacity and service characteristics are grouped together to form seven basic groups which are designated as A-1 through A-7. In general, the best soils are classified as A-1, with the soils becoming progressively poorer as you proceed to A-7. The exception is A-3, which are better subgrade soils than A-2. An additional group A-8 is used to designate organic soils.

E.1.1 Classification

The procedure for classifying soils into the seven basic groups (A-1 to A-7), or further subdividing into the subgroups resulting in twelve categories (A-1-a to

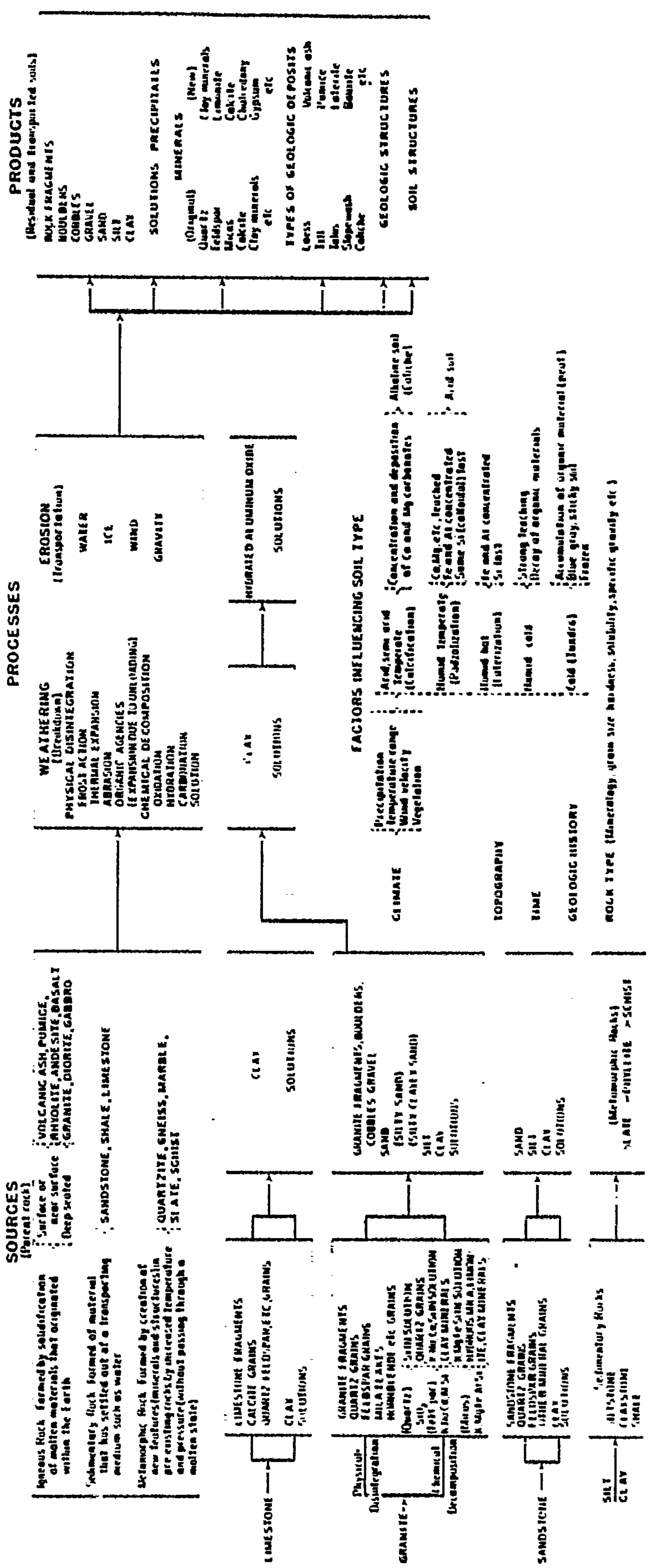


Figure E-1. Schematic diagram of the geologic origins of engineering soil types and the processes by which the various engineering soil types are produced (From Holtz, W.G., 1969).

SOIL GROUP	IMPORTANT PROPERTIES				RELATIVE DESIRABILITY FOR VARIOUS USES *						
					ROADWAYS				FOUNDATIONS		
						FILLS					
	PERMEABILITY WHEN COMPACTED	SHEARING STRENGTH WHEN COMPACTED AND SATURATED	COMPRESSIBILITY WHEN COMPACTED AND SATURATED	WORKABILITY AS A CONSTRUCTION MATERIAL	EMBANKMENT	FROST HEAVE NOT POSSIBLE	FROST HEAVE POSSIBLE	SURFACING	SEEPAGE IMPORTANT	SEEPAGE NOT IMPORTANT	EROSION RESISTANCE
GW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	—	1	1	3	—	1	1
GP	VERY PERVIOUS	GOOD	NEGLIGIBLE	GOOD	—	3	3	—	—	3	2
GM	SEMIPERVIOUS TO IMPERVIOUS	GOOD	NEGLIGIBLE	GOOD	2	4	9	5	1	4	4
GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	1	5	5	1	2	6	3
SW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	—	2	2	4	—	2	6
SP	PERVIOUS	GOOD	VERY LOW	FAIR	—	6	4	—	—	5	7 A
SM	SEMIPERVIOUS TO IMPERVIOUS	GOOD	LOW	FAIR	4	8	10	6	3	7	8 A
SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	3	7	6	2	4	8	5
ML	SEMIPERVIOUS TO IMPERVIOUS	FAIR	MEDIUM	FAIR	6	10	11	—	6	9	—
CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	5	9	7	7	5	10	9
OL	SEMIPERVIOUS TO IMPERVIOUS	POOR	MEDIUM	FAIR	8	11	12	—	7	11	—
MH	SEMIPERVIOUS TO IMPERVIOUS	FAIR TO POOR	HIGH	POOR	9	12	13	—	8	12	—
CH	IMPERVIOUS	POOR	HIGH	POOR	7 C	13 C	8 C	—	9 C	13 C	10
OH	IMPERVIOUS	POOR	HIGH	POOR	10	14	14	—	10	14	—
PT	—	—	—	—	—	—	—	—	—	—	—

NOTES:

* - No. 1 is best A - If gravelly. B - Erosion critical. C - Volume change critical.

Figure E-2. Generalized engineering properties and desirability of various types of soils for foundations, roadways, and embankments (adapted from Holtz, W.G., 1969).

A-7-6), is based on performing laboratory tests on samples of the soil. Refer to Table E-1. The laboratory tests performed include the determination of particle size distribution, liquid limit and plasticity index. Standard AASHTO or ASTM testing procedures can be used to obtain the necessary data for classification.

Highly organic soils (peat or muck) may be classified in an A-8 group (not listed in Table E-1). Classification is based on visual inspection. The material is composed primarily of partially decayed organic matter, generally has a fibrous texture, dark brown or black color and odor of decay.

The evaluation of soils within each group is made by means of a "group index" which is a value calculated from an empirical formula. The group or subgroup classification including group index should be useful in determining the relative quality of the soil material for use in earthwork structures, particularly embankments, subgrades, subbases, and bases. However, for the detailed design of important structures, additional data concerning strength or performance characteristics of the soil under field conditions will usually be required.

E.1.1.1 Soil Fraction Definitions. According to the AASHTO system, soils are divided into two major groups as shown in Table E.1. These are the granular materials with 35 percent or less passing the 0.075 mm (No. 200) sieve, and the silt-clay materials with more than 35 percent passing the 0.075 mm (No. 200) sieve. In subsequent word descriptions of the various soil classes, five soil fractions are defined as follows:

Table E-1. Classification of Soils and Soil-Aggregate Mixtures (AASHTO, 1986)

GENERAL CLASSIFICATION	GRANULAR MATERIALS (35% or less passing 0.075 mm)							SILT-CLAY MATERIALS (more than 35% passing 0.075 mm)			
	A-1			*A-2*							*A-7*
Group Classification	*A-1-a*	*A-1-b*	*A-3*	*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*	*A-4*	*A-5*	*A-6*	*A-7-5, A-7-6*
Sieve analysis: percent passing:											
2.00 mm (No. 10)	50 max.	—	—	—	—	—	—	—	—	—	—
0.425 mm (No. 40)	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
0.075 mm (No. 200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit	—		—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.[1]
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

[1] Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

Boulders: material retained on the 75 mm (3 in.) sieve. They should be excluded from the portion of a sample to which the classification is applied, but the percentage of such material should be recorded.
Gravel: material passing sieve with 75 mm (3 in.) square openings and retained on the 2.00 mm (No. 10) sieve.
Course Sand: material passing the 2.00 mm (No. 10) sieve and retained on the 0.425 mm (No. 40) sieve.
Fine Sand: material passing the 0.425 mm (No. 40) sieve and retained on the 0.075 mm (No. 200) sieve.
Combined Silt and Clay: material passing the 0.075 mm (No. 200) sieve. The term "silty" is applied to fine material having a plasticity index of 10 or less, and the term "clayey" is applied to fine material having a plasticity index of 11 or greater.

E.1.1.2 Classification Procedure. With required test data available, proceed from left to right in Table E.1. The first group in which the test data will fit is the correct classification. All limiting test values are shown as whole numbers. If fractional numbers appear on test reports, convert to nearest whole number for purposes of classification. Group index values should always be shown in parentheses after group symbol as: A-2-6(3), A-4(5), A-6(12), and A-7-5(17).

E.1.1.3 Group Index Determination. The group index is calculated from the following formula:

Group Index (GI) = (F − 35) [0.2 + 0.005 (LL − 40)] + 0.01 (F − 15) (PI − 10), in which
F = percentage passing 0.075 mm (No. 200) sieve, expressed as a whole number. This material is based only on the material passing the 75 mm (3 in.) sieve.
= Liquid Limit
= Plasticity Index

When the calculated group index is negative, the group index shall be reported as zero (0). The group index shall be reported to the nearest whole number. Figure E-3 may be used in estimating the group index, by determining the partial group index due to liquid limit and that due to plasticity index, then obtaining the total of the two partial group indexes. When calculating the group index of A-2-6 and A-2-7 subgroups, only the PI portion of the formula (or of Fig. E-4) shall be used. An example is included in Figure E-4. Other examples of calculating the Group Index are shown in subsection E.1.1.4.

E.1.1.4 Examples of Group Index Calculations (AASHTO, 1986).

- Assume that an A-6 material has 55 percent passing the 0.075 mm sieve, liquid limit of 40, and plasticity index of 25. Then
  Group index = (55 − 35) [0.2 + 0.005 (40 − 40)] + 0.01 (55 − 15) (25 − 10)
  = 4.0 + 6.0 = 10
- Assume that an A-7 material has 80 percent passing the 0.075 sieve, liquid limit of 90, and plasticity index of 50. Then,
  Group index = (80 − 35) [0.2 + 0.005 (90 − 40)] + 0.01 (80 − 15) (50 − 10)
  = 20.3 + 26.0, or 46.3 (Report as 46)
- Assume that an A-4 material has 60 percent passing the 0.075 mm sieve, liquid limit of 25, and plasticity index of 1. Then
  Group index = (60 − 35) [0.2 + 0.005 (25 − 40)] + 0.01 (60 − 15) (1 − 10)
  = 25 × (0.2 — 0.075) + 0; .01 (45) (—9)
  = 3.1 = 4.1 = —1.0 (Report as 0)
- Assume that an A-2-7 material has 30 percent passing the 0.075 mm sieve, liquid limit of 50, and plasticity index of 30. Then
  Group index = 0.01 (30 − 15) (30 − 10)
  = 3.0 or 3 (Note that only the PI portion of formula was used)

E.1.2 Description of Classification Groups

E.1.2.1 Granular Materials. Includes materials passing the 75 mm (3 in.) sieve and containing 35 percent or less passing 0.075 mm (No. 200) sieve. (Note 1)

Group A-1. Well graded mixtures of stone fragments or gravel ranging from course to fine with a nonplastic or slightly plastic soil binder. However, this group also includes coarse materials without soil binder.

Subgroup A-1-a. Materials consisting predomi-

Note 1. Any specifications regarding the use of A-1, A-2, or A-3 materials in construction should state whether boulders (retained on the 3-in. sieve) are permitted.

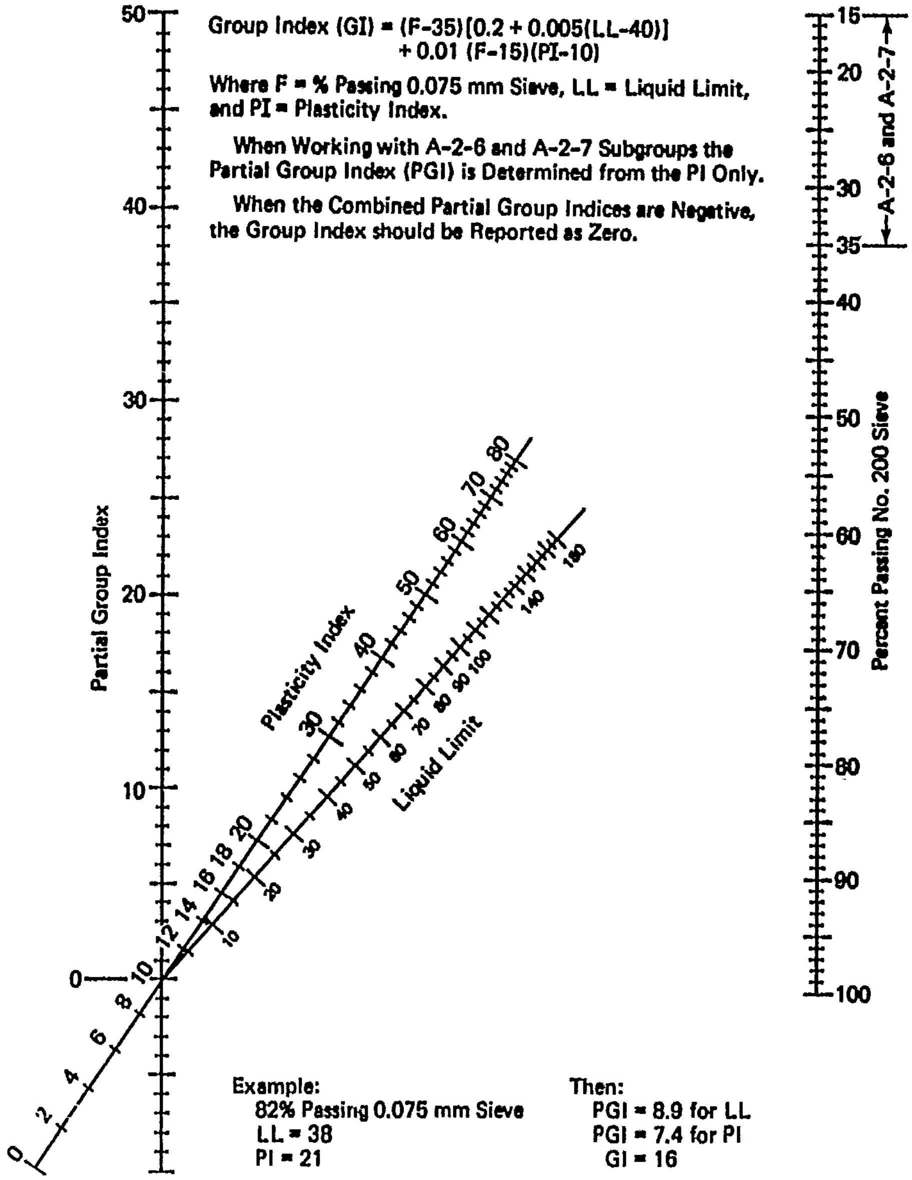


Figure E-3. Group Index Chart (AASHTO, 1986)

nantly of stone fragments or gravel, either with or without a well-graded soil binder.

Subgroup A-1-b. Materials consisting predominantly of coarse sand either with or without a well-graded soil binder.

Group A-3. Materials consisting of sands deficient in coarse material and soil binder. Typical is fine beach sand or fine desert blow sand, without silt or clay fines or with a very small amount of nonplastic silt. This group also includes stream-deposited mixtures of poorly-graded fine sand and limited amounts of coarse sand and gravel.

Group A-2. This group includes a wide variety of "granular" materials that are borderline between the materials falling in Groups A-1 and A-3 and the silt-clay materials of Groups A-4, A-5, A-6, and A-7. It includes all materials containing 35 percent or less passing the 0.075 mm (No. 200) sieve which cannot be classified as A-1 or A-3, due to fines content or plasticity or both, in excess of the limitations for these groups.

Subgroups A-2-4 and A-2-5. Include various granular materials containing 35 percent or less passing the 0.075 mm (No. 200) sieve, and with that portion passing 0.425 mm (No. 40) sieve having the characteristics of the A-4 and A-5 groups. These groups include such materials as gravel and coarse sand with silt contents or plas-

UNIFIED SOIL CLASSIFICATION				
MAJOR DIVISIONS			GROUP SYMBOLS ⓤ	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is larger than No 200 sieve size	GRAVELS More than half of coarse fraction is larger than No 4 sieve size	CLEAN GRAVELS	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.
			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.
	SANDS More than half of coarse fraction is smaller than No 4 sieve size.	CLEAN SANDS	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands, gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS More than half of material is smaller than No 200 sieve size.	SILTS AND CLAYS Liquid limit less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silt-clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

ⓤ Boundary classifications. Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.

Figure E-4. Soil Classification System (USCS) grain-size and liquid limit determinations used to classify soils (From Holtz, W.G., 1969).

ticity indexes in excess of the limitations of Group A-1, and fine sand with nonplastic silt content in excess of the limitations of Group A-3.

Subgroups A-2-6 and A-2-7. Include materials similar to those described under Subgroups A-2-4 and A-2-5, except that the fine portion contains plastic clay having the characteristics of the A-6 or A-7 group.

E.1.2.2 Silt-Clay Materials. Containing more than 35 percent passing the 0.075 mm (No. 200) sieve.

Group A-4: The typical material of this group is a nonplastic or moderately plastic silty soil usually having the 75 percent or more passing the 0.075 mm (No. 200) sieve. The group includes also mixtures of fine silty soil and up to 64 percent of sand and gravel retained on the 0.075 mm (No. 200) sieve.

Group A-5: The typical material of this group is similar to that described under Group A-4, except that it is usually of diatomaceous or micaceous character and may be highly elastic as indicated by the high liquid limit.

Group A-6: The typical material of this group is a plastic clay soil usually having 75 percent or more passing the 0.075 mm (No. 200) sieve. The group includes also mixtures of fine clayey soil and up to 64 percent of sand and gravel retained on the 0.075 mm (No. 200) sieve. Materials of this group usually have high volume change between wet and dry states.

Group A-7: The typical material of this group is similar to that described under Group A-6, except that it has the high liquid limits characteristic of the A-5 group and may be elastic as well as subject to high volume change.

Subgroup A-7-5: Includes those materials with moderate plasticity indexes in relation to liquid limit and which may be highly elastic as well as subject to considerable volume change. Plasticity Index = or < (LL - 30).

Subgroup A-7-6: Includes those materials with high plasticity indexes in relation to liquid limit and which are subject to extremely high-volume change. Plasticity Index > (LL - 30).

E.2 UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

The Unified Soil Classification System (USCS) is based upon the sizes of particles, the distribution of the particle sizes, and the properties of the fine-grained portion of the soil. The elements of the USCS indicate general properties and desirability for various engineering uses (Fig. E-4). Details of the system are summarized on Figure E-5.

The Unified Soil Classification System was developed by Casagrande in the early 1930's (Casagrande, 1948). With minor modifications it has been adopted by the Corps of Engineers and the Water and Power Resources Service (formerly U.S. Bureau of Reclamation). The Unified Soil Classification System has been revised and updated from time to time (ASTM D2487-85). The system distinguishes between three broad groups of soils: (1) coarse-grained soils, comprising gravel and gravelly soils, sands and sandy soils, which are distinguished on the basis of grain size composition and plasticity of the binder (if present); (2) fine grained soils, comprising all types of soils containing more than 50 percent by weight finer than 0.074 mm in size, except those containing high percentages of fibrous organic matter, such as peat. The fine-grained soils are distinguished on the basis of the presence or absence of organic matter and the interrelation between plasticity index and liquid limit.

Coarse-grained soil (sand and gravel) is that material retained on a No. 200 sieve, or having particle sizes larger than 0.074 millimeter. The smallest size in this category is about the smallest particle size which can be distinguished with the naked eye.

Fine-grained soil (silt and clay) is that material passing a No. 200 sieve, or having particle sizes smaller or finer than 0.074 mm.

Highly organic soils are peat or other soils which contain substantial amounts of organic matter. No laboratory criteria exist for the highly organic soils; however, they can generally be identified in the field by their distinctive color and odor and by their spongy feel and fibrous texture.

Only particle sizes 76m (3 in.) or less are considered in USCS. Fragments which are larger than 76 mm (3 in.) are classified as cobbles or, if larger than 203 mm (8 in.), boulders.

Soils can be USCS classified by simple laboratory procedures. However, with practice and experience, it is possible to accurately identify a soil in the USCS by visual means, supplemented by manual tests described later in this section.

The specific details of the USCS contained in Figure E-5 are described below:

E.2.1 Coarse-Grained Soils

The two major divisions of coarse-grained soils are gravel and sand. A coarse-grained soil having more than 50 percent of the coarse-grained fraction (fraction retained on No. 200 sieve) retained on No. 4 sieve is classified as gravel, and it is denoted by the symbol

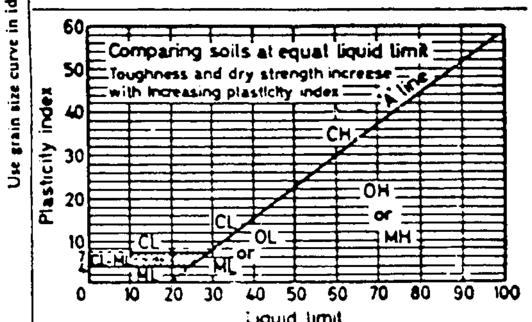
UNIFIED SOIL CLASSIFICATION SYSTEM

Field identification procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)				Group symbols 1	Typical names	Information required for describing soils	Laboratory classification criteria
Coarse grained soils More than half of material is larger than No. 200 sieve size 1 (The No. 200 sieve size is about the smallest particle visible to naked eye)	*Gravels* More than half of coarse fraction is larger than No. 4 sieve size (For visual classification, the ¼ in. size may be used as equivalent to the No. 4 sieve size)	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses	$CU = \frac{D_{60}}{D_{10}}$ Greater than 4; $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below 'A' line, or *PI* less than 4 / Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above 'A' line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than No. 4 sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	*Example:* *Silty sand*, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic lines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$CU = \frac{D_{60}}{D_{10}}$ Greater than 6; $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below 'A' line or *PI* less than 4 / Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits shows 'A' line with *PI* greater than 7

Use grain size curve in identifying the fractions as given under field identification

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows:
Less than 5% — *GW, GP, SW, SP*
More than 12% — *GM, GC, SM, SC*
5% to 12% — *Borderline* cases requiring use of dual symbols

Fine grained soils More than half of material is smaller than No. 200 sieve size	Identification procedures on fraction smaller than No. 40 sieve size	Dry strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group symbols 1	Typical names	Information required for describing soils
	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or dictomaceous fine sandy or silty soils, elastic silts	*Example:* *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly organic soils	Readily identified by colour, odour, spongy feel and frequently by fibrous texture				*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

Figure E-5. The Unified Soil Classification Chart, showing the means by which all particulate soil materials may be identified by name and symbol familiar to geotechnical personnel throughout the world.

G. A coarse-grained soil having more than 50 percent of the coarse-grained fraction passing a No. 4 sieve is classified as sand, and it is denoted by the symbol S. Coarse-grained soils are further subdivided either by their gradation (distribution of grain sizes) or by the properties of the fine-grained fraction of the soil. The classifications and criteria for each group are given in the Unified Soil Classification Chart in Figure E-5. Also shown in Figure E-5 is a Plasticity Chart which is used in classification by the USCS.

E.2.1.1 Less than 5 Percent Minus 200 Sieve. Those coarse-grained soils having less than five percent, by weight, passing the No. 200 sieve are subdivided by their gradation and are given the classifications of GW, SW, GP and SP meaning, respectively, Gravel (Well-Graded); Sand (Well-Graded); Gravel (Poorly-Graded); and Sand (Poorly Graded). Well-graded sands have a predominance of several sieve sizes.

GW Group. Well-graded gravels and sandy gravels which contain little or no fines are classified as GW. In these soils, the presence of fines has no effect on strength or free draining characteristics. In addition to the criteria stated previously, this group must have a uniformity coefficient (Cu) of greater than 4, and the coefficient of curvature (Cc) of the soil must be between 1 and 3. (See Figure E-5 for definition of Cu and Cc.)

SW Group. This group of soils is similar to the GW Group except that the predominant grain size is sand rather than gravel. It includes well-graded sands and gravelly sands. The uniformity coefficient of SW soil must be greater than 6, and the coefficient of curvature must be between 1 and 3.

GP Group. Soils which classify as gravels and which will not meet the grading requirements of the GW group are placed in the GP group. These soils include poorly-graded gravels and sandy gravels having little or no fines.

SP Group. Soils which classify as sands and which will not meet the grading requirements of the SW group are placed in the SP group. These soils include uniformly-graded and gap-graded sands and gravelly sands.

E.2.1.2 More than 12 Percent Minus 200 Sieve. Those coarse-grained soils having more than 12 percent, by weight, passing the No. 200 sieve are subdivided by the plasticity characteristics of the fine-grained portion and are given the classifications of GM, GC, SM and SC meaning, respectively; Gravel–With Silt Fines; Gravel–With Clay Fines; Sand–With Silt Fines; and Sand–With Clay Fines. The amount of fines in these groups is enough to affect engineering characteristics.

GM Group. Soils comprising this group are those in which the predominant coarse-grained fraction is gravel and the predominant fine-grained fraction is silt. This group of soils includes silty gravels and mixtures of gravel, sand, and silt. Soils which classify as gravels and have a fine-grained portion for which the Atterberg limits (liquid limit and plasticity index: see section 9) will plot below the A-line in Figure E-5 are placed in the GM group.

GC Group. Soils which classify as gravels and have a fine-grained portion for which the Atterberg limits will plot above the A-line and for which the plasticity index is more than 7, are placed in the GC Group. This group includes clayey gravels and poorly graded gravel-sand-clay mixtures.

SM Group. This group is similar to the GM group except that the predominant coarse-grained fraction is sand. The group includes silty sands.

SC Group. This group is similar to the GC group except that predominant coarse-grained fraction is sand. The group includes clayey sands and sand-clays.

E.2.1.3 Borderline (5 to 12 Percent Fines). Those coarse-grained soils containing between five and twelve percent, by weight, material passing the No. 200 sieve are termed borderline and are given a dual classification such as SW-SM. Also, those coarse-grained soils containing more than 12 percent material passing the No. 200 sieve and for which the Atterberg limits plot in the hatched zone of the Plasticity Chart (Fig. E-5) receive a dual classification such as SM-SC. These double symbols are appropriate to the grading and plasticity characteristics.

E.2.2 Fine-Grained Soils

These soils are subdivided by plasticity and compressibility instead of by grain size. They are classified as silt and clay and as material having either low or high compressibility. Criteria for classification are based upon the relationship between the liquid limit (LL) and the plasticity index (PI) and are given in the Plasticity Chart shown on Figure E-5. On this Chart, for classification, the PI is plotted against the LL.

The A-Line shown on the Plasticity Chart divides clay soils from silts. Soils for which the Atterberg limits plot above this line are clays and are designated by the symbol C; while those which plot below the A-Line are silts and are given the designation M. This Plasticity Chart was also developed by Arthur Casagrande who found that fine-grained soils could be reliably grouped in accordance with their position on such a chart.

Soils (both silt and clay) which have a liquid limit less than 50 percent are judged to have low plasticity and are designated by the symbol L. Those soils having a Liquid Limit (LL) greater than 50 percent are termed highly plastic and are designated by the symbol H. Hence a soil determined to be a highly plastic clay is designated as CH, etc. In general, the more plastic a material is, the lower will be its shear strength and permeability, and the higher its compressibility.

ML Group	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.
CL Group	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays and lean clays.
MH Group	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
CH Group	Inorganic clays of medium to high plasticity, fat clays.

E.2.3 Organic Soils

As pointed out previously, the placement of soils into this group is based upon visual inspection. However, they are subdivided within the group in accordance with their plasticity characteristics. All of these soils should plot below the A-Line on the Plasticity Chart. They are considered to have low plasticity and compressibility (L) if their liquid limit is less than 50 percent, otherwise they are considered to have high plasticity and compressibility (H).

Organic matter tends to decay with time and thus create more voids in the soil mass. Organic matter can promote chemical alterations which change the physical properties of the soil.

OL Group	This group consists of organic soils having a liquid limit of less than 50. Organic silts and organic sandy clays are included in this group.
OH Group	This group consists of organic soils having a liquid limit of more than 50. Organic clay and organic silty clay will usually be included in this group.
PT Group	Peat and other highly organic soils.

E.3 FIELD IDENTIFICATION

Classification by the USCS can be readily done after laboratory testing for gradation and Atterberg limits as indicated on Figure E-5. With practice, classification is possible in the field without the aid of laboratory tests.

A representative sample of the soil is visually examined and is first classified as to whether it is highly organic, fine-grained, or coarse-grained. This classification for fineness and coarseness is made by estimating whether or not one-half of the individual grain can be seen with the naked eye. If 50 percent or more of the particles can be seen, the soil is classified as coarse-grained; otherwise the soil is classified as fine-grained.

E.3.1 Coarse-Grained Soil

If the soil is coarse-grained, it is classified as gravel or sand, depending upon whether or not 50 percent of the coarse grains are larger or smaller than the openings in a No. 4 sieve (4.8 mm; 3/16 in.).

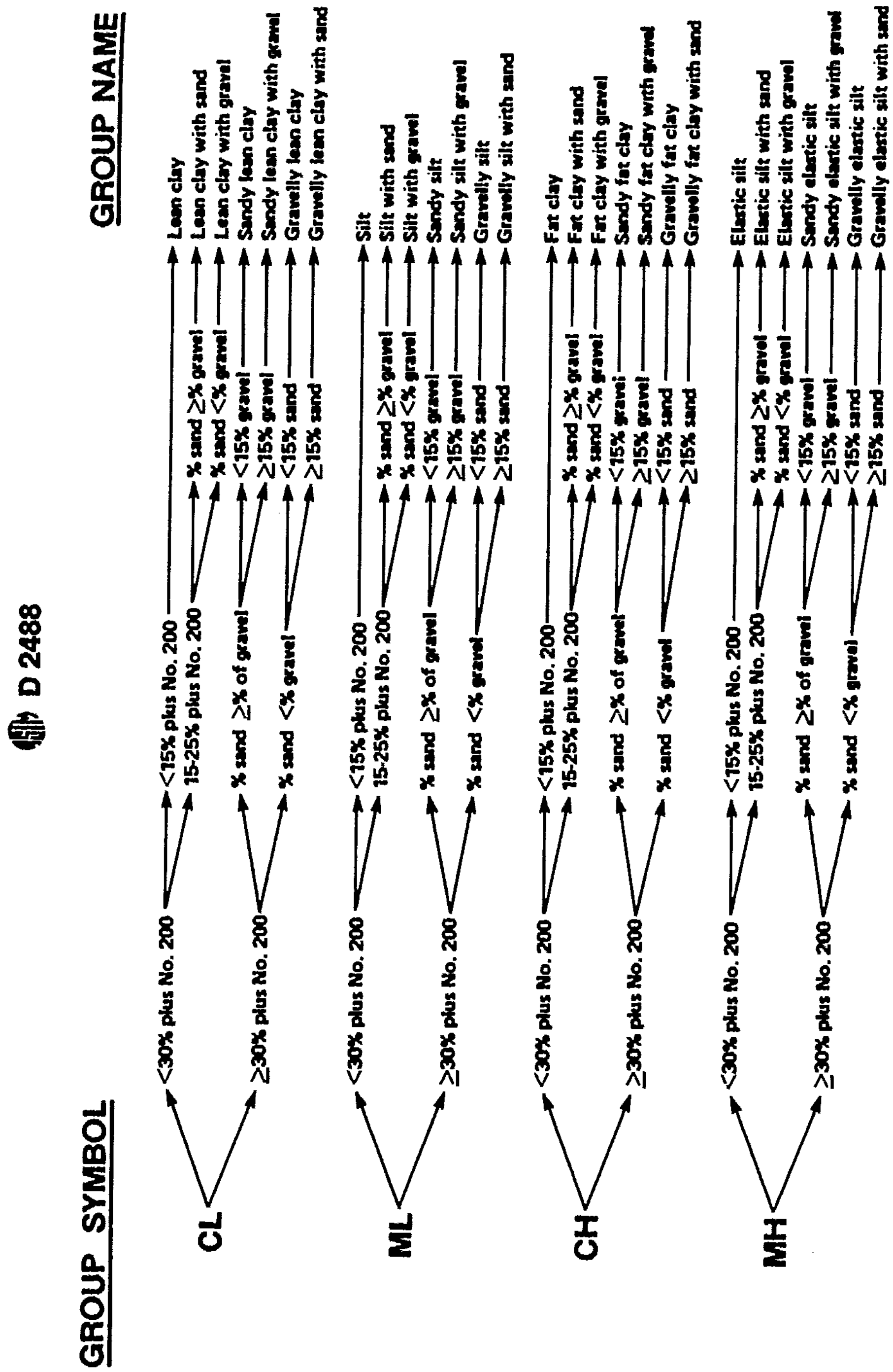
If the soil is classified as gravel, it is then identified as to whether it is *clean* or *dirty. Dirty* means that the gravel contains an appreciable amount of fines, and clean means that it is essentially free of fines. If the gravel is clean, then gradation criteria apply and the material is classified as well-graded (GW) or poorly-graded (GP). The differentiation between clean and dirty is not a formal part of the USCS; rather the distinction is made in passing, as part of the classification process. The formal process calls for determining the percent by weights, finer than the No. 200 mesh sieve. Well-graded soils will have a good distribution of particle sizes from coarse to fine; poorly-graded soils will be either uniform-size or gap-graded.

If the gravel is dirty, the fine-grained portion is determined to be either silt or clay, and the soil is classified as GM (silty gravel) or GC (clayey gravel) respectively. The manual test used in the classification of the fine-grained portion is discussed under Fine-Grained Soils in subsection E.3.2.

If the soil is predominantly sand, the same criterion as that for gravel is used–*clean* or *dirty.* If *clean,* the gradation is examined, and the soil is classified as well-graded (SW) or poorly-graded (SP). If the sand is *dirty,* the fines are evaluated, and the soil is classified as SM (silt fines) or SC (clay fines).

E.3.2 Fine-Grained Soils

If the soil is fine-grained, its field classification will be based primarily upon the estimate of its *dilatancy, dry strength,* and *toughness.* See subsection 4.4 for Field Identification of Fine-Grained Soils or Fractions and Table E-2 for Silt and Clay Characteristics. Silt fractions will have nil to medium dry strength, quick to no reaction to shaking, and nil to medium thickness. On the other hand, clay fractions will have medium to



NOTE—Percentages are based on estimating amounts of fines, sand, and gravel to the nearest 5 %.

Table E-2. Flow Chart for Identifying Inorganic Fine-Grained Soil (50% or more fines)

very high dry strength, no reaction to shaking, and medium to high toughness. Dispersion, crumbling and taste may also help to identify the silt or clay fractions as indicated on Table E-2. Classification will be ML, MH, CL or CH.

E.3.3 Highly Organic Soils

These soils are readily identified by color, odor, and spongy feel and frequently by a fibrous texture. Organic matter is often indicated by the presence of olive green, and light brown to black colors. Organic soils usually emit a distinctive odor of decaying vegetation. The odor is strong for fresh samples and can be intensified by heating a sample quickly. Dilantancy, dry strength and toughness are also an aid in identification.

E.3.4 Borderline Classifications

With experience, soils which fall well within one group can be readily classified. However, soils which are near boundary requirements are more difficult to classify and may require a dual classification such as GC-SC or CL-CH.

E.4 MANUAL TEST FOR FIELD IDENTIFICATION OF FINE-GRAINED SOILS OR FRACTIONS

These tests are to be performed on the minus No. 40 sieve size particles, approximately 0.4 mm (1/64 in.) in the manner described below. For field classification purposes, screening is not required; coarse particles which interfere with the tests may be removed by hand.

E.4.1 Dilatancy (Reaction to Shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about 8191 mm^3 (0.5 $inch^3$). If necessary, add enough water to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake vigorously against the other hand several times. A positive reaction is indicated by the appearance of water on the surface of the pat, which changes to a liver-like consistency and becomes glossy. When the sample is squeezed by slightly closing the palm of the hand, the water and gloss disappear from the surface, the pat stiffens, and finally cracks or crumbles. The rapidity of appearance of water during shaking and disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine, clean sands give the quickest and most distinct reaction, whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

E.4.2 Dry Strength (Crushing Characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air-drying, and then test its strength by breaking and crumbling it between the fingers. This strength is a measure of the character and quality of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry-strength is characteristic for clays of the CH group (Inorganic clays of high plasticity). A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty, whereas a typical silt has the smooth feel of flour.

E.4.3 Toughness (Consistency near Plastic Limit)

After particles larger than the No. 40 sieve size are removed, mold a specimen of soil about 12.7 mm (0.5 in.) into a cube, to the consistency of putty. If too dry, water should be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Roll the specimen out by hand on a smooth surface or between the palms into a thread about (one-eighth in.) 3.2 mm in diameter. Fold and roll the thread repeatedly until a 1/8 in. diameter thread shows signs of crumbling; this is the plastic limit. During this manipulation, the moisture content is gradually reduced, and the specimen stiffens, finally loses its plasticity and crumbles when the plastic limit is reached.

After the thread crumbles, lump the pieces together and continue kneading until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-group clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

E.5 DESCRIPTIVE TERMINOLOGY

The Unified Soil Classification System provides a conventional system for categorizing soils by gradation and plasticity characteristics. However, it does not provide guidelines for adequate descriptive terminology for identifying soils. For example, it gives no guidelines for determining color, density or consistency, and other pertinent properties of the soil that may be useful in describing a soil layer and correlating engineering properties. Likewise, no criteria are given in the system for determining if sands or gravels are coarse, fine or medium in grain size. This distinction is frequently important; for example, in determining liquefaction susceptibility and other engineering properties. In addition, no guidelines are presented for the use of adjectives in describing major soil constituents in the USCS. A sandy gravel may mean a gravel with different percentage of sand to different people. Furthermore, no procedures are given for writing a descriptive classification for soils in the USCS; such a descriptive terminology could incorporate all of the above shortcomings. It may be expedient to describe a soil by the use of a symbol (such as GW, ML, etc.), but a written, descriptive terminology, followed by a group symbol presents a much more complete picture of the nature, composition and properties of a given soil.

Suggested procedures and guidelines for preparing a description of a soil deposit or sample are presented below. This descriptive terminology is not intended to replace the USCS, but to expand it, in order to make it more precise, better understood and more universally applied and accepted. In all cases, the descriptive terminology is to be followed by the USCS symbol in parenthesis.

As a minimum, the descriptive terminology for a soil should include the following, in the order given:

- Density or consistency
- Color
- Major and secondary soil constituents (major constituents to be determined by gradation and plasticity as per the USCS
- USCS Symbol
- Other pertinent terms

Two examples of such a description would be (1) medium compact, brown silty SAND, (SM) (slightly plastic); or (2) stiff, yellow CLAY, (CH), (high dry-strength).

E.5.1 Density and Consistency

The density of coarse-grained (granular) soils and the consistency of fine-grained (cohesive) soils is determined by the standard penetration test performed in test borings. The number of blows of a 63.6 kg (140-pound) hammer falling 76 cm (30 in.) required to drive a 51 mm (2-in.) O.D. split-spoon sampler into virgin soil is recorded for each 15 cm (6 in.) of penetration for a total penetration of at least 45 cm (18 in.). The blow count for the first 15 cm (6 in.) of penetration is ignored and the succeeding two 15 cm (6 in.) blow-counts are added to obtain the blows per foot of penetration.

The density or consistency of a soil based on the standard penetration test is shown in Table 6-5.

E.5.2. Soil Color

Soil color description is generally confined to a few basic terms such as brown, black, gray and white. These terms are often combined in pairs to give brown-gray and gray-brown. Rust-brown and red-brown are also useful descriptive terms. The color is descriptive of the fresh sample as it comes out of the ground; the sample color may change with time. More accurate color descriptions based on hue value and chroma may be obtained by use of Munsell Soil Color Charts (Kollmorgen Corp., 1973), however, such refinement is usually not required.

E.5.3 Primary and Secondary Soil Constituents

The primary soil constituent is to be determined on the basis of the grain size and plasticity characteristics in accordance with the USCS, Figure E-5. Coarse-grained soils should be further delineated on the basis of grain size as follows:

Soil Components	*Size Range*
Boulders	Above 20.3 cm (8 in.)
Cobbles	7.6 to 20.3 cm (3 to 8 in.)
Gravel:	
Coarse gravel	7.6 to 1.9 cm (3 to 0.75 in.)
Fine gravel	0.75 in. to No. 4 screen* (4.76 mm)
Sand:	
Coarse sand	No. 4 (4.76 mm) to No. 10 screen (2.0 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 screen (.074 mm)

*(Numbers such as 4, 10 etc., refer to U.S. Bureau of Standards standard sieve sizes.)

For rapid and easy identification, the primary constituent should be indicated by upper case letters, e.g., GRAVEL, SAND, SILT or CLAY. Subcategories of the coarse-grained soils should be written as "coarse to fine SAND" or "fine GRAVEL".

Secondary soil constituents should be determined also on the basis of gradation and plasticity as per the USCS. However, to provide consistency in descriptive terminology, the following format may be used for secondary components, based on percentages passing standard screens:

- The second most predominant constituent, if present in an amount between 20 and 50 percent of the total sample is indicated by an adjective modifying the major constituent, e.g.: a sample consisting of 70 percent gravel and 30 percent sand would be described as a sandy GRAVEL.
- If a third component comprises more than 20 percent of the total sample, it is used following the primary constituent and prefixed by the word "some", e.g. a sample consisting of 55 percent sand, 25 percent gravel and 20 percent silt would be described as gravelly SAND, some silt.
- Constituents which comprise between 10 percent and 20 percent of the total sample are indicated by prefixing the word "little" before their name and adding the constituent after the major constituent, e.g., a sample consisting of 55 percent gravel, 30 percent sand and 15 percent silt would be described as a sandy GRAVEL, little silt. For a sample comprised of 85 percent sand and 15 percent silt, the description would be SAND, little silt.
- Any material which is present in amounts between 5 percent and 10 percent is indicated by the word "trace" and the descriptive term is the final item of the grain size description, e.g.: a sample comprised of 50 percent gravel, 30 percent sand, 12 percent silt and 8 percent cobbles would be described as a sandy GRAVEL, little silt, trace cobbles. It should be stressed however, that a description involving four constituents is the exception rather than the rule. Most soil descriptions using this system would consist of a maximum of three soil constituents.
- The use of qualifiers (i.e., some, little, trace) is not uniform or standard among agencies and others in geotechnical practice. Use of qualifiers for minor constituents (third components) of soils is useful in field descriptions, however. For critical design use or for inclusion in specifications, a program of laboratory verification should be employed.

E.5.4 USCS Symbols

The Unified Soil Classification System Symbol as determined from Figure E-5 and Section E.3., should be added in parentheses at the end of the soil description.

E.5.5 Other Pertinent Properties

Descriptive terminology may include some or all of the following items. These items should be added at the end of the description.

- The shape of gravel and coarse sand grains, i.e., rounded, subrounded, subangular, or angular.
- Degree of plasticity of the fine grained fraction of granular soils. A plasticity designation is not required for fine-grained soils, as their identification is based on the plasticity chart of the USCS. It may be useful, however, when describing inorganic silts.
- The geologic origin of a soil may simplify its description, especially in regional areas where such terms are used and readily accepted. Examples of such terms are *glacial till, saprolite, loess, caliche, varved clay,* and *fill.*
- Other concise descriptive comments about appearance or engineering properties which add information about the soil should also be used.

Examples of soil description based on the principals set forth in preceding paragraphs are given below. Note not only the order of the descriptive terms, but the use of commas, hyphens, parentheses and upper case letters.

- Compact, gray, silty, coarse to fine SAND, little fine gravel, trace clay, with few cobbles and small boulders (SW-SM) (very dense, well-bonded *in situ*). -GLACIAL TILL-
- Very soft, dark gray, clayey SILT, trace fine sand partings (MH)
- Soft, dark brown, medium to fine, sandy ORGA SILT, trace root fibers (OL)
- Loose to medium compact, mottled, gray to brown, gravelly, coarse to fine SAND, trace silt, brick and ash. -FILL-
- Very loose, rust-brown, fine, sandy SILT (ML) -LOESS-
- Loose, light brown, silty, fine SAND (SM) (medium plastic)

E.6 CLASSIFICATION OF ROCK

Classification of rock is an essential part of the geotechnical information developed to support design and construction of any transportation project which will be built wholly or partially in rock. In addition to the definition of each separate geologic unit in field mapping, each rock type must be traced in surface outcrop and estimated in occurrence where geologic contacts are not directly observable. The classifications apply to both surface excavations in rock and to underground construction such as stations and tunnels. Rock classification for engineering purposes consists of two basic assessments; that for intact character, such as a hand specimen or small fragment; and *in situ* character, or engineering features of rock masses:

- Intact character: classification of the intact rock, such as hand specimens or core, as to its origin, geologic formational identity, mineralogical makeup, character of internal fabric, and degree and nature of chemical and physical weathering or alteration.
- *In situ* character: classification of the rock, in-place as rock masses includes the nature and degree of its constituent interlocking blocks, plates, or wedges formed by bounding discontinuities such as foliation planes, joints, shear planes, shear zones and faults.

Both assessments are usually presented in geotechnical reports; design or contractor personnel make use of the separate components of the classification as appropriate. Intact classifications are essential for design. Bidding contractors must evaluate the general nature of the rock, mainly in terms of general technical knowledge and according to individual experience in each type of rock. Intact classifications are the basis for rock excavation program design and many facets of rock anchorage and bearing capacity determinations. *In situ* classification data are then applied, where applicable, to the evaluation of the behavior of whole masses of lithologically similar rock, such as in rock cuts and underground structures taken as a whole.

An important facet of rock classification is the determination of what constitutes *rock,* as opposed to extremely weathered or altered material which approaches *soil* in its character and engineering characteristics. An appropriate manner of viewing rock classification over the entire spectrum of very hard to very soft rock is to consider rock itself to be the primary earth material present in a construction site and to classify all rock according to accepted geotechnical practice, examples of which are presented herein. In the course of this classification, soil-like rocks will be distinguished in terms of degree of weathering and field hardness. Geologists and geotechnical engineers will then determine what rock is represented by engineering properties that are more like those of soil than rock. A statement should then be made as to the presence of such soil-like rock, an appropriate name given for each of these units and the remainder of the classification should be developed according to accepted methods of soil classification. The contact between soil-like rock and rock is generally called top-of-rock and is noted on boring logs and in interpreted profiles and cross sections.

E.6.1 Visual-Manual Descriptions

For small projects or during the early stages of a large project, rocks are initially classified by visual means. Given below are guidelines for the visual-manual classification of rock.

- *Color.* When describing color, use only common colors such as gray, brown, green, etc., or simple combinations of these such as yellow-brown. Also degree of color such as light vs. dark should be employed. For special purposes, the Munsell Soil Color charts may be specified; giving hue, value, and chroma numbers as the basis of the description. Munsell colors are quite useful in working with severely weathered rock.
- *Texture.* Terminology used to identify size, shape and arrangement of the constituent elements: e.g., porphyritic, glassy, amygdaloidal, etc. Where applicable, the following size classification is utilized:

Aphanitic	Constituent mineral grains too small to be seen with naked eye.
Fine Grained	Constituent mineral grains barely detectable with naked eye.
Medium Grained	Mineral grains barely detectable with naked eye; to 2.5 mm (0.1 in.)
Coarse Grained	Mineral grains between 2.5 mm (0.1 in.) and 6 mm (0.25 in.)
Very Coarse Grained	Particles greater than 6 mm (0.25 in.)

- *Lithology.* Rocks are classically divided into three general categories; igneous, sedimentary and metamorphic. The most conspicuous feature of most igneous rocks is texture which forms one of the bases on which igneous rocks are classified, in addition to mineralogy and genetic occurrence. Sedimentary rocks are classified on the basis of grain size and on the relationship between grains. The most conspicuous features of metamorphic rocks are generally their structural features, especially foliation. The complete name of a rock should include color, texture, alteration if any, accessory minerals, and lithologic name. In most cases the classification of a rock in the field should be checked in the laboratory with a petrographic analysis.

- *Field Hardness.* Field hardness is determined by striking or scratching the rock outcrop or rock core. Field hardness is a qualitative assessment of the general integrity of intact rock, that is, the hardness of individual mineral grains and the relative strength by which the grains are bonded together. For projects involving machine rock excavation such as tunnel boring machines (TBM), field hardness is a secondary measures of rock integrity, after laboratory hardness measurements. Field hardness assessments are usually included in outcrop station notes in the geological field book and are also made a part of boring log descriptions:

Very Hard	Cannot be scratched by knife or sharp pick. Breaking of hand specimens requires several hard blows of the geologists pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard hammer blows required to detach hand specimen.
Moderately Hard	Can be scratched by knife or pick. Gouges or grooves to 6 mm (0.25 in.) deep can be excavated by hand blow of point of a geologists pick. Hand specimens can be detached by moderate blows.
Medium	Can be grooved or gouged 2 mm (0.05 in.) deep by firm pressure of knife or pick point. Can be excavated in small chips to pieces about 25 mm (1 in.) maximum size by hard blows of the point of a geologists pick.
Soft	Can be gouged or grooved readily by knife or pick. Can be excavated in fragments from chips to several inches in size by moderate blows of a pick point. Small, thin pieces can be broken by finger pressure.
Very Soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces one inch or more in thickness can be broken by finger pressure. Can be scratched readily by fingernail.

Many workers use the Schmidt Hammer Test in the field as a measure of rock hardness. The Schmidt Hardness Test should be considered a laboratory test procedure and when used in the field, the Schmidt Hammer should incorporate all of the specified laboratory test conditions.

- *Weathering.* Weathering and chemical alteration are important aspects of rock classification that can affect both intact and *in situ* rock properties. In the earliest stages, weathering is manifested by discoloration of intact rock and only slight changes in rock texture. With time, significant changes in rock hardness, strength, compressibility and permeability occur and the rock mass is altered until the rock is reduced to soil. Alteration may occur as zones and pockets and can be found at depths far below that of normal rock weathering. Weathering and alteration can be classified as part of the verbal rock core description.

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very Slight	Rock generally fresh, joints stained, some joints may show thin clay coatings if open, crystals on a broken specimen face shine brightly. Rock rings under hammer blows if of a crystalline nature.
Slight	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in.) Open joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer blows.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored, some show clay. Rock has dull sound under hammer blows and shows significant loss of strength as compared with fresh rock.

Moderately Severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and a majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock gives "clunk" sound when struck.
Severe	All rocks except quartz discolored or stained. Rock "fabric" clear and evident but reduced in strength to strong soil. In granitoid rocks all feldspars are kaolinized to some extent. Some fragments of strong rock usually remain.
Very Severe	All rock except quartz discolored or stained. Rock fabric elements are discernible but the mass is effectively reduced to *soil* status, with only fragments of strong rock remaining. Saprolite is an example of rock weathered to a degree such that only minor vestiges of the original rock fabric remain.
Complete	Rock reduced to *soil*. Rock fabric not discernible, or discernible only in small and scattered concentrations. Quartz may be present as dikes or stringers. Saprolite is also an example.

- *Voids.* Open spaces in the subsurface are generally due to removal of rock materials by chemical dissolution or the action of running water. Since most of these voids result from the action of groundwater, the openings are usually elongate in the horizontal plane. As in weathering classification, voids can be related either to intact properties or to *in situ* rock properties, depending on their size.

Pit	Voids barely seen with the naked eye, to 6 mm (0.25 in.)
Vug	Voids 6 to 50 mm (0.25 to 2 in.) in diameter
Cavity	50 to 600 mm (2 to 24 in.) in diameter
Cave	Voids 50 to 600 mm (24 in.) and larger in diameter

- *Miscellaneous Features.* Include any additional characteristics to further identify and evaluate the rock from the standpoint of intact properties such as secondary mineralization, fossils, and swelling and slaking properties.

E.6.2 Classification of *In Situ* Rock

Structural elements of the rock mass should be assessed in an attempt to define the overall engineering characteristics of the mass. Discontinuities are the major elements of *in situ* classification. These fractures should be described in terms of frequency, spacing, roughness, bonding quality and general continuity. The various structural features should be described when encountered as follows:

E.6.2.1 Geologic Discontinuities. Geologic discontinuities which separate the rock mass into discrete units.

Types of Discontinuities

Joint	A simple fracture along which no shear displacement has occurred. May occur with parallel joints to form part of a joint set.
Shear Plane	A fracture along which differential movement has taken place parallel to the surface sufficient to produce slickensides, striations or polishing. May be accompanied by a zone of fractured rock up to a few inches wide.
Fault	A major fracture along which there has been appreciable displacement and accompanied by gouge and/or a severly fractured adjacent zone of rock.
Shear Zone or Fault Zone	A band or zone of parallel, closely spaced planar breaks and associated broken (brecciated) rock and gouge

- *Attitude.* Attitude refers to the orientation of a discontinuity in space in terms of strike and dip. Strike can not be obtained from rock core without special techniques such as oriented core or borehole photography. Correlation of test boring results with nearby rock outcrops can be very useful for estimating strike. A quantitative expression for dip is given below:

Dip	*Angle*
Horizontal	0°– 5°
Shallow or low angle	5°–35°

Moderately dipping	35°–55°
Steep or high angle	55°–85°
Vertical	85°–90°

- *Spacing.* The spacing refers to the perpendicular distance between adjacent discontinuities and should be described as follows:

Fractures	*Spacing*	*Foliation or Bedding*
Very close	Less than 5 cm (2 in.)	V. Thin
Close	5–30 cm (2–12 in.)	Thin
Moderately close	30–100 cm (1 to 3 ft.)	Medium
Wide	1–3 m (3–10 ft.)	Thick
Very wide	More than 3 m (10 ft.)	V. Thick

- *Tightness.* The degree of closure of the opposing faces of the discontinuity. The following terminology should be employed when describing tightness: Tight, Open, Healed.
- *Planarity.* Relative smoothness of the surface of the discontinuity, for example: Smooth, Wavy, Irregular.
- *Regularity.* The surface of the discontinuity may be plane, curved or irregular on a large scale and/or slick, smooth or rough on a small scale.
- *Continuity.* Continuity is an expression of the lateral extension of the discontinuity, as measured or projected along strike and dip:

Discontinuous	0–1.5 m (0–5 ft.)
Slightly continuous	1.5–3 m (5–10 ft.)
Continuous	3–13 m (10–40 ft.)
Highly continuous	More than 13 m (40 ft.)

  Continuity is a very important property of the rock mass, as a single continuous joint may actually control the behavior of the entire mass. It is essential to realize that continuity cannot be determined with test borings along; some type of large diameter exploration, field mapping or a well-coordinated boring program is necessary in order to determine continuity.
- *Filling.* This refers to the nature of the material, if any, in the space between adjacent surfaces of the discontinuity. The filling material may consist of weathered or hydrothermally altered products, secondary mineral precipitates, mylonite or gouge. The mineralogy, thickness and hardness of fill material should be described.

E.6.2.2 Rock Quality Designation (RQD). Rock Quality Designation (RQD) is an evaluation of the frequency of occurrence of discontinuities in a rock mass. In general, RQD is defined as the total length of core segments equal to or greater than 10 cm (4 in.) in length recovered from a borehole divided by the total length of core run. This value is expressed as a percent. Rock Quality Designation is determined as described in a qualitative description of rock quality given below:

Rock Mass Description	*RQD*
Excellent	90–100
Good	75–90
Fair	50–75
Poor	25–50
Very Poor	Less than 15

RQD is sometimes correlated with Fracture Spacing.

Drilling Fractures. Only natural fractures such as joints or shear planes should be considered when calculating the RQD.

Fractures due to drilling and handling of the rock core must be discounted.

Core Barrel Size and Type. RQD is most frequently calculated for NQ size core or larger. The core is typically obtained with double-tube core barrels. Use of smaller diameter cores and single-tube core barrels can severely penalize rock core quality as a measure of *in situ* rock mass quality and should not be utilized for RQD determinations.

Weathering. Rock assigned a weathering classification of moderately severe, severe or very severe should not be included in the determination of RQD, regardless of length.

Core Recovery. RQD measurements assume that core recovery is at or near 100 percent. As core recovery varies from 100 percent, explanatory notes may be required to describe the reason for the variation, and the effect on RQD. In some cases, RQD will have to be determined on the basis of the total length of rock core recovered, rather than on the length of rock cored.

E.6.2.3 Weathering Profile. Detailed descriptions of various weathered rock conditions were given earlier in this section. Of much greater importance is description of the weathering profile of the rock mass. The weathering profile should be carefully described regardless of core run lengths or other variables. Degree of weathering should be carefully noted on the test boring log.

E.6.2.4 Miscellaneous Features. Additional characteristics to further identify and evaluate the rock from the standpoint of *in situ* properties such as large voids, zones of very high permeability combustible gas content, groundwater quality, and *in situ* stress conditions.

E.6.2.5 Sample Rock Description. Given below are typical geological and engineering descriptions for rock based on the above intact (rock core) and *in situ* (rock core) classification methods. The visual geological description was verified by petrographic analysis. The project involved construction of a tunnel.

Geological Description. Dark gray, fossiliferous Mudstone. Upper 7.5 m (25 ft.) referred to as Gates Dolomite; dark gray, fine to medium-grained, slightly fossiliferous dolomite. Underlying material is dark gray calcareous shale with numerous dolomite and limestone partings, occasional gypsum filled seams and vugs, and abundant fossils. Lowest 3–4.5 m (10–15 ft.) is more shaley and subject to disintegration upon exposure.

Engineering Description and Classification. Dark gray, soft, medium to high strength, highly durable Mudstone. Tangent modulus measurements (E_{t50}) suggest an intact rock with relatively high compressibility. A hypothetical, average, *in situ* modulus of elasticity for this formation based on measured intact rock moduli and RQD is approximately 3×10^{10} N/m^2 (4×10^6psi). Average *in situ* permeability is estimated as equivalent to that of a fine sand and characterized as medium. Predominant geologic discontinuities are bedding planes and joints with little shearing.

E.6.3 Field Testing of Rock

Field testing of rock is usually very expensive and is generally used only on very large projects such as dams, underground powerhouse galleries, and some larger tunnels. Three methods of field testing observations (as mentioned above) are: Rock Quality Designation, Oriented Rock Coring and Water Pressure Testing. Commonly used methods of field testing rock are discussed in Sections 6 and 7 and Appendix B.

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APPENDIX F
Rock Excavation Programs

The construction of major civil engineering projects frequently involves excavation of rock in order to establish grade, to produce roadway cuts, or to create underground space for stations or tunnels. Just as the integrity of rock varies widely, from loose and soft sedimentary rock and weathered or altered rock of all types, to massive and essentially fracture-free crystalline varieties, the effort required to excavate it varies accordingly. A significant percentage of project funding can be expended by rock excavation. The method to be employed in excavation remains one of the most variable components of most contracts for construction in rock. Roadway designers and structural engineers are generally concerned with the effect of this factor on the range of contract bids and on the quality of construction and contract performance of the bidders.

Agency personnel involved in subsurface exploration can provide essential information to design engineers early in the project so that design considers the effect of rock excavation requirements. Carefully planned preliminary and design-level geotechnical and geological studies should be conducted to provide these data. Certain other raw and interpreted filed data will be provided in bid packages for use by prudent contractors in formulation of their proposed construction method and of their bids in general.

In a 1972 review of the role of Engineering Geology in rock excavation, Leonard Obert clearly stated that the effects of geologic conditions constitute the factors of greatest impact on individual rock excavation programs. Blue-ribbon review panels such as the National Academy of Sciences (1968) and the European Organization for Economic Cooperation and Development (OCED; 1970) stress high-priority for achieving technological advances in the development of improved method of rock fragmentation, prediction of geologic conditions ahead of the excavated face, and improved techniques of handling blasted rock. The greatest present impact on rock excavation programs is that of careful observation and analysis by engineering geologists and geotechnical engineers assigned to individual project teams.

F.1 THE NATURE OF ROCK EXCAVATION

Rock excavation is characterized mainly by the application of force of a mechanical or explosive nature to the task of breaking rock (Figure F-1). Naturally, the contractor wishes to employ the excavation method which expends the least force necessary to create a rock mass that is manageable through the means which he has planned for handling, transport and disposal/placement. The reader can appreciate that these requirements are at once complicated because most projects require that the spoil or muck may be further utilized as a fill material.

When muck or spoil is not utilized directly in construction it must meet the requirements of an environmentally-acceptable placement or must be suitable in size gradation to be acceptable to a third party who wishes to use it for another purpose. The form in which rock excavation waste is produced governs its acceptability for cost-effective disposal.

The main factors relating to the nature of rock excavation waste are concerned with the force applied to the rock to break it and the intrinsic nature of the rock itself; that is:

- Rock strength
- Rock discontinuities
- Nature of explosive or excavation force
- Placement, orientation and timing of the explosive

If the method of excavation or blasting is incompatible with the geologic character of the rock mass, the waste produced will either vary from the desired char-

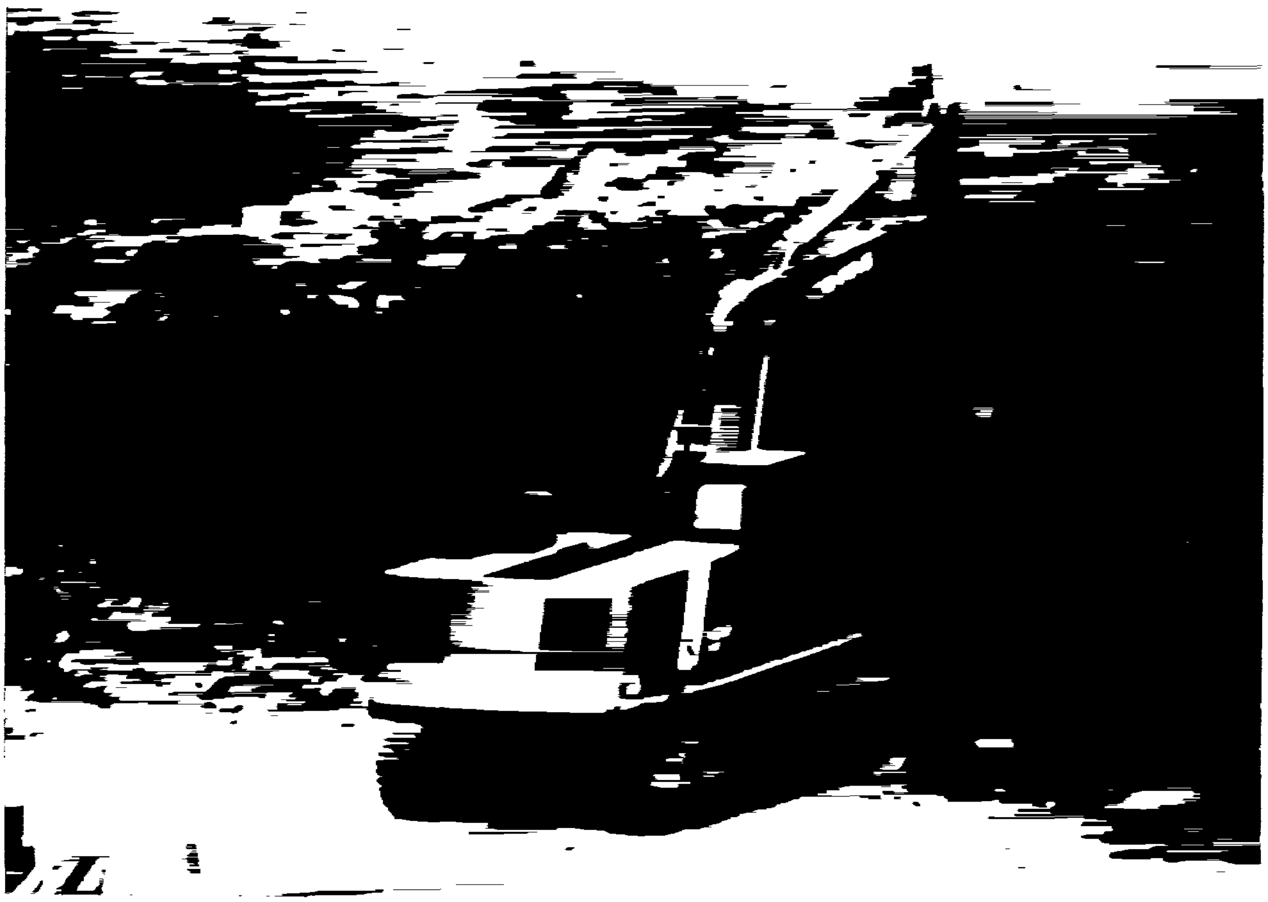

Figure F-1. Routine machine excavation of weak rock results in production of high-tolerance cutfaces. (A.W. Hatheway)

acteristics or will require expenditure of more than optimal energy to handle or treat the waste. Usually continued breaking or crushing are necessary to achieve fragments of the desired size. Additional handling is necessary to move, distribute, haul or place the resulting waste. Of the four main factors listed above, the two dealing with the type and placement of force are dependent upon geologic conditions.

F.2 GOALS OF ROCK EXCAVATION PROGRAMS

Relatively few transportation agencies actually undertake rock excavation programs using their own personnel and equipment. Those agencies that conduct rock excavation on a force account basis should develop a complete program for planning and executing such projects from the preliminary geological exploration through the optimization of rock excavation techniques. For those agencies that develop plans and specifications for contract rock excavation, it is usually essential to provide the bidding contractors with sufficient geologic information to provide for sensible bids. The manner in which the rock excavation program is developed will vary considerably with the general design philosophy of the agency and with its method of developing contract documents. Some of these key data and interpretations for a rock exploration program are shown in Table F–1.

In summary, the rock excavation program should include any activities which are felt will be useful to the bidding contractors in developing costs to remove rock of a quality and size appropriate to other designated uses on the project and to leave the excavated area in the desired condition. Good rock excavation programs should yield the following results:

- A relatively narrow spread of bid components dealing with rock excavation unit costs;
- Contractor adherence to contract schedules;
- Reduced incidences and bases for changed-claim conditions; hence lower final construction costs.

Table F-1
Geologic Data Requirements for Rock Excavation Programs

Rock Excavation Data	Rock Excavation Interpretations
Geologic maps of the areas slated for excavation; the detail shown should be commensurate with the outcrop exposure and need to collect strikes and dips and to characterize the discontinuities. The geologic maps should portray both the areal extent of all lithologic units and representative structural geologic symbols (Section 4).	Definition of basic lithologic types to be expected on the project. Presence and expected orientation of dikes, sills and other intrusions of variable hardness from surrounding rock; and veins of essentially hard minerals such as quartz.
Seismic refraction traverses along and across the area to be excavated.	Geologic sections or profiles showing the expected, generalized distribution of lithologic types with depth. Profile representations of top-of-rock, along with a definition of the nature of that surface. Estimation of volumes of rock to be excavated
A tabulation of averaged compressional wave velocities; from seismic refraction traverses	Generalized remarks concerning the applicability of various rock excavation techniques.
A representation of the nature and frequency (spacing) of occurrences of the various types of discontinuities; may include RQD (Appendix E) of applicable core logs.	Expected zones, pockets, or lenses of alteration or weathered rock; lenses of variable rock hardness (sedimentary strata).
Such engineering property data as are necessary and which have been developed through laboratory testing of representative samples (Section 9)	A definition of the nature and expected extent of rock that is considered unsuitable for the intended construction use; such rock will be considered as waste.

F.3 TYPES OF ROCK EXCAVATION

Rock is excavated by machine or by detonation of explosives in machine-drilled blast holes. When rock is relatively soft, such as sedimentary units or weathered or altered crystalline units, machine excavation usually proves to be the least expensive and most time effective.

Contractors generally favor the employment of machine excavation because of the relatively simple nature of the one-step removal and disposal/placement of the waste. Most machine-excavation programs can be planned to keep the spoil moving in a continuous chain from removal to replacement, hence holding costs at a minimum for the operation. The contractor must review the bid documents to make the fundamental decision concerning the relative volumes of rock that may be excavated or blasted and the expected characteristics of waste. The data and interpretations that are included in the Agency's bid documents usually form the basis for such a decision. Good bidding practice should be based on the mix of information contained in contract documents, the contractor's general experience in rock excavation, his specialized experience in the geographic area or the particular rock type, and the advice from his technical staff or consultant. For these reasons, the contract documents should be carefully planned, executed, and reported.

F.4 CHOICE OF EXCAVATION METHOD

Basic decisions relating to the method of excavation are the key to compilation of a sound, construction bid. For the purposes of this decision, rock is viewed basically as either "hard" or "soft"; soft includes most sedimentary rock and a wide range of weathered and altered igneous and metamorphic rock. The underlying rationale relates directly to the volume of rock that is specified for removal and the selection of the method that is most effective for production purposes.

As a basis for decision making, the bidding contractor will need geologic data relating to the expected areal extent and depths of each definite type of rock (such as by lithology and degree of weathering (Appendix E), profiles, cross sections, ground water levels, seismic velocity data, core logs, and photographs showing the character of rock recovered during exploratory drilling.

If final finishing is required for excavated surfaces or faces in rock, such efforts may represent non-production rock excavation and should be provided for in a separate unit price payment item.

F.5 RIPPABILITY OF ROCK

Rock that is otherwise not removable by blade or scraper pan in open cuts is often loosened or broken by ripping (Figure F-2). Most ripping is undertaken

Figure F-2. Maximum resistance to machine excavation occurs in highly competent sedimentary rock such as this sandstone with siliceous cement; a borderline case nearly requiring blasting. (A.W. Hatheway)

by dozers equipped with ripping teeth; single or double-tooth appendages located to the rear of the tractor and capable of being raised or lowered under power to gouge into the soft, weathered, or fractured rock. The ability to rip is limited by the ability of the tractor to force the ripper teeth into the rock and by the tractive energy of the tractor to lift fracture-bounded blocks of rock or to shear forward through rock blocks. Ripping is generally not considered too expensive to be used routinely as a production-oriented rock excavation method. Since rippable rock lies midway between machine-excavatable softer rock and the blasting required for sound and massive rock, ripping programs should be studied carefully before commitment on a large scale.

The action of ripping of rock consists of machine applied compressive or tensile force against discrete blocks of rock bounded by discontinuities of some sort (bedding planes, joints, shear planes, planes of schistosity, faults, and microfractures). The machine force is applied by a dozer cutting blade, a back-mounted ripping tooth (or teeth) or the cutting edge of a pan scraper. If the joint frequency (speaking of all discontinuities) is less than perhaps 15–30 cm (Figure F–3), the blade or tooth can usually be forced into a fracture within a few feet of forced dragging. If the point or edge pressure applied by the dozer or scraper exceeds the compressive strength of the rock, the changes are good that joint-bounded blocks of rock will be dislodged or the rock itself will then be crushed and the tooth or edge will again gain entry into the rock mass to begin the action of pushing the material up and out into the excavation.

Various workers and equipment manufacturers have developed charts (Figure F-4) relating seismic wave velocity (compressive) to lithologic rock type, as a guide to rippability. The velocity values represent a number of interdependent rock properties (unit weight and mineral hardness) and characteristics (nature and frequency of discontinuities; thickness of open bedding layers); the more dense, harder, and unjointed is the rock, the higher the compressive wave velocity. At the same time, if seismic velocity is used as an indicator of rippability, it must be matched

Joint spacing description	*Spacing of joints*	*Rock mass grading*	*Excavation characteristics*
	mm		
Very close	> 50	Crushed / shattered	Easy ripping
Close	50 – 300	Fractured	Hard ripping
Moderately close	300 – 1 000	Blocky/seamy	Very hard ripping
Wide	1 000 – 3 000	Massive	Extremely hard ripping and blasting
Very wide	> 3 000	Solid/sound	Blasting

Figure F-3. Generalized relationships between joint spacing and ripping characteristics, and grading types of rock waste produced by ripping (From Weaver, 1975).

against the size of a particular piece of equipment. Equipment matches consider, of course, that the dozer is in good repair and is driven by a competent operator.

Many agencies consider it important to provide seismic velocity data as part of the bid package for contractors to use in their interpretation of what type of excavation method will be most efficient for the project. The California DOT has studied the aspects of seismic velocity as a guide to rippability, since the 1960s. In a 1977 report, Elgar Stephens, of the Transportation Laboratory of CALDOT, found that two of the most modern dozers, (Caterpillar D9G and Fiat-Allis HD41), could generally rip rock with seismic velocities in the maximum threshold range of about 1600–3350 mps (5300–11,000 fps) for granitic rock of joint spacings in the range of 0.9 to 4.6 m (3 to 15 ft) for lower-velocity rock to less than 0.15 m (0.5 ft) for high-velocity rock for the Fiat-Allis machine and 150–300 mps less for the Caterpillar model. This is in good agreement with the scheme of rippability assessment of Weaver (1975; Figure F-5), discussed later in

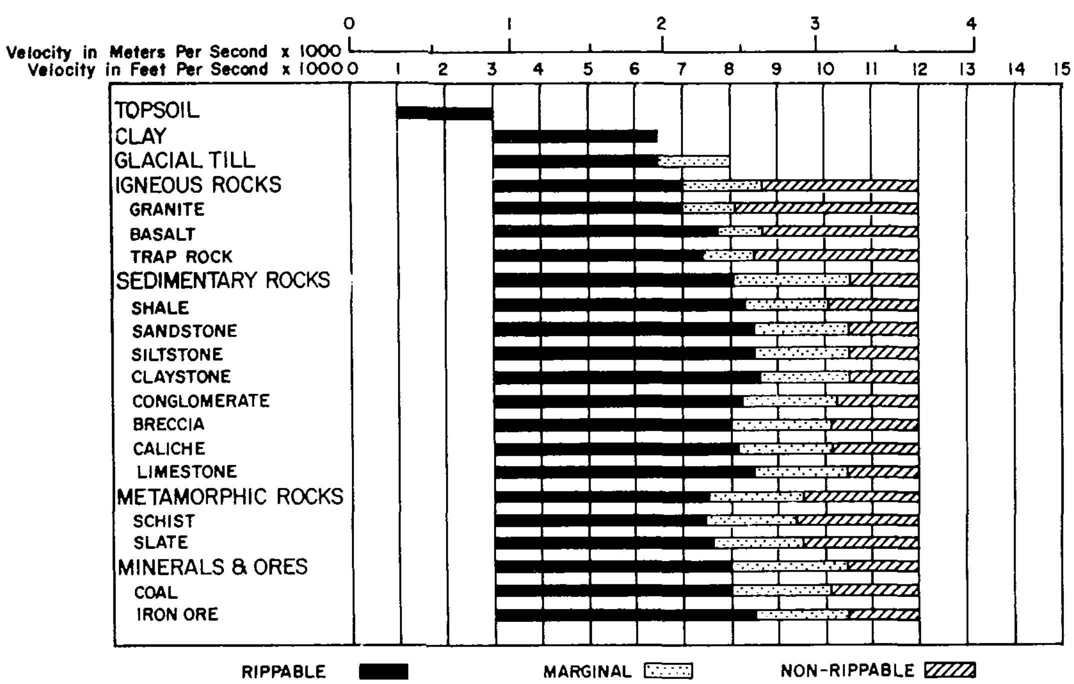

Figure F-4. Generalized seismic compressional wave velocities for various types of rock and soil, with an indication of relative ease of ripping as a method of excavation (From Weaver, 1975).

Excavation characteristics	Velocity for normally weathered profile	Velocity for boulder situations
	m/s	m/s
Easy ripping	450 – 1 200	450 – 900
Hard ripping	1 200 – 1 500	900 – 1 200
Very hard ripping	1 500 – 1 850	1 200 – 1 500
Extremely hard ripping or blasting	1 850 – 2 150	1 500 – 1 850
Blasting	>2 150	>1 850

* Tractor-ripper with a working mass of 45 to 49.5 t and a 280 to 360 kW engine.

Figure F-5. Generalized relationships between seismic compressional wave velocity, as determined from refraction surveys, and excavation character (From Weaver, 1975).

this Section. The HD41 could rip higher-velocity rock because of its 27 percent larger horsepower and 33 percent greater weight.

The CALDOT study also found that at about 1600 mps, rock may require at least an accessory blasting program to introduce fractures and displacement in the otherwise nearly unrippable rock. Weathering provides a general assist to ripping in general softening of minerals and opening up of microfractures and joints. In some cases, weathering leads to softening of feldspars which themselves begin to crush at about 1670 mps, according to the CALDOT study.

Weaver, (Weaver, 1975) working in southern Africa, has developed an integrated system of assessing rippability on the basis of the seven most important factors of rock mass strength:

- Seismic velocity (Fig. F-4)
- Rock hardness (Fig. F-6)
- Degree of Weathering
- Joint (discontinuity) spacing
- Joint openness and filling
- Attitude of major discontinuities
- Joint continuity

Weaver's scheme (Fig. F-7) assesses each of the factors on a numerical rating basis, the maximum scoring being represented as the value assigned to the most intact or "very good" rock and the least values assigned to "very poor" rock. As with all other summary rating schemes that have been developed for use in rock engineering, this system relies heavily on engineering judgment based on sound and representative

Rock hardness description	Identification criteria	Unconfined compression strength	Seismic wave velocity	Excavation characteristics
		MPa	m/s	
Very soft rock	Material crumbles under firm blows with sharp end of geological pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger pressure.	1,7 – 3,0	450 – 1 200	Easy ripping
Soft rock	Can just be scraped with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer.	3,0 – 10,0	1 200 – 1 500	Hard ripping
Hard rock	Cannot be scraped with a knife; hand specimen can be broken with pick with a single firm blow; rock rings under hammer.	10,0 – 20,0	1 500 – 1 850	Very hard ripping
Very hard rock	Hand specimen breaks with pick after more than one blow; rock rings under hammer.	20,0 – 70,0	1 850 – 2 150	Extremely hard ripping or blasting
Extremely hard rock	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.	>70,0	>2 150	Blasting

Figure F-6. Interrelationships of relative rock hardness, compressive strength, and seismic compressional wave velocity, with excavation characteristics (From Weaver, 1975).

Rippability rating chart

Rock class	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Seismic velocity (m/s)	>2 150	2 150 – 1 850	1 850 – 1 500	1 500 – 1 200	1 200 – 450
Rating	26	24	20	12	5
Rock hardness	Extremely hard rock	Very hard rock	Hard rock	Soft rock	Very soft rock
Rating	10	5	2	1	0
Rock weathering	Unweathered	Slightly weathered	Weathered	Highly weathered	Completely weathered
Rating	9	7	5	3	1
Joint spacing (mm)	>3 000	3 000 – 1 000	1 000 – 300	300 – 50	<50
Rating	30	25	20	10	5
Joint continuity	Non continuous	Slightly continuous	Continuous – no gouge	Continuous – some gouge	Continuous – with gouge
Rating	5	5	3	0	0
Joint gouge	No separation	Slight separation	Separation <1 mm	Gouge – <5 mm	Gouge – >5 mm
Rating	5	5	4	3	1
**Strike and dip orientation*	Very unfavourable	Unfavourable	Slightly unfavourable	Favourable	Very favourable
Rating	15	13	10	5	3
Total rating	100 – 90	90 – 70**	70 – 50	50 – 25	<25
Rippability assessment	Blasting	Extremely hard ripping and blasting	Very hard ripping	Hard ripping	Easy ripping
Tractor selection	–	DD9G/D9G	D9/D8	D8/D7	D7
Horsepower	–	770/385	385/270	270/180	180
Kilowatts	–	575/290	290/200	200/135	135

* Original strike and dip orientation now revised for rippability assessment.
** Ratings in excess of 75 should be regarded as unrippable without pre-blasting.

Figure F-7. A method of rating rock in terms of rippability as a function of eight physical characteristics and properties, and the equipment size required for ripping (From Weaver, 1975).

field geologic observations. The rationale behind the scheme is sound and the method represents an excellent means of incorporating the most important rippability factors into a weighted assessment for a body of rock characterized by uniformity within each of the seven factors. In using Weaver's system, engineers and geologists should also take care to identify geologic boundaries and structural geologic domains for which the factors are different and which will therefore produce different rippability ratings.

F.6 BLASTING AS AN EXCAVATION METHOD

Blasting is an expensive method of rock excavation. However there are a number of reasons why blasting may be chosen, either as the preferred method of excavation, or as the only practicable method. Lutton (1977) has classified these reasons as separate construction-related criteria (Table F-2).

For most transportation project work, blasting will be used to remove rock and generally in non-sensitive locations. However, Table F-2 should be consulted in the course of developing bid document requirements for rock excavation indicators and the requirements of the project should be reflected directly in the specifications for blasting, in the bid documents (see Section F.7).

The Agency may also require specifications for use in the construction process. For rock slopes and for the walls and faces of underground structures, the wall rock remaining at the end of blasting must also be intact according to the requirements of the project. Some of the concerns that should be addressed in specifying, designing or monitoring rock blasting operations are shown in Table F-3 (modified from Lutton, 1977).

A primary understanding of blasting mechanics is essential for personnel who are charged with designing rock excavation programs, with preparation of specifications and in monitoring construction activities. Although the contractor generally has the option of determining type of explosive, shot patterns, delays and other facets of blasting, the Agency may

Table F-2
Criteria for Blasting in Rock Excavation

Sensitive Blasting	Undertaken in such near proximity to have a damaging or otherwise unfavorable effect on existing structures or human activities.
Restricted Blasting	Conducted in the vicinity of slopes or foundations which may suffer unacceptable damage; generally in the construction area.
Direct Rock Blasting	Excavation removal in the course of construction or in quarry operations, so that the fragmented rock is hauled and used directly as a construction material.
Crusher Source Blasting	Blasting used to produce a feedstock for mine or quarry crushing operations without strict adherence to fragment size.
Rock Removal Blasting	As required for removal of rock for ensuing construction; muck is to be wasted or used for a non-size-critical purpose.
Specialized Blasting	Employment of such techniques as pre-splitting and fracture control blasting to achieve a desired breakline at the edge of the blasted area; also includes underwater removal of rock masses which may hinder navigation.

[1] As modified from Lutton, 1977

Table F-3
Maintenance of Natural Conditions in Rock Subjected to Blasting

In Rock Masses	*At Rock Slopes*
Modulus of elasticity	Rock modulus of elasticity not to be degraded
Existing permeability	Minimum of tensile fractures along the breakline
Shear strength parameters, cohesion and friction, along surfaces of discontinuities	Avoidance of offsets or displacements along rock joints, bedding planes, or other discontinuities
Appropriate roughness at concrete pour line for base or faces of facility structures	Minimal backbreak along the face or slope crest Avoidance of ensuing slope movements

wish to retain some degree of control over the process with special respect to the nature of fragments produced in rock breakage, the condition of rock along the breakline, and the vibrations, noise and air pressure felt by abutters. Agency personnel at the job site should be familiar with the physical indications of improper or non-optimal blasting so that supervisory personnel may be advised of conditions contrary to what has been specified or otherwise intended in the contract documents.

F.6.1 Explosives

Chemical explosives come in a wide variety of types, detonation velocities and strengths, and other characteristics. Agency representatives should take note of these characteristics in daily reports and in efforts to associate the character of rock produced by blasting and that of the resultant breakline surface, with the contractor's blasting program. Such notes will be helpful in ongoing evaluations of contractor performance and in later discussions or legal actions. Basically, the important characteristics of explosives are:

- *Strength:* commonly expressed as percentages by weight or by volume (cartridge strength), with the percentage referring to the explosive agent as mixed with filler.
- *Detonation Velocity:* the speed, generally in feet per second, at which the explosive detonation wave travels through the explosive.
- *Density:* Measured in terms of specific gravity; generally in the range of 0.6 to 1.7 gm/cm^3.
- *Water Resistance:* qualitative measure of resistance to deterioration when submerged in water, when loaded in a wet shotholes.

Table F-4
Generic Types of Explosives and Blasting Agents

Dynamite

Straight nitroglycerin variety: in decreasing usage

Straight nitroglycerin ditching variety: highly sensitive explosive useful in sympathetic detonation without use of detonators and placed in linear arrays

High-density Ammonia (Extra) variety: most widely used; favorable handling qualities, lower detonation velocity less fuming

Low-density Ammonia (Extra) variety: produces a slow, heaving action; well-suited to softer rock such as clay shale or in production of coarse fragments such as riprap

Gelatin

Blasting Gelatin: powerful, very high-speed, water-resistant; emits large volumes of noxious fumes

Straight Gelatin: water-proof, plastic-type explosive; suitable for use with hard rock, as a shothole bottom charge and in underwater rock removal

Ammonia Gelatin: cheaper substitute for high-density ammonia dynamite; water resistant, good fume qualities; a favored underground explosive

Semigelatin: comparable to low-density ammonia dynamite; good fume qualities; a favored underground explosive

Blasting Agents

Dry Blasting Agents: also known as ANFO (Ammonium Nitrate and Fuel Oil); if not premixed, not considered an explosive until such is accomplished; pours into shotholes; safe, easy to handle, relatively cheap

Slurries: depending on ingredients, can be classified as either an explosive or a blasting agent; require priming by high explosive detonators; good explosive coupling in boreholes; higher charge loadings possible per shothole.

Explosives are manufactured in a wide variety of ingredient types. The essential characteristics mentioned above vary considerably with types of explosives and regional and contractor-specific preferences. Changes of explosive type in the course of an ongoing project are generally limited, the major variations being in the nature of the blast setup itself; shothole depth, spacing, stemming, and the like. Some of the generic types of explosives and blasting agents are shown in Table F-4 (Dick, 1968).

F.6.2 Mechanism of Explosive Rock Fragmentation

Detonation of an explosive produces a spherically-advancing shock wave which, in turn, develops four spherical zones of rock stressing, as shown in Figure F-8. The explosion cavity, innermost and outermost zones, and the seismic zone do little to produce broken rock of a useful nature. Most rock is fragmented in the crushed zone and the blast-fractured zone. There is a crude relationship between the width of the crushed zone and the compressive strength of the host rock. According to Atchison and Pugliese (1964), the crushed zone generally extends outward from the shothole to only about twice the charge radius, making the blast-fractured zone the main volume of rock breakage. This zone is usually some six times that of the charge radius. Due to attenuation of blast energy, through dislocations and other rock fragmentation action, the degree of rock fragmentation decreases radially outward, leaving a condition of increasing spacing outward, between blast-induced rock fractures.

Aside from fragmentation of rock which was otherwise unfractured before the blast, the explosive action tends to promote fragmentation by spalling. Spalling represents increased tensile-stress splitting of incipient discontinuities such as microfractures in the rock, as well as the separation and breaking of cohesion along bedding planes and cemented or rough joints and other discontinuities. At most of these discontinuities, the passing tail portion of the incident compressional wave is transferred into a reflected tensile wave and the rock at each particular point of incidence is racked with an elastic rebound, tending to fragment many rocks with relatively low tensile strengths. The greater the ratio of difference between compressive and tensile strengths, the more pronounced is rock breakage by spalling. Tensile strengths of less than about 1.03×10^{-7} N/m^2 (1500 psi) may be considered as having a low tensile strength.

F.6.3 Basic Surface Blasting Techniques

In order to take advantage of the spherical propagation of rock-breaking shock waves, most blasters use geometric shothole patterns designed to achieve optimal breakage between individual shotholes. The basic patterns are rectangular, staggered, and single-row (Fig. F-9). Minor variations are employed wherever single obstacles of unusual breaklines are encountered. The presence of a slope face, bench, or other rock to free-air interface calls for consideration of delay firing in order to accommodate the relative lack of confinement in the direction of the free face. Delays are employed to produce successive free faces within a single shot pattern. Properly designed delays can achieve optimal fragmentation, reduced throw of rock fragments, control over the extent of rock breakage, and control of ground vibration associated with the blasting. Figure F-10 depicts the seven basic delay

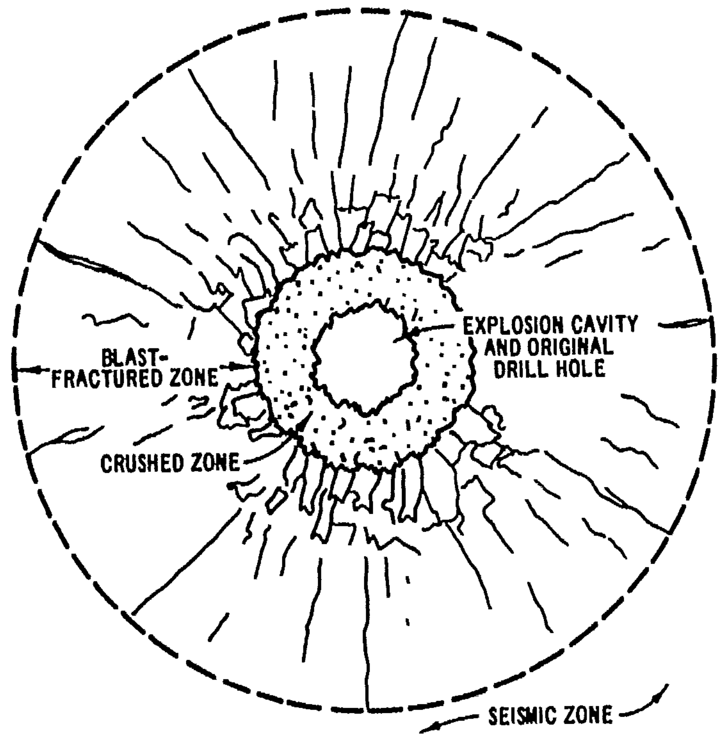

Figure F-8. Geometry of an explosion as viewed perpendicular to a horizontal plane penetrated by the shothole at the center of the explosive charge (US Army Corps of Engineers, EM 1110-2-3800, 1972).

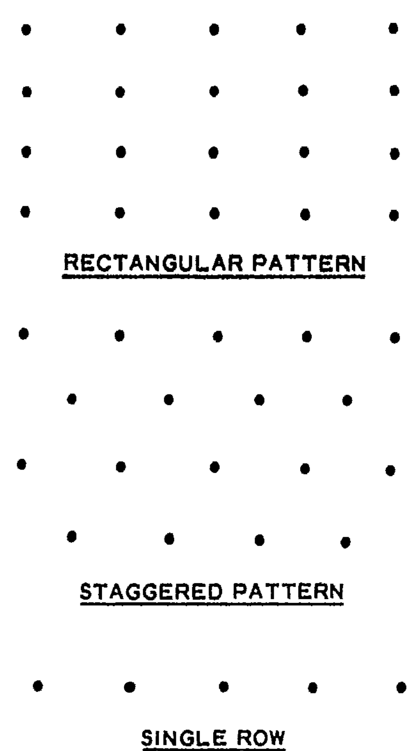

Figure F-9. Typical blasting patterns (US Army Corps of Engineers, FM 1110-2-3800, 1972).

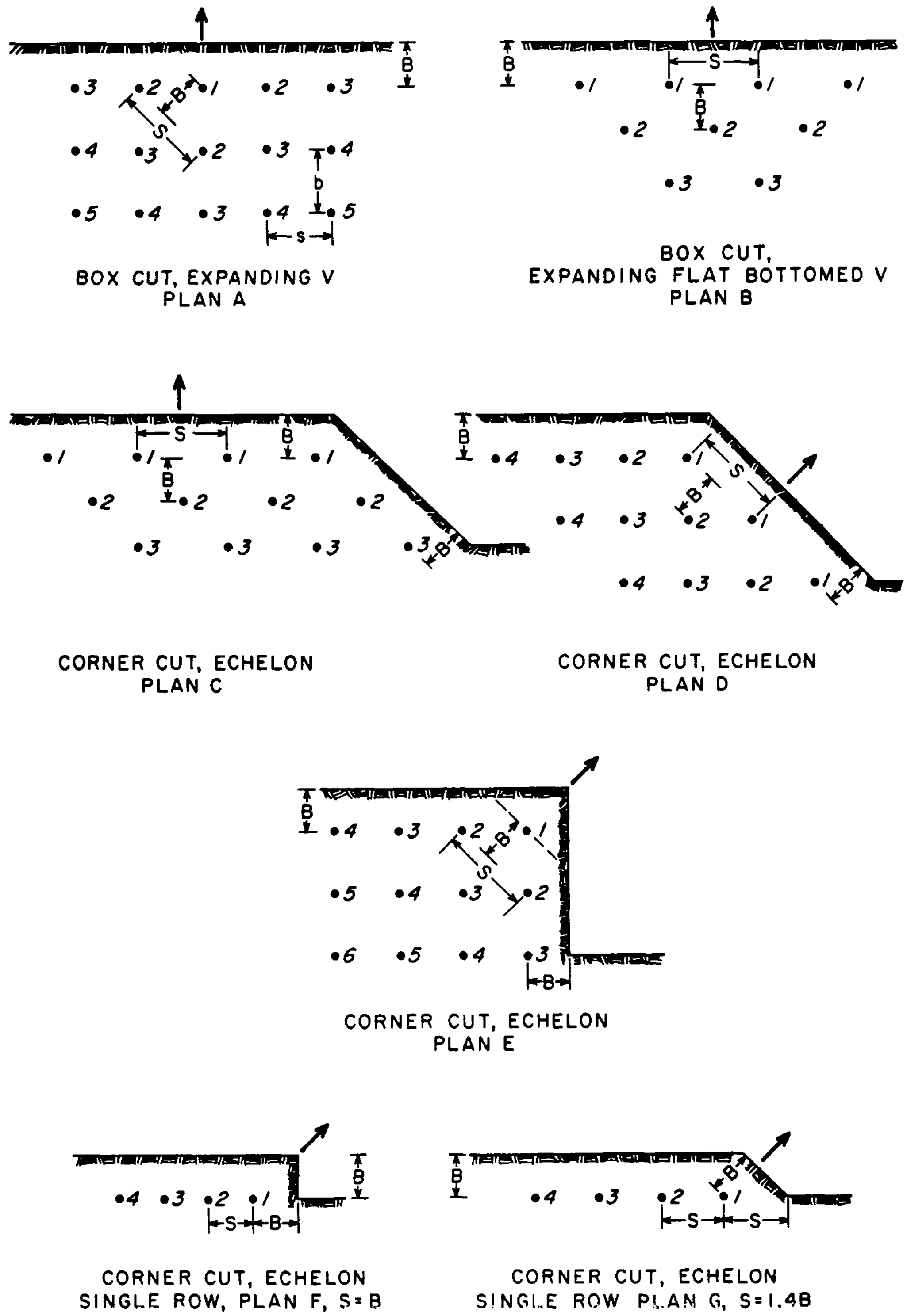

Figure F-10. **Seven basic blast delay patterns, all of which are influenced by the geometry of nearest rock free face (From Pugliese, 1972). Dimensions S and B are scaled proportionately from blast layout plan.**

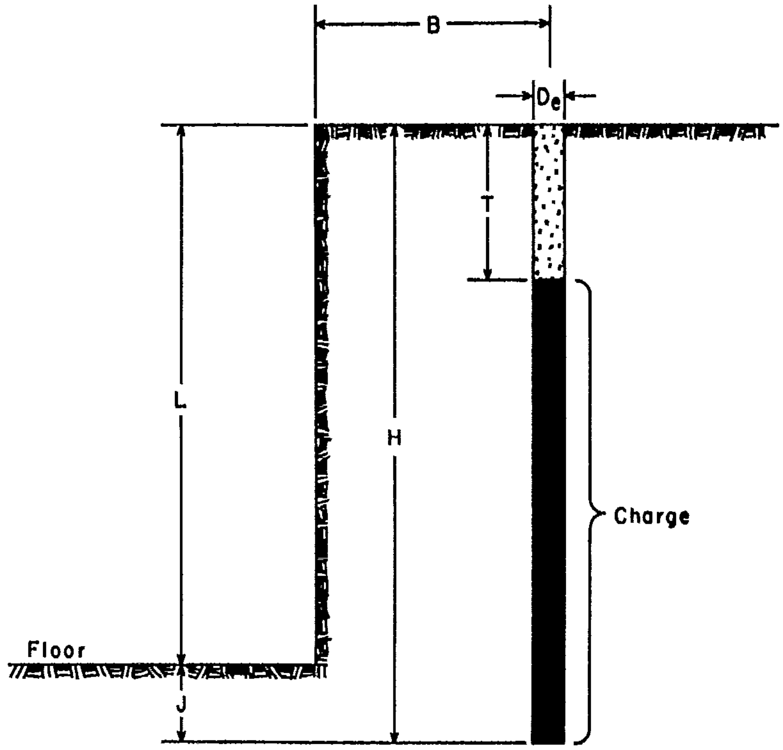

Figure F-11. Geometric variables used to describe shothole placement in blast patterns (From Pugliese, 1972).

patterns. The numbered sequence indicates the usual firing order, which is designed to continually remove confinement and achieve unidirectional rock breakage by enhanced spalling, at the same time placing the broken rock slightly outward into the excavation without excessive throw. When delays are used, the ground shock wave is perceived as a momentarily-longer rumble of sound and vibration and the peak amplitude of ground motion resulting from the blast is significantly reduced.

Many surface blasting programs are inadequately planned with respect to accommodating geological characteristics of the host rock. Geological characteristics of the host rock exert the greatest of all controls over the results achieved from blasting. When the contractor is willing to accept the results of non-geologically planned blasting and these results are within the limits of contract specifications, little action by the Agency is possible or desirable. However, if the results of blasting do not meet the specifications or if the contractor is experiencing severe difficulties of a non-profitable nature, the resident engineer or geologist should at once alert Agency superiors and continue to maintain a careful record of the elements of the blast program, as well as the geologic controls present in the rock.

Geologic notes maintained by the Agency resident should include typical shothole geometry (Fig. F-10 and F-11), charge distribution and delay pattern, representative measurements of rock discontinuities, an occasional hand specimen of rock from specified shots, and sketch maps at the exposed face after selected blasts, with photographs noting the size and gradational spread of rock produced by the shot. The equal-area projection is best suited for plotting points to poles of discontinuities mapped at the face and held to be representative of a given volume of rock affected by the shot. In general, the equal-area plot should be constructed to note if the shothole pattern and delay sequence is taking advantage of dominant discontinuity orientations, as weighted by the geologist in terms of spacing and surface characteristics of each set of joints or other pre-existing rock fracture. Agency policy must be followed carefully with respect to making such studies or mapping available to the contractor as well as in making comments which may later be construed to represent direction of the contractor by Agency personnel. As is well known in construction circles, "direction" by the owner or the owner's representative may be held as the basis for payment of claims for work outside the scope of the contract. Findings of non-compliance with geological conditions should be filed directly with the observer's Agency supervisor for appropriate action.

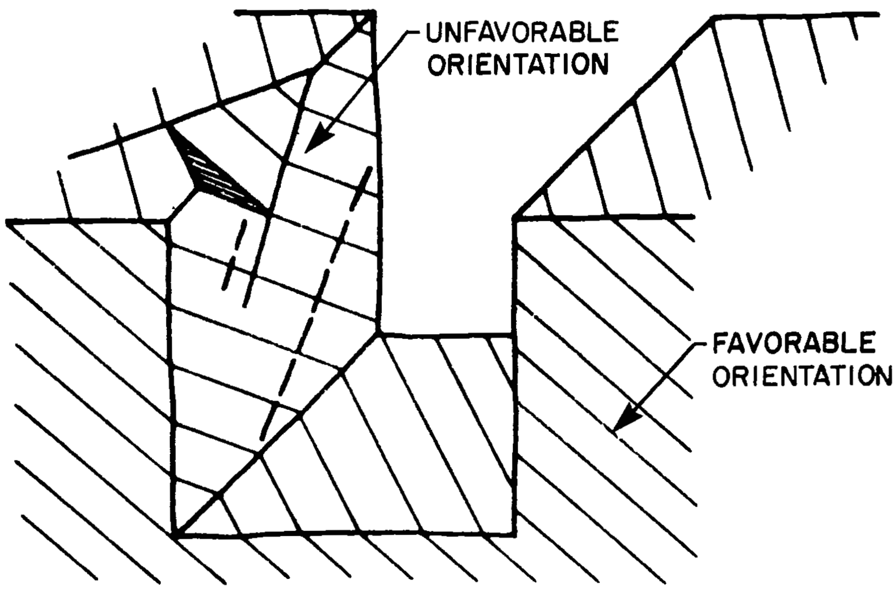

Figure F-12. Rock discontinuity orientation viewed as favorable or unfavorable in terms of free-face stability in open excavations (US Army Corps of Engineers, EM 1110-2-3800, 1972).

The blasting program should be altered whenever bodies of rock are encountered which vary from adjacent rock in terms of lithology, degree of weathering or alteration, discontinuity spacing, and orientation of bedding.

F.6.4 Effects of Discontinuities

Discontinuities have three basic effects on rock excavation by blasting; attenuation of blast energy, lower resistance to fragmentation of those rock fractures lying essentially perpendicular to incident blast waves, and the potential for blast-related and later gravitationally-induced failure of blocks of rock along discontinuities which pitch downward and toward the open face of the excavation. Orientation is described in terms of effect on blasting by the terms *adverse* and *favorable* (Figure F-12). Adverse orientation is represented by discontinuities with strikes lying at or nearly parallel to the nearest free face of rock excavation and with dips inclined rather steeply into the excavated area. The various types of discontinuities (see Section 4) affect rock excavation in the following ways:

- *Joints* are usually the most common of discontinuities in hard rocks (Figure F-13). Joints are usually the result of previous periods of tectonic stressing in three-dimensional stress fields. Such stressing has usually resulted in the formation of the joints in brittle elastic failure of the rock mass. Some joints, such as those found in volcanic, igneous plutonic and metamorphic rock were formed by thermal effects incidental to the origin and emplacement of the rock masses. Where found, joints usually occur in statistically-prevalent groups or *sets* and can be evaluated by the use of equal-area polar plot diagrams (see Section 4). In rock which has been subjected to recurring tectonic or other stressing throughout geologic time, the number of joint sets may increase in representation of changes in stress field orientation between each episode of stressing.

  Joints are often filled by later mineralization and can often be found in such a *healed* condition as to be essentially stronger than intact rock itself. Such joints should be identified and carried separately in evaluations of the effect of jointing on blasting programs.
- *Faults, shear planes and shear zones* are essentially joints and groups of semi-parallel or parallel (zones) joints along which displacement has occurred between the opposing surfaces of rock. In increasing order of magnitude of width and amount of displacement are *shear plane, shear zone, fault* and *fault zone.* Some fault zones are hundreds of meters in width and extend for hundreds of kilometers in length. Faults and their related discontinuities usually represent a greater degree of rock breakage than is desired from blasting and hence exceed the positive effect of joints in assisting rock breakage. Faults are generally considerably less prevalent than joints and may serve mainly to diminish the effect of adjacent blast detonation and can create gas venting.
- *Dikes and sills* are tabular bodies of intrusive

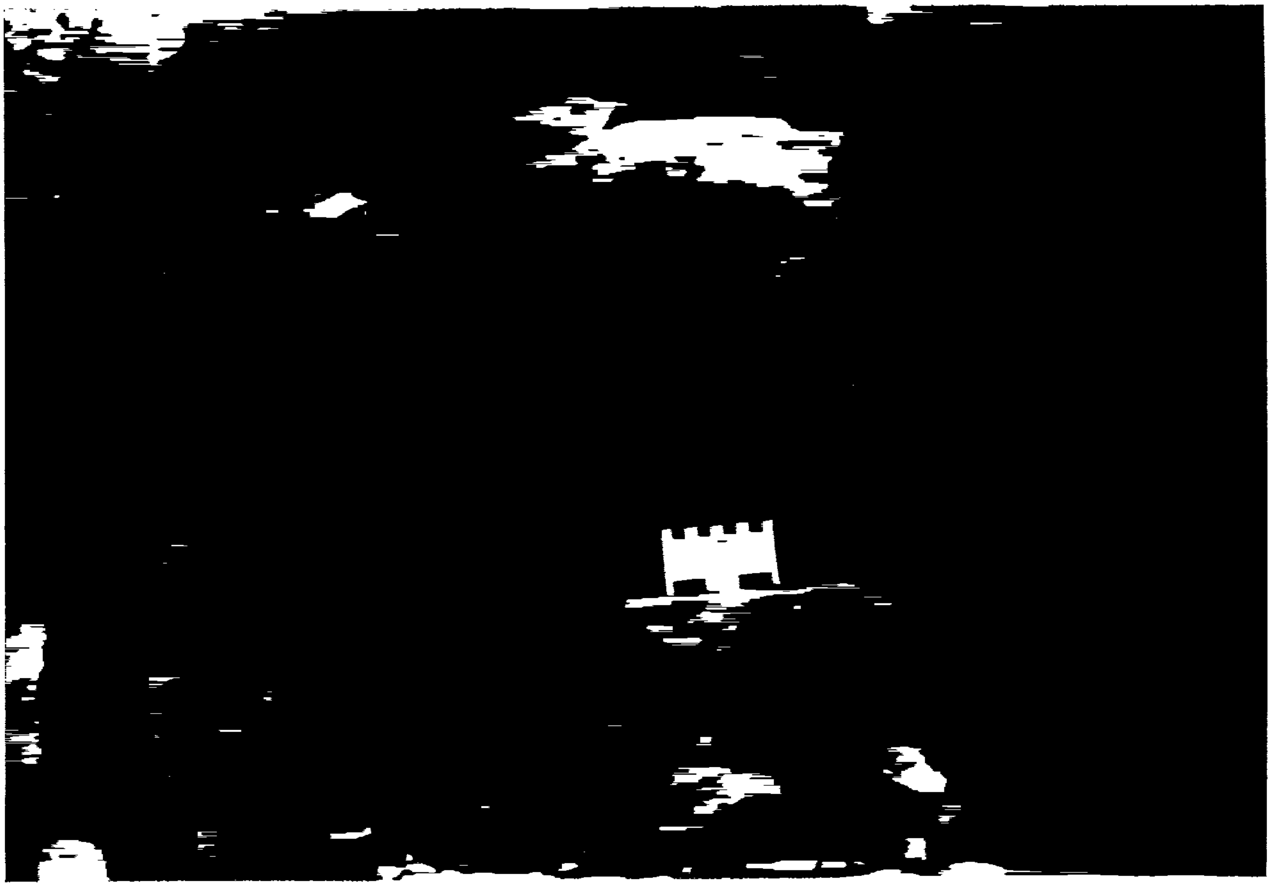

Figure F-13. Joints typical of hard, competent limestone, showing presplit, line-drilled shot hole to the left of the view. Joints are fractures along which no discernable deformation or slippage has occurred. (A.W. Hatheway)

rock and are important to blast programs because they are generally of a different rock type than the host rock. The dike rock usually represents a different blast medium with respect to drillability for shot holes and rock fragmentation characteristics. These intrusives should be mapped for their general response to blasting, and for attitude, position, rock type, width, and absence of weathering or alteration.

Dikes and sills were generally intruded along preexisting discontinuities such as faults and joints and are also frequently the place of weathering or alteration that is different from the host rock. Alteration may be found at considerable depths and may reduce the dike rock to a weak material that cushions shock waves and is detrimental to the blasting program.

- *Bedding* represents the primary structural discontinuity of sedimentary rocks and is the repetitive, parallel occurrence of planar separations between layers of variable grain size and/or mineralogical/lithological content. Many sedimentary rocks possess bedding which is not open and in the form of discontinuities and which has no direct effect on rock breakage; however, this is not the usual case.

F.6.5 Other Important Geologic Features

In addition to discontinuities, several other geologic features can be present which will control the effect of blasting. These are the general effects of *rock fabric* and the more localized effects of *zones of weathering and alteration* and *cavities and voids.*

- *Rock Fabric* is the overall arrangement of the constituent minerals making up the rock and the interactive effect of many factors making up intrinsic rock strength. These factors are the size and spatial orientation of the mineral grains, the nature of the bonds between these, other grains, and the matrix of the rock, microfractures lacing the rock, partial and total alteration of individual minerals and evidence of locked in tectonic or residual stress from past episodes of rock stressing. Rock fabric is best analyzed by individuals skilled in engineering petrography (see Appendix E) In crystalline igneous rocks, such as dimension-stone-quality, granitic rocks, a skilled observer can detect microscopic fabric effects that produce a *grain* which imparts clean and continuous breaks in the rock on an orthogonal and repetitive pattern.

- *Zones or Pockets* of rock weakness are sometimes found in otherwise stronger rock and represent subtle physical changes in original rock characteristics resulting from groundwater alteration. Zones or pockets of degraded rock are more common in other than sedimentary rocks and are of a very unpredictable nature. Even in previously glaciated terrain, rock excavations often encounter pockets of rock so weathered as to be shovel-excavatable. Due to the gouging and plucking nature of the ice sheets, such pockets of rock are generally small.
- *Cavities and Voids* are of concern in any carbonate rock which is soluble in its present or past groundwater. Cavities may be large, in the instance of limestone caves, but for most projects, cavities and voids are man-sized or smaller in proportion and often filled with water of water-saturated fine soil accumulations brought into the cavity by down-gradient cleft water flow. Shothole drillers should be able to detect the presence of cavities encountered by the drill string by greatly reduced penetration resistance or by tell-tale rod drops. The cavities and voids are negative factors in rock excavation for most purposes and, if water filled, may result in changes in explosive type and method of charge placement.

F.6.6 Damage Prediction and Control of Blasting Operations

Rock excavation by blasting is usually designed to expend the least amount of funds to create the desired volume of broken rock whether for creation of a cut or underground opening and/or to create construction material. Agency personnel are concerned with the quality of rock and rock surfaces at the edge faces of the void and with the character and quality of the rock muck that is produced during the blasting program. Although the contractor is usually made to assume responsibility for damages associated with the blasting operation, both to the public and to the rock quality along the excavated faces, it will be in the best interests of the Agency to help insure that the blasting program does fulfill these objectives. Most of the available Agency control must be exerted through the medium of the contract specification for blasting (see Section F.6.7), but the Agency may elect to monitor (Fig. F-14 and F-15) the strength and character of

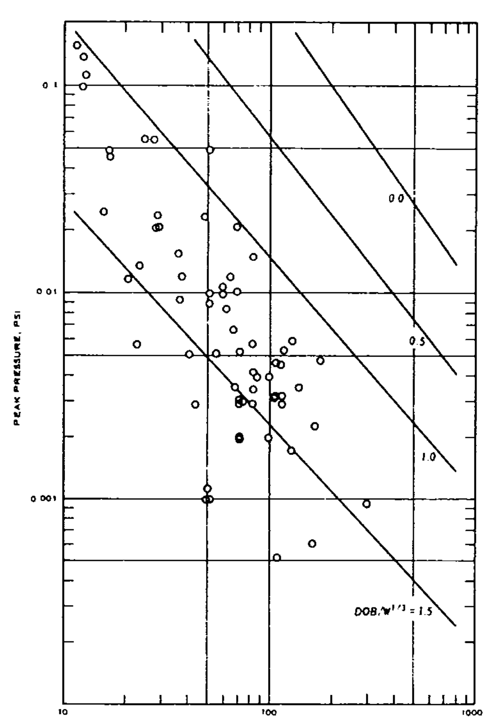

Figure F-14. Propagation relationships for Fig. F-11 airblast pressure from spherical changes at various scaled depths of burial. D = distance from point of interest (ft.); W = change size (lbs.) (US Army Corps of Engineers, EM 1110-2-3800, 1972).

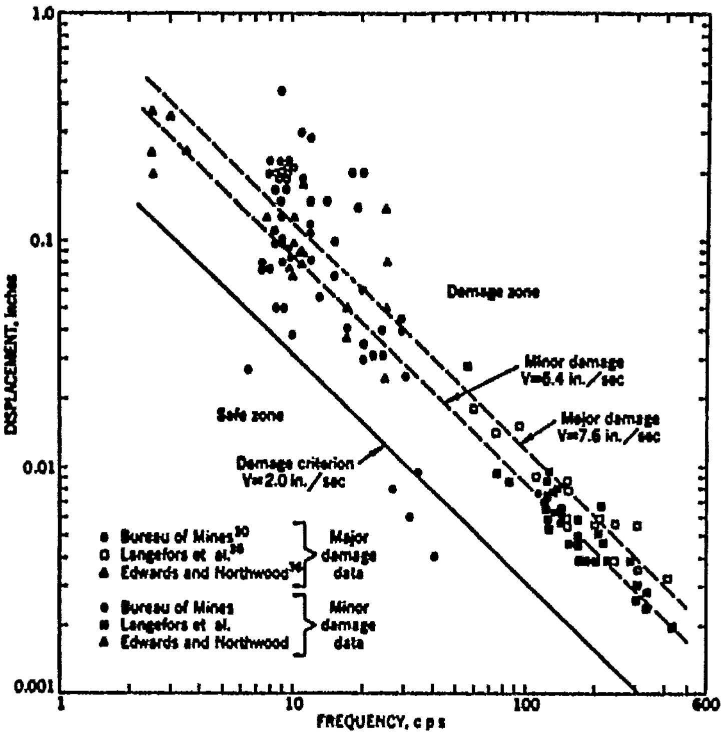

Figure F-15. Summary of damage criteria for blast-generated ground motion affecting frame structures (US Army Corps of Engineers, EM 1110-2-3800, 1972; after U.S. Bureau of Mines, Duvall, W. I. and Fogelson, D. E., 1962).

blast-generated ground and air waves in order to act when the blasting program is not following specifications or to deal with an otherwise unexpected problem of public safety or nuisance stemming from blasting on the project.

Monitoring of the effects of airblast and ground shock waves is generally undertaken using a special blast vibration seismograph with accessory piezoelectric airblast pressure gage. This is a one-man operation and may be conducted at specified or unannounced intervals and at such times at which the contractor elects to change the ongoing blast program parameters. The records should be carefully annotated as to time and location of the instrument and the nature of blasting in terms of charge size, placement, delay and other key factors making up the blasting program for that shot. The seismograph should be moved from time to time in order to register effects along different bearings from the shot area in the event that localized geologic conditions are affecting the nature of shock wave transmission radially away from the shot area.

Geotechnical personnel are often asked to undertake physical inspections of structures surrounding the area to be blasted, in order to establish the relative condition of buildings and other facilities, before initiation of blasting. If the issue of possible blast-related damage is critical to the abutters or public in general, it may be wise to select a consultant to make independent evaluations by way of personal interviews of the abutters and photography of existing cracks and fractures in the structures.

Monitoring personnel may wish to make use of existing empirical relationships for airblast propagation and levels of ground vibration as related to distance/charge magnitude and dominant frequency of vibration. Again, as Agency personnel generally do not participate in the contractor's operation, the data may be made available for contractor inspection or may form the basis for reports to Agency superiors for appropriate action.

- *Airblast Propagation* Airblasts are pressure waves created by venting of high-pressure explosive-generated gasses to the atmosphere and by conversion of ground shock waves to air vibrations at free-air interfaces (such as bench faces). The depth of charge burial is related to the scaled distance (D = distance in feet; W = charge size in lbs.) and expected peak pressure of the resultant air blast for multiple-hole, stemmed quarry shots (Figure F-11).
- *Ground Vibration* Most of the basic relationships used to explain levels of ground vibration

began with the work of Theonen and Windes (1942) and of Crandell (1949) and are summarized in Figure F-13. Agency personnel will be most concerned with the damage to abutting structures with existing damage which may be made more pronounced by project blasting, or which may appear to be originated or aggravated by the abutters. A goal for blast control may be to specify compliance with threshold vibration damage levels taken from the plots of Figure F-15.

F.6.7 Blasting Specifications

Specifications may be developed to provide basic controls over project blasting, in terms of damage potential to abutters, the nature and quality of rock produced as muck and the condition of free-standing rock faces after blasting is completed. Some of the key aspects of a blasting specification are as follows (modified from Lutton, 1977):

General Requirements

- Review of a detailed proposed blasting plan, by the Agency representative, before the start of drilling for each shot; requirement for submission of a simple sketch for the record
- Use and placement of presplitting charges
- Restriction of blasting within designated proximity to curing concrete or grout and minimal spacing between charge centers and concrete or grout of any age
- Scaling of permanent slopes or faces remaining at the conclusion of blasting

Perimeter Control Methods of Blasting

- *Presplitting* to achieve prescribed final cut slopes and faces
- *Line drilling* to achieve critical tolerances at designated breaklines
- *Zone blasting* (also known as cushion blasting) to create a buffer of broken rock left in place as a protective barrier for minimization of damage to critical faces; the buffer to be removed in the last stages of cleanup

Special Restrictions

- Maximum acceptable depth and inclination deviation of presplit shotholes
- Depths of individaul shot lifts (a minimum and maximum figure constitute the acceptable range)
- Minimum separation distances from shot pattern to sensitive structures or rock faces and slopes

Precautions At or Near Final Grade and Final Slope

- Avoid subdrilling which may tend to weaken or break rock below final grade
- Reduce spacing, burden and powder factor on shot holes adjacent to presplit surfaces
- Use delay patterns especially designed to provide relief of confinement for the shot row nearest the presplit line
- Provide lines, grades, and tolerances on drawings.
- "A" and "B" lines (maximum and minimum limits) are shown as the basis for payment of unit rock excavation prices. No rock will remain inside the excavated area as defined by the "A"-line. Measurement and payment is made to the "B"-line. Rock broken beyond the "B" line will result in nonpayment and replacement by the contractor with suitable fill material at no extra cost to the Agency. "A" and "B" lines are the tolerances for rock excavation.

Achievement of Desired Quality of Muck (Figure F-16)

- Choice of lower-velocity explosives or blasting agents to reduce percentages of unwanted, blast-produced fines;
- Choice of higher-velocity explosives or blasting agents to produce high fragmentation, and;
- Modification to the blast pattern delay sequence and quantity of explosives employed.

F.7 PRE-BID EXCAVATION TESTS

For projects in which a significant amount of rock is planned for excavation, the Agency can undertake a pre-bid rock excavation test program. The program should be designed for conduct at one or more locations in the area slated for excavation and should test the effects of machine and blast removal of rock of representative types as identified by the project geologist. The test can be made as a separate demonstration contract and should attempt to define optimal blast design, the desired rock-break gradation for subsequent material usage, and to provide raw data for contractor interpretation. The study should be designed to return benefits in terms of reduced contractor bids, through removal of contingencies.

Figure F-16. Nature of excavation muck is a cost-controlling factor. This muck, from a transit tunnel, is about as broadly graded as can be employed for a variety of useful site-fill, hence reducing the costs of its management/disposal. (A.W. Hatheway)

F.8 ESTIMATION OF BULKING

Balanced cut and fill estimates for rock excavation depend heavily on estimation of the *bulking factor* (also known as *earthwork factor*). Thus is the ratio of embankment (fill) volume to excavation volume. A factor in excess of unity indicates that compaction or placement density will be less than that found *in situ,* prior to excavation. For factors computed to be less than unity, the fill volume will exceed the volume of compensating cut.

Determination of bulking factors for soils are rather straightforward and depend mainly on the state of preconsolidation of the soil and the density to which design requirements call for compaction of the embankment earthwork. For rock, however, a variety of conditions and characteristics affect the resulting density of rock fill. The California DOT has found that pre-excavation compressional wave seismic velocity (determined by refraction survey) offers the best basis for estimation of the bulking factor. The most recent of a series of reports comparing bulking factor with seismic velocity (Stephens, 1978) contains a number of predictive curves that have been based on careful topographic surveys of pre-excavation and post-excavation surfaces and pay-volume surveys at the roadway embankment.

The California DOT studies indicate that a five-percent accuracy of determination should be attainable if the velocity versus bulking factor charts are constructed for lithologic types which are essentially similar. Two of Stephens' (1978) sets of curves are shown as Figures F-17 and F-18, providing expected bulking (earthwork) factors for sedimentary and granitic rocks. One can note that a general rule of thumb appears such that for most rock types, a compressional wave velocity of about 900 mps (3000 fps) equates to a one-to-one ratio between excavated rock and compacted rock fill.

Some of the factors which should be taken into consideration in setting up bulking factor estimates or confirming studies involving seismic refraction surveys and development of bulking factor curves are as follow:

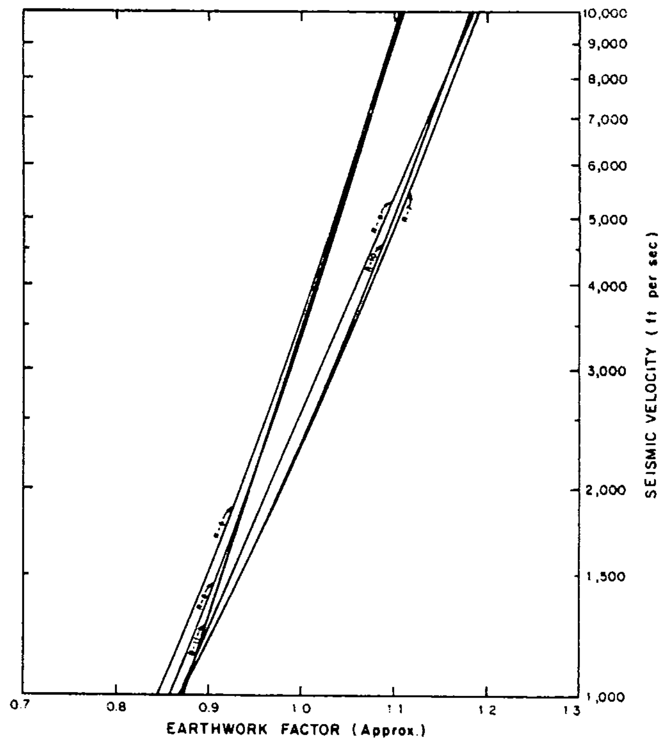

Figure F-17. **Earthwork factors associated with a series of excavation tests in sedimentary rock (Modified from Stephens, 1979). (Stephens, 1978)**

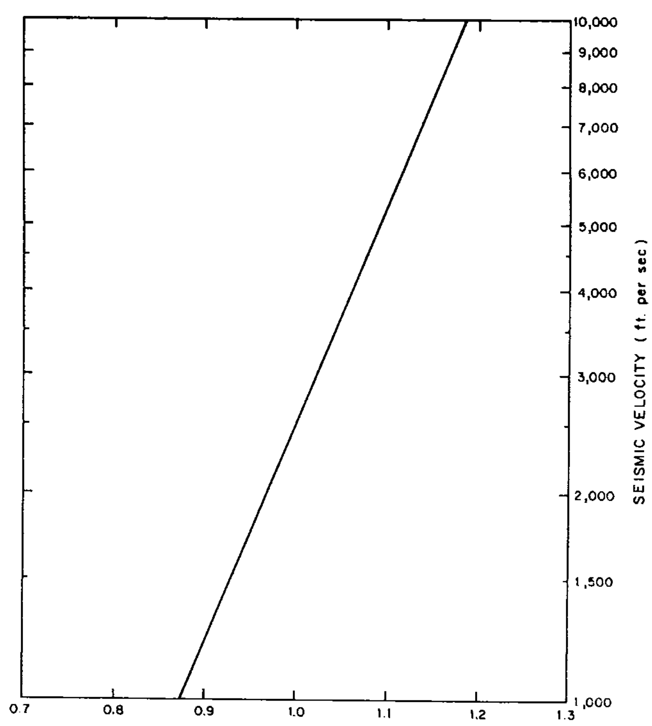

Figure F-18. **Earthwork factors from a single excavation test in granitic rock. (Stephens, 1978).**

Table F-5
Factors Affecting Bulking Factor Estimation by Seismic Velocity

- Shape of rock fragments (dominantly flat fragments can tend to increase the factor)
- Size gradation (gap-grading can tend to increase the factor)
- Degree of compaction (if additional fragment breakage occurs during compaction, the factor may be decreased)
- Loss of materials (materials not reaching the embankment or falling outside its measured limit will tend to decrease the factor)
- Deviation from plans (embankments constructed to other than design specification will result in a variance in the factor from the original estimate)
- Accuracy of volume estimates (errors in determination of excavated and placed volumes will result in a variance in the factor from the original estimate

The California DOT studies further indicated that the upper limit of expected bulking factors will lie something short of 1.3 for most rock types. It is difficult to imagine a rock that would exceed 1.3 and this only due to some unusual aspect of size gradation and particle shape; both probably would lie outside of design specifications and would probably be detected by the contractor or resident engineer as being therefore unsuitable.

Bulking-factor estimates are made primarily for the benefit of the Agency, both in terms of costs associated with achieving balanced cut and fill and in efforts to locate sufficient supplies of rock fill material. As with most other geotechnical aspects of transportation system design, the bulking factor estimates are only as reliable as the geological observations that support them. The refraction surveys made to produce velocity values must be made over ground that is known or thought to be underlain by rock of a similar lithology and condition (with respect to weathering and alteration). This is not to say that the estimates cannot be made for minor variables in lithology and a variety of states of weathering and alteration. Indeed, that is what each set of curves depicts in terms of variable seismic velocity. However, pains must be taken to distinguish the relationship of velocity to lithology or weathering/alteration grade so that the overall curve may be constructed. On the other hand, the confirmation of estimates must also be done on discrete embankment sections constructed entirely of similar rock, so that the resulting measured rock fill volume will represent emplacement conditions in one rock type/grade only.

F.9 GEOTECHNICAL DATA FOR TUNNEL BORING MACHINE EXCAVATION

Tunnel Boring Machines (TBM) represent relatively new technology, having been initially used in the United States at Oahe Dam, South Dakota, in the early 1950s. A significant amount of tunnel advance is now accomplished by TBM and the economic aspects of their employment have resulted in a good deal of competition for design advantages among the various domestic and foreign manufacturers. TBM have been employed in rock with compressive strengths approaching 2.4×10^8 N/m^2 (35,000 psi) and advance rates in the tens of meters per day have been routinely accomplished using machines with drive capacities and cutters correctly matched to rock type and operated by competent individuals.

Matching of machine to geology is a complicated matter of employment of personal experience of the machine manufacturer, his field representative, the contractor, and the contractor's geological or geotechnical consultant. For the most part, these individuals rely on previous experience related to geologic parameters such as compressive strength and elastic modulus, the various rock hardness variables, rock quality designation (RQD), fracture spacing, and lithologic descriptions from core logging (including grade-related assessments of weathering and alteration). These rock classification factors have all been described in Appendix E.

For this reason, geotechnical reports prepared for tunneling and underground structure contracts must contain representative test values for most or all of the tests discussed in Appendix E. The tunneling contractor is most concerned with the following primary and secondary considerations:

Table F-6
Considerations Affecting TBM Usage

Primary Considerations	*Secondary Considerations*
Rate of advance	Nature of muck produced Overall
Cutter wear	Wear on the machine

TBM advance occurs by way of crushing, elastic deformation, and/or fracturing of the rock in contact with the cutters, which are usually disc cutters, rollers, drag bits, or picks. The relationships governing the amount of rock disintegrated by the cutters from the cutting face are complex: the main objective is to break mineral bonds or to fracture or otherwise break and dislodge individual mineral grains in the rock under attack. Indications of these primary rock breakage factors come from visual (petrologic) descriptions, from petrographic descriptions (in which elements of rock fabric and weaknesses are detected), and from laboratory tests defining the general strength and hardness of intact rock.

A second level of geological/geotechnical assessment is then made, concerning the effect of rock structure on the behavior of the overall rock mass. Although the machine can be designed for the purpose of breaking rock under the cutters, there is the companion aspect of actual rock discontinuities in producing discrete blocks of rock which are bounded by two or more edges and exposed entirely within the confines of the cutting face or along the inner surface of the tunnel bore. The main concern in this respect is with the gravitational stability of the rock blocks which are so exposed. If the geometry of the surfaces forming individual rock blocks is such that gravitationally-induced, static forces can overcome the friction and cohesion present along the surfaces, the block is likely to fall into the tunnel during or after passage of the cutter head. Gravitationally dislodged rock blocks can present some difficulties to TBM operation if such occur at the cutting face or around the advancing machine. Blocks dislodging after passage of the TBM must be considered as a problem of ground support and given attention in that respect.

The Agency should provide information relating to the geometry, frequency and surface characteristics of each type of discontinuity observed on the ground surface (in outcrops and in exploratory trenches, if such are used) and from exploratory borings. As noted in Section 4, the generally accepted method of presenting data of strike and dip is the equal-area, polar projection. Outcrop measurements of attitudes are a minimal level of data input for contract documents. Significant expenditures of borehole logging time for geologists and for drilling rig time will be required to upgrade these observations to include oriented data to describe discontinuities encountered in coring. Some of these techniques are discussed in Section 4. The Agency must decide on the economic returns associated with bids prepared on the basis of oriented core observations. Such observations are not generally considered to be basic to tunnel and underground structure prebid packages, and should be viewed as an option of the Agency which may lead to a more narrow and cost-effective spread of bids.

The tunneling contractor should always review all prebid geologic and geotechnical data. This information, as described in Sections 2 and 10, may be prepared on the basis of uninterpreted field and laboratory data, and a separate package of interpretations based thereon. Bidding contractors should always employ geological and geotechnical professionals on their staffs or as consultants to give advice in preparation of the assumptions which will underlie the bids.

In this connection, the presence of groundwater, either as cleft rock water (in the rock mass) or as pore pressure (in soil units) must be considered along with the potential effect of geologic structure on the mining operation and in achieving ground stability in the bored tunnel. One of the basic interpretations that must be made by the contractor is the potential effect of rock discontinuity attitudes and their combinations on TBM performance and tunnel stability.

F.10 ENVIRONMENTAL ASPECTS

Nearly all rock excavation programs now require some form of permitting, ranging from the use of explosives to environmental impact assessment and review. Contracts requiring significant amounts of off-site rock or aggregate materials may often go to those bidders who have a nearby, permitted an operating quarry. The most basic consideration often becomes the choices of routing which affect balanced cut and fill. Rock excavation associated with construction in the ROW itself are not so much subject to the environmental assessment and review process as are quarry operations in support of construction. Any rock excavation slated in or near urban areas should be of great concern to highway planners. These activities will be subjected to close scrutiny by environmental regulatory officials and will be almost certainly targeted by citizen's groups opposed to construction activities.

Costs related to the transportation of natural materials usually rise nearly exponentially with distance from source to project. This is naturally related to time on the road, the requirement of an ever increasing fleet of trucks to provide the basic supply, and problems related to timing of hauls versus traffic patterns on the haul route. The U.S. Bureau of Mines, Twin Cities Mining Research Center, Minnesota, has conducted a program of defining the planning needs for opening new quarries near urban areas. Most of the findings are applicable to providing rock materials for transportation construction projects located in or near urban areas (Pugliese, Swanson, Engelmann and Bur, 1979).

A large and long-term quarrying operation in or near an urban area will quite likely never be feasible, in terms of permitting, especially for the purpose of supplying rock material for only a single project. The permitting requirements are simply too stringent to be accomplished in the limited time frames available. However, along with the overall permitting advice given by Pugliese, and others, (1979) are the initial project and siting considerations which would affect the operation of limited project related quarrying operation on land supplied by the agency or to be developed by bidding contractors. When Agency-supplied land is to be considered for inclusion in the bid package as an available resource, the following basic considerations (modified from Pugliese, and others, 1979) should be investigated and answered at least to the point of indicating that permitting is possible:

Table F-7
Environmental Planning for Rock Excavation

- Production objectives and probable resources in terms of project requirements
- The economics of the local aggregate market and its outlook over the project duration. Includes alternate sources and associated haul costs
- Bonds and permits required
- Estimated cost of environmental assessment required for submission to regulatory agencies
- Nature of the quarry site; topographic, geologic, hydrologic and wildlife/biologic character
- Probable plan of optimal development
- Amount of capital needed to put plans into operation
- Ability to acquire the property and mineral rights (if such apply)
- Previous experience of other rock production activities in the general area
- Local zoning ordinances, and;
- Existing alternative sources of rock, with estimated haul costs and availability during construction.

Many of the inputs to the above review can be accomplished by geotechnical and geological personnel from the Agency field exploration unit. Some of the answers will be based on previous Agency experience, other answers will come from field reconnaissance, and other data can be collected through limited interviews with regulatory agency personnel, rock producing firms, and contractors in the site region.

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APPENDIX G
Instrumentation

Transportation systems are often affected by a variety of physical changes occurring in their foundation soil and rock. Most of these changes are brought about by stress redistribution created by the cuts, fills and foundation loads of transportation system structures. Other changes are brought about by man-induced and natural phenomena in the near vicinity of the structures; others yet are created by changes in the presence and nature of groundwater and pore fluids in the surrounding rock and soil. Almost all of these physical changes are rate-dependent and are sometimes seasonally variable.

Physical changes in the engineering properties of earth and rock beneath and surrounding engineered facilities are always a matter of potential concern to geotechnical and structural engineers. Redistribution of ground stresses or pore fluid pressures are almost always felt to some degree by the various components of embankments, bridges, tunnels, viaducts, piers, retaining walls and other primary and secondary transportation structures. In many cases the evidence of stress redistribution and accumulation lies in microscopic fractures and hair-line cracks in concrete (Fig. G-1), activated fractures in rock, slight displacements in earth embankments and deflections in steel and wood. Whether the component is permanent or temporary in nature, the transmitted stress is transitory and is absorbed in some form of deformation throughout the structure. When yield points of structural components, earth, rock, concrete or steel are exceeded, noticeable damage occurs, which usually impair the function and safety of the transportation system. At this point, the magnitude of structural damage to the system often exceeds the correctional capability of owner or constructor. Instrumentation of engineered structures is a way of detecting present or potential structural damage before the magnitude of deformation becomes uncorrectable.

G.1 NATURE OF INSTRUMENTATION

Instrumentation is a collective term for various mechanical, electrical, hydraulic and optical devices that are designed to actively or passively monitor and record the physical position and/or stress condition of an engineered structure (Fig. G-2) or one or more of its structural components. The instrumentation is designed to detect physical changes that are generally unobservable to the human senses and to make these detections observable to human data collection or to transfer some form of analog of the change to a continuous or periodic data collection device. The design, installation, monitoring and analysis of instrumentation and its output data is a highly specialized and rapidly growing field of geotechnical engineering. There are virtually no formal standards governing the manufacture and installation of instrumentation and the current state-of-the-art of data collection and analysis closely follows the changing environment of design and application of instrumentation.

The purpose of this Section of the Manual is to describe basic classes of instruments and how they can be employed to provide different classes of data that are important to the design and safe functioning of transportation systems. The state of technology of instrumentation is advancing rapidly, mainly through applications of miniaturization and materials science. The Section has as its goal the development of an appreciation so that the user will recognize potential applications for instrumentation and will know the elements of design or specification that will lead to a correct selection of consulting advice or sales/rental

Figure G-1. Mechanical scratch gage which is self actuated and measures dilatancy across an existing fracture in structural concrete or a rock discontinuity (Photograph by J. R. Wheeler).

support from instrumentation experts or firms dealing in instrumentation.

G.2 PURPOSES OF INSTRUMENTATION

Instrumentation should be considered whenever the Agency feels that some otherwise unobservable or undetectable event will adversely affect the construction, operation or maintenance of a project. The best way to play for the use of instrumentation is to carefully analyze the need for such devices and then justify this in terms of construction or operational requirements of the particular project. Some of the more usual purposes of employing instrumentation are shown in Table G-1.

Instrumentation can also be used to provide input data required for theoretical analysis in geotechnical and structural engineering. These are observations such as:

- Earth and rock pressure
- Loads in or on structural members
- Displacements of earth or rock bodies or masses, and structural members
- Tilt or inclination of earth/rock masses or structural members
- Pore or cleft water pressure
- Upper groundwater (piezometric) surface
- A combination of the above, in the form of approach or exceedence of established threshhold or warning levels.

G.3 PLANNING FOR INSTRUMENTATION

A successful program of instrumentation involves creation of a plan for equipment acquisition, installation, training of personnel, monitoring, and data analysis. Most users have extensive catalog collections from manufacturers and yet are not always aware that many suppliers will design variants of devices that more nearly fit the requirements of the user.

The instrumentation plan can be subdivided into major components of *objectives, equipment, person-*

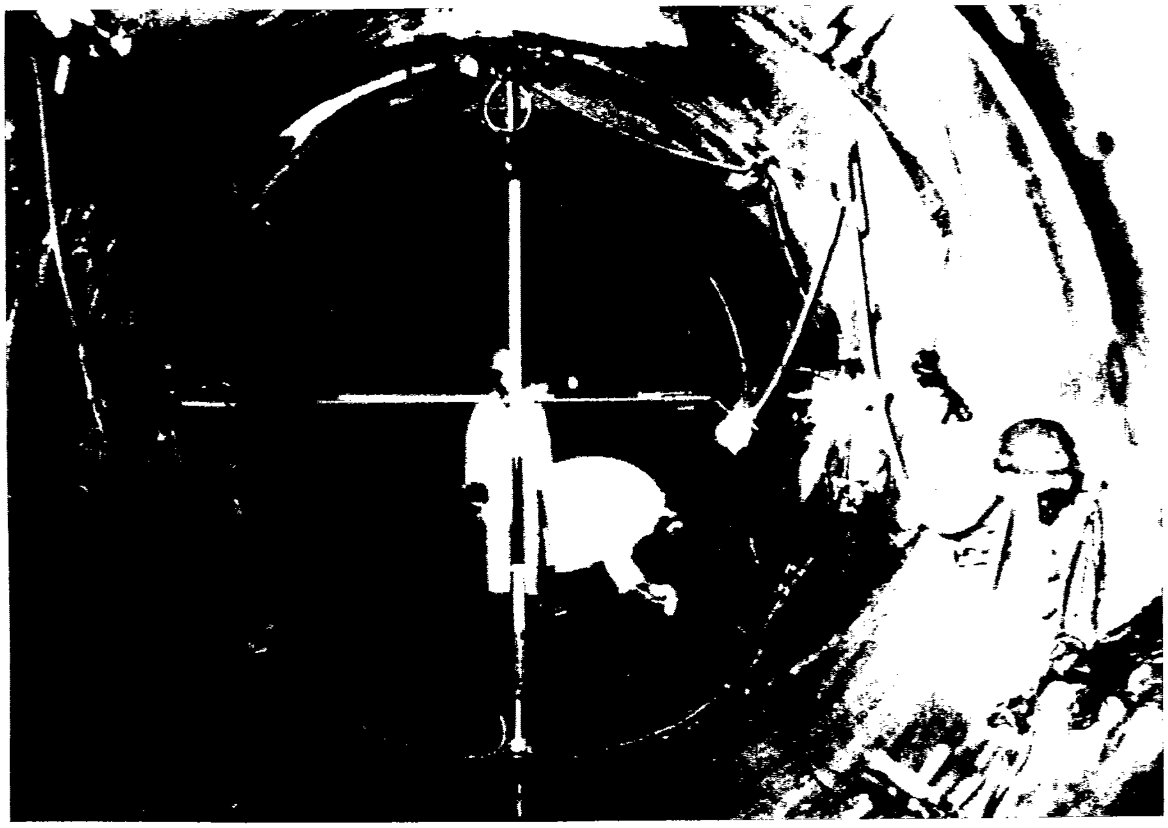


Figure G-2. Full-suite tunnel instrumentation designed to monitor response-in-service under high-levels of hydrostatic pressure. The installation consists of piezometers, crack deformation gages, convergence meters and extensometers, all designed to be read remotely at the ground surface. The tunnel is 3 m (10 ft.) in diameter. (Photography by J. R. Wheeler).

nel, installation, monitoring and *analysis of data.* The planner should answer some key questions:

- What will be the net effect of stress compensation between the geologic medium and the engineered structure?
- What basic types of instruments will be required to detect and measure this effect?
- How much sensitivity (accuracy) will be required to develop meaningful data for use in analysis and/or hazards warning?
- What is the medium (media) in which the measurements must be made?
- How long will the measurements be required?
- What type of personnel will be available to install and monitor the devices?
- What type of record is most desirable?
- What are the basic instruments that deliver the requirements?
- How will the data be analyzed?

With this in mind, the user then develops a basic set of specifications and begins to outline the kind of instrumentation (by type) that will fulfill the basic requirements. The most important single aspect of equipment type will be the perceived degree of accuracy of data required for engineering use. Reliability should generally be considered next, followed by cost.

Simplicity of design and construction of the instrumentation is important because the more simple devices should not only be less expensive, but will perhaps be more reliable in the long term (e.g. more durable against the elements and construction activity).

When the basic list of requirements and conditions has been developed, it is a good idea to call in a review consultant from within the organization or from a local firm, agency or university.

Only a specialist can maintain an on-going assessment of all of the factors and apply them to project requirements. Often the consulting specialist can save significant amounts of money in helping to prepare the equipment purchase and installation specifications.

Some typical uses for instrumentation are as follows:

- Stress/strain
- Slope stability
- Wall/face stability
- Stability/deformation of adjacent structures
- Stability of underground openings
- Subsidence and settlement monitoring
- Water level and pore/cleft water pressure determination.

Instrumentation can begin to be effective when soil or rock masses are opened and exposed by site grading or excavation, just before any support systems are put in place. Instrumentation is often used to establish a baseline of conditions existing prior to construction and then to relate these data later to the stability or functionality. Therefore, it is best to consider the need to establish conditions as they existed prior to construction. An example of this is the need to determine the preconstruction state of buildings or other structures adjacent to the project; those which owners or occupants may later perceive to have been affected by construction.

Most instruments are made to collect data; the personnel who are chosen to work with the instrument should have an appreciation for collecting accurate data. There are schemes and systems that can be used to reduce the need for the human element in monitoring the devices. At a minimum, personnel will be required to inspect the functionality of the instrumentation, especially if the device is designed to provide continuous or frequency-interval records by mechanical, electrical or electronic means. A consideration in selection of personnel is also paramount in terms of installation of the instrumentation. Many suppliers express interest in installation and often request to visit the site and train the installation team to set the first few devices.

The choice of location for each device is extremely important in meeting the objectives of instrumentation. Geologic conditions almost always influence the need for the instruments *and* such conditions usually control the functionality of the devices as placed. Some instrumentation is designed to monitor the physical behavior and stress state of relatively large volumes of soil and rock which are not directly attached to or adjacent to the project structure. The degree of geologic influence is less sensitive in such cases. But, when instrumentation is installed in rock or soil bodies with geologic contacts, or discontinuities, the effect of geology is greater than any other factor of potential concern. Each instrumentation position should be individually selected first on the basis of the overall geometry of immediate geologic conditions and the engineered structure. Secondly, the location should be carefully inspected for its immediate surroundings and geologic conditions when the location is to be finalized. A final inspection should be held at the time of installation. Rock structure is of particular concern for devices secured di-

Table G-1
Purposes for Employing Instrumentation on Transportation Projects

Purpose	Typical Application
Detection: to magnify user's sensitivity or awareness of physical changes taking place in earth/rock masses or in structural components of engineered facilities	• Increase the observer's level of detection sensitivity • Create a continuous non-attended monitoring system • Quantify otherwise human-nonquantifiable effects • Detect adverse effects of a known nature
Diagnosis: to provide data that describe the nature of detected phenomena; to establish trends and magnitudes of dynamic changes	• Establish an absolute record of a phenomenon • Compare possible mutually-dependent or singly-dependent phenomena • Observe the effect of time, temperature, and other independent variables
Prediction: to establish the basis for continued changes as they affect performance of a structure or the activities related to construction; to create a basis for warning systems	• Comparison of trends with events of consequence to the project • Link the instrumentation to an alarm-raising system
Substantiation/Verification: to create the data bases supporting decision making; to compile a record of effects that will support legal claims or defense	• Provide baselines for use in changed-conditions claims (plaintiff or defendent) • Provide rationale for implementation of remedial or strengthening measures • Establish adequacy of design • Verify suitability of techniques • Verify contractor performance
Research: to assist in explanation of important phenomena requring an empirical approach in analysis or sufficient data in order to model according to existing or developing theory	• Support analysis of variables and mechanisms/phenomena/theories

rectly in or on rock masses. At a minimum, rock cores from the exact location should be examined for discontinuities, their structural attitudes (strike and dip) and the character of their open faces (roughness, planarity, continuity, etc.) Some workers prefer to use simple devices such as the "Rochester seam criterion"; a bent coat-hanger wire used to probe for open fractures in boreholes (personal communication with Prof. Fred Kulhawy, Cornell University, January, 1980). The third action is to ask the simple question of "will what I see in terms of geologic controls affect the ability of the instrumentation to function as I have designed it to?"

The monitoring program is the whole and final purpose of the installation. Once installed, the instrumentation must not be allowed to sit without attention, both in terms of its physical condition and the data which it records. The critical aspect of monitoring is analysis of the data. Data must never be allowed to accumulate without reduction and evaluation. Most data require some degree of laborious reduction and plotting. Most instrumentation experts agree that the data should be recorded in a form that is easily transferrable to computer calculation and plotting (Fig. G-3), thus reducing the human errors and the natural tendency to set the data aside for later attention. Most data reduction can now be handled by programmable calculators and many desk-top machines can plot the reduced data. For some uses, it will be necessary to employ substantially larger computers, if previously-reduced data must be stored for sequential plotting along with new data. Computer programs usually allow scale-adjustable plotting and can easily accommodate developing trends of magnitude without resort to manual replotting. Data should be plotted on single sheets for long-term appreciation of trends.

G.4 STANDARDS

The literature of geotechnical instrumentation is presently quite varied and dispersed. Standards have been developed by the International Society for Rock Mechanics (ISRM) and by the American Society for Testing and Materials (ASTM). However, most important references still consist of individual papers. Specifications for many projects must be individually developed. ISRM has developed suggested methods for reports dealing with installation and with monitoring (see References list at end of section). One reference that has been available for some years is Chapter 9 of the California DOT Materials Manual, Vol. VI (1973); entitled, *Monitoring Devices to Control Embankment Construction on Soft Foundations.*

G.5 INSTRUMENTATION SYSTEMS

Instrumentation systems can be broadly classified into six basic types (Table G-2). The types are discussed in the following sections.

Cleft water pressure (or *fissure* water pressure) is defined as hydrostatic pressure acting along discontinuities in fractured rock; whereas *pore-pressure* acts on mineral grain surfaces through saturated interstitial voids in soil or porous rock.

G.5.1 Load/Stress on Structural Members

The interaction between earth and engineered structure is often critical to maintenance of the construction process, until all structural members are connected and ground stress is equilibrated over the facility. Ground stresses are often concentrated at geometric complexities in underground openings, both at design features and from unusual overbreak during excavation. Load and pressure cells, strain meters, strain gages and flat jacks (Table G-3) are designed to be installed at key locations in the ground support system in order to measure the ground stress being imparted to the structural system as loads at points or over defined areas.

The load cell deforms with incidence of ground stress. Cell deformation is measured by strain cells bonded to the cell. The cell is laboratory-calibrated against known loads applied by a universal test machine. Load cell designs are both solid and hollow-cylinder. The hollow varieties are suitable for installation around rock bolts and tieback anchors.

A shortcoming of load cells is that they measure axial loads only and their orientation must be carefully planned to provide stress accumulation data at key locations in the ground support system. In addition to strain-gage detection system, hydraulic pressure cells, photoelastic cells and stiff-spring-loaded cells have been designed. The photoelastic cells are particularly useful for emplacement in boreholes extending outward from structural support members in underground structures. Photoelastic cells are semi-

Table G-2
Functional Instrumentation System Types

• Load/Stress on Structural Members	(G.5.1)
• Earth Pressure	(G.5.2)
• Vertical Deformation/Movement (Settlement/ Heave)	(G.5.3)
• Pore/Cleft Water Pressure*	(G.5.4)
• Lateral Deformation/Movement	(G.5.5)
• Tilt (Inclination)	(G.5.6)

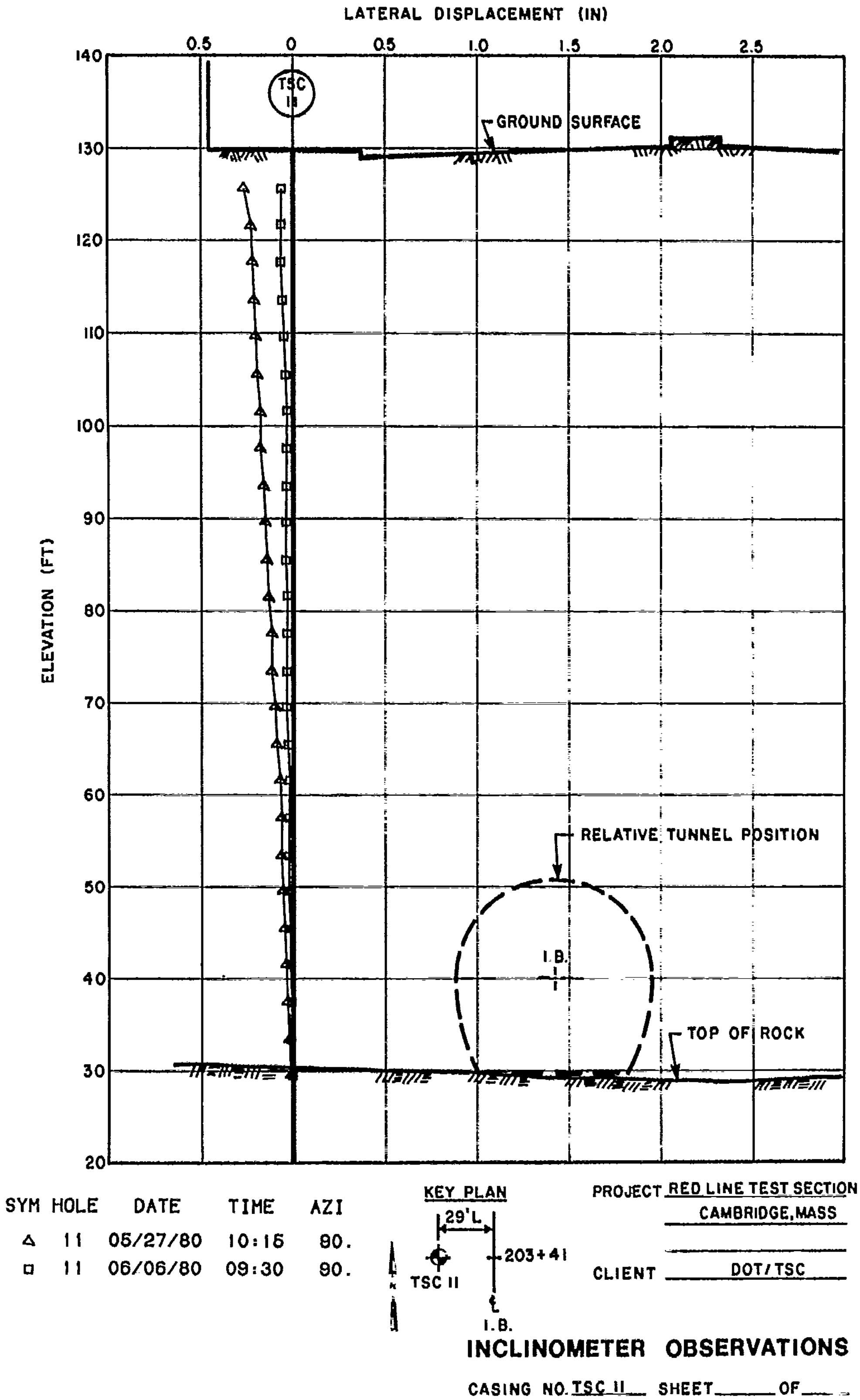

LATERAL DISPLACEMENT (IN)
0.5 0 0.5 1.0 1.5 2.0 2.5
ELEVATION (FT)
140 130 120 110 100 90 80 70 60 50 40 30 20
TSC 11
GROUND SURFACE
RELATIVE TUNNEL POSITION
I.B.
TOP OF ROCK
SYM HOLE DATE TIME AZI
△ 11 05/27/80 10:15 90.
□ 11 06/06/80 09:30 90.
KEY PLAN
29'L
203+41
TSC II
I.B.
PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS
CLIENT DOT/TSC
INCLINOMETER OBSERVATIONS
CASING NO. TSC 11 SHEET ___ OF ___

Figure G-3.

Table G-3
Instrumentation used to Sense Load or Stress in Rock or Structural Components

Instrumentation Type	Operating Principle	Transportation System Usage	Advantages	Limitations	Accuracy
Load Cells	Annular steel collar is fitted with strain or photoelastic gauges to sense one-dimensional strain	Walls; underground structure liners and load bearing components	Relatively inexpensive; can be fabricated locally in some instances	Sensitivity range must be estimated before installation	$\pm 5 \times 10^{-3}$N
Strainmeter	Strain gauge transducer is embedded in concrete wall or liner	Walls; underground structure liners and load bearing components	Moderate expense; long-lived	Sensivity range must be estimated before installation	$\pm 10^{-3}$ strain units; $\pm 3.5 \times 10^{-3}$ N/m^2
Vibrating Wire Strain Gauge	Embedded in concrete to sense axial strain; frequency of resonance is proportional to strain	Walls; underground structure liners and load bearing components		Sensitivity range must be estimated before installation	$\pm 5 \times 10^{-4}$ strain units
Flatjack	Cell is made of two plates and thin film of mercury transmitting pressure in surrounding concrete	Walls; underground structure liners and load bearing components	Moderate expense; long-lived	Sensitivity range must be estimated before installation	$\pm 3.5 \times 10^{-3}$ N/m^2
Photoelastic Stressmeter	Pattern of superimposed stress is viewed through analyzing and polarizing filters; qualitative sense only	Walls; underground structure liners and load bearing components	Relatively inexpensive	Sensitivity range must be estimated before installation	$\pm 3.5 \times 10^{-4}$ N/m^2

quantitative and are read optically; the number of stress-related deformation fringes are counted and compared to calibrated counts determined under laboratory conditions for each sensor. Accuracies of about ± five percent are to be expected, when the observer is well trained.

A critical part of load cell design is determination of the probable range of incident stress. The user must then decide to what level of sensitivity the device should be capable of detecting incremental stress changes. The cells are usually required to be sensitive at least to a range of 50 to 100 parts of total expected strain.

G.5.2 Earth Pressure

The most difficult of all instrumentation assignments are those dealing with determination of the state of stress in otherwise disturbed soil or rock masses. The problem is complicated by the general requirement of some sort of disturbance associated with emplacing the device meant to sense stress at the subject point. Requirements for sensing the stress present in soil and rock masses are vastly different; rock probably being less difficult by virtue of the fact that fresh, massive rock (unjointed) is essentially non-particulate and cannot dissipate stress to the degree that disturbance in a particulate soil mass tends to spread remaining stress to an equilibrium condition.

Knowledge of the state of stress existing in a mass of engineered fill (such as a dam or embankment) or in the natural ground surrounding a tunnel, or beneath certain critical foundations in which there is concern for soil-structure interaction characteristics. For virgin ground, it is obvious that disturbance is associated with emplacement of the measured device. For engineered fill bodies, emplacement of the instrumentation is not of so great a concern due to the fact that the stress field grows and comes to a state of equilibrium during the construction process.

Earth pressure cells of several varieties (Figs. G-4; G-5) are sold for adaptation to these stress-sensing problems:

- pneumatic
- hydraulic
- vibrating wire strain gauge
- semi-conductor, pressure transducer
- bonded-resistance strain gauge
- unbonded-resistance strain gauge

Earth and rock masses expected to accommodate stressing from engineered facilities are often tested in order to detect their deformation range in terms of elastic moduli. Table G-4 lists some of the instrumentational techniques for achieving these determinations.

Rock stress measurements are generally undertaken by the *strain-relief overcoring* method, developed by the U.S. Bureau of Mines in the late 1950's. Most of the details of the technique are found in ASTM STP 429 (1966). As shown in Figure G-6, a compressed-air drilling machine, pedestal-mounted, is employed opposite the face of rock to be investigated. A small-diameter (usually EX size, 38.1 mm) pilot hole is drilled to the depth of the first of a series of strain-relief measurements, set far enough into the rock face to avoid general strain relief from joint-bounded blocks that are smaller than the smallest dimension of the face. As the pilot bore is completed, a 15.24 cm overcoring barrel begins to remove a 14.6 cm annulus surrounding the pilot bore. At a distance outward of the tip of the pilot bore, amounting to one annular core barrel length, a multipositional strain gauge sonde (bore hole deformation gauge with three recessed, lever-type strain meters, mounted at 120-degree radial spacing around the axis) is inserted into position in the pilot bore. As the core barrel begins to overcore the pilot core and contained sonde, electrical resistance readings are made of each of the multipositional strain gauges. The barrel is left to cut far enough (a few cm at the most) beyond the tip of the sonde to result in a complete stress relaxation. In order for strain-relief measurements to be considered successful, an overcore of at least 30 cm should extend from the tip of the pilot bore toward the open face. This will insure that effects from rock discontinuities are at a minimum. The three strain gauges give implied stress variations that can be used to make up a stress elipsoid in the place perpendicular to the axis of the overcoring test. This, of course, must be repeated along other axes in order to approximate an overall three-dimensional stress field that is present in the rock mass.

G.5.3 Vertical Deformation

Embankment constructed over the finer-grained soils (silt, clay and organic materials) are subject to considerable amounts of vertical deformation where such materials are water saturated. The phenomenon is known as settlement and results from the volumetric shrinkage during the process of consolidation, as foundation live and dead loads cause pore water to be expelled or drained internally from foundation soil mass. The main mitigation technique is to spread the foundation loads so that settlement over the period of concern becomes tolerable to structural members or to the elevation of the roadbed itself. Theoretical analyses used to predict settlement are accurate within limits, but it may be often necessary to collect actual measurements of this vertical deformation in order to establish the rates and absolute magnitudes of settlement that are actually being experienced and which will likely continue to affect the structure (Table G-5).

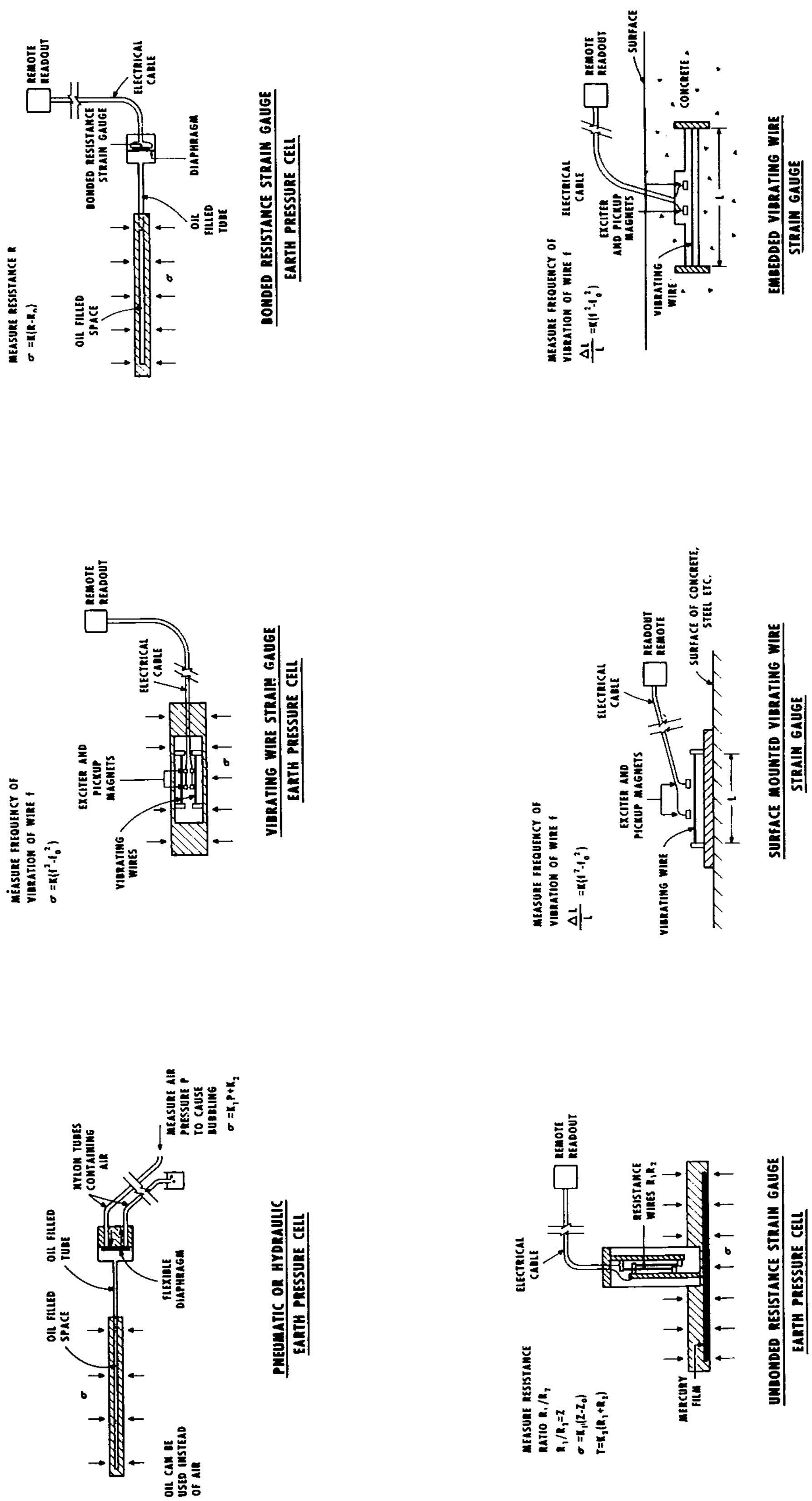
The main types of instrumentation that are used to monitor settlement of structures and embankments are (CAL DOT, 1973; Fig. G-7, Fig. G-8):

- Settlement platforms
- Heave stake lines
- Inclinometers
- Buried flexible casings
- Piezometers
- Vertical Extensometers (Fig. G-9)

In the case of embankments, settlement often occurs in association with lateral deformation related to embankment stability. As in other cases employing instrumentation, the data requirements are often competing but representing inter-related phenomena. An example of the comparative placement of settlement indicators, along with lateral deformation detection devices on a highway embankment, is shown as Figure G-10.

G.5.3.1 Settlement Indicators. Two basic types of settlement platforms are employed. The first type involves the sensing of the piezometric surface of ground water if such occurs freely within the foundation load depth of influence and the second type is employed above the saturated zone.

In the *sealed-fluid level* device (Figure G-11), water is sealed into a system represented by a rigid base plate and a protective riser pipe containing a water relief line (spill tube) and a pressure-equalizing air vent tube. As the base of the platform sinks during consolidation of the foundation soils below it, water in the spill tube is forced out and spilled into the protective casing. The sensing platform and riser are buried



SCHEMATIC ARRANGEMENTS OF VARIOUS TYPES OF EARTH PRESSURE CELLS AND STRAIN GAUGES

Figure G-4.

(Soil and Rock Instrumentation, 1972)

Figure G-5. Earth pressure cell placed for calibration in a hydrostatic pressure chamber (Photography by D. G. Gifford).

in the roadway embankment and the observation riser pipe with an equivalent water level siting port and measurement scale are generally located off-roadway at convenient points along the right-of-way.

The *vented standpipe* (Figure G-11a) device is employed at points above the piezometric level of free-standing pore water and drains its overflow directly into the embankment (Fig. G-11a). Readings representing changes in the relative elevation of the base of the device are made in an off-roadway indicating device as used for the sealed-fluid device. Accuracy of measurements depend on the choice of indicating unit standpipe (meniscus effect depending on indicator riser tube diameter) and the graduated scale by which visual observations are made. The vented-fluid level type of instrumentation must be designed and placed so that the piezometric surface of free water (ground water level) does not inundate the overflow tube as the base of the settlement platform settles.

Riser pipes are non-water-level sensitive attachments of a standpipe extending to the ground surface from a rigid base buried during construction of an embankment and at various depths. Many agencies prefer to use these settlement indicators along the

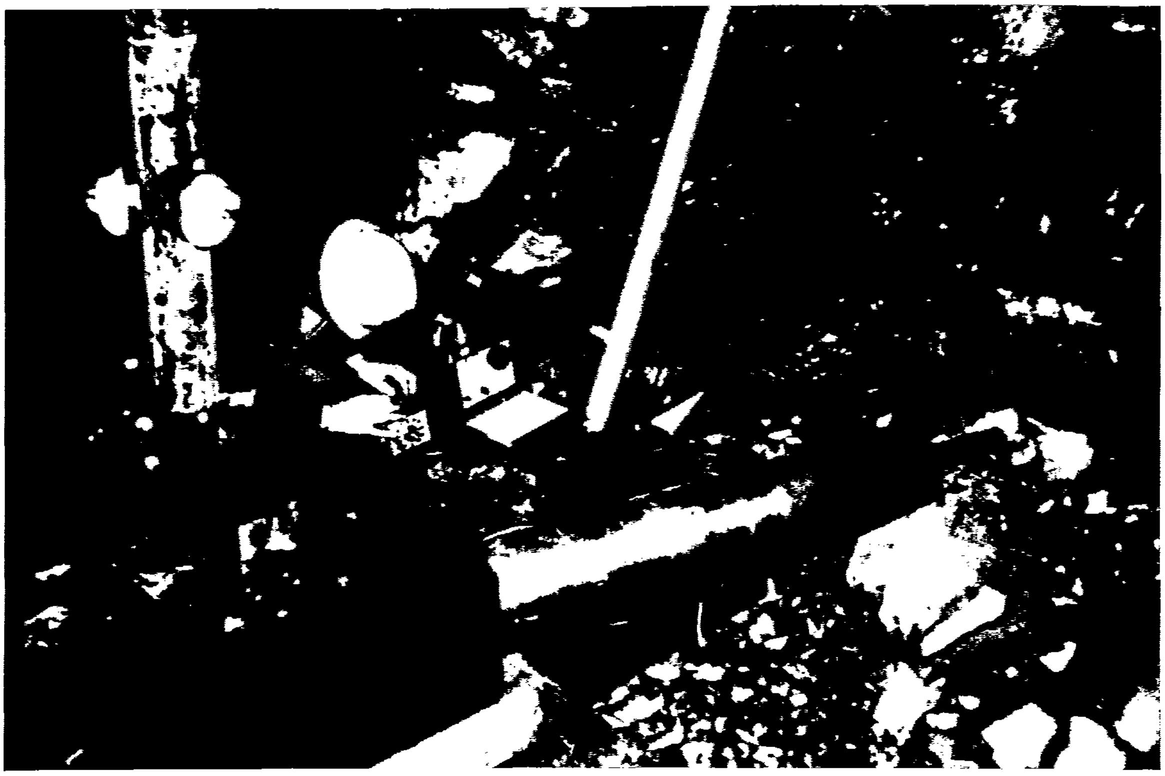


Figure G-6. Strain-relief overcoring readout equipment, vertical stanchion (stull) mount for pneumatic drilling machine and the 15-cm (6 in.) diameter overcoring barrel (foreground) (Photograph by D. G. Gifford).

centerline of divided roadways, leaving them protected by the median strip of protective barriers or to place them in protected locations at the shoulder. The top of the riser pipe becomes a survey station, and its relative elevation is representative of total settlement of the embankment below that point. Since settlement measures are total representations, a series of risers with variable baseplate elevations is required in order to determine the profile of settlement with depth. The reference benchmark must be placed far enough from the embankment not to be affected by the settlement being measured.

The U.S. Bureau of Reclamation has pioneered the use of *multi-point, vertical-tube settlement gauges* constructed of telescoping tubes anchored at intervals by the use of integral cross arms. The telescoping segments were installed as lifts of embankment construction were added and consisted usually of 38 mm cross-arm pipes set into 51 mm overall casing. The casing base is anchored to a settlement platform or earth anchor. The system is generally applicable only to constructed embankments and is accurate to determination of the location of settlement only to the intervals between the cross-arm installations.

Improvements in the Bureau's multi-point gauges have been made with the use of *compressible, accordian-fold couplings* design to shorten as the embankment settles. A mechanical torpedo sensor is lowered to measure the depth to known intervals. These telescoping mechanical settlement sensors are also used with telescoping electrical settlement sensors, in which the sensor sonde is lowered to detect metal induction coil plates at the predetermined station intervals. Both installations must be carefully backfilled with soil to an equivalent unit weight of the surrounding fill in order to escape differential settlement of backfill versus that of the surrounding embankment. The electrical variety is read by use of an impedance bridge which notes a maximum imbalance opposite the induction coil. Neither of the installations are normally fitted for use as inclinometers and, hence, do not provide measurement of lateral deformation.

Heave stakes represent the most simple and inexpensive of all slope movement indicators. The geometry of the subject slope is studied and one or more rows of survey hubs (5 × 5 × 46 cm) are placed parallel to the toe of the existing slope at one to eight metre spacing. More than one row is generally necessary and lateral deformation of each row will tend to pinpoint the location of maximum slip. The stakes

Table G-4
Instrumentation used to Sense Earth and Rock Pressure and Deformation Moduli

Instrumentation Type	Operating Principle	Transportation System Usage	Advantages	Limitations
Pressuremeter	Expandable metal/ rubber cylinder is pressure activated to deform host material of tested borehole	Tunnels and underground stations	Moduli so determined may be used in a variety of computations	Requires separate pressuremeter for each soil/rock modulus range; effective only to about 10^5 N/m^2
In Situ Deformability (modulus determination)	Borehole packer is pressure-expanded to deform surrounding rock	Tunnels and underground stations	Fast; employs available borehole; can be used to sense creep	Not applicable in massive high modulus rock
Plate jacking (modulus determination)	Walls of underground opening form reaction frame for jack-applied deformation force	Tunnels and underground stations	Models rock response in place and as a mass	Force capability of jacks; usually less than 4.5×10^6 N
Strain-relief overcoring	Angular displacement strain gauges are emplaced in sonde at end of fresh pilot bore; overcored and record change in pilot bore dimension	Relatively large underground openings at relatively deep (300 m) locations or in regions of active tectonic stress or high horizontal rock stress	Unique method for the need; some equipment can be reverse-pressured to measure modulus of deformation	Relatively expensive; one-time determination; requires estimate of Poisson's ratio of host rock as basis for calculations

are, therefore, both a supplemental means of detecting movement and defining the approximate shape of the moving mass.

Electronic measurement of settlement within embankments has been undertaken on an experimental basis by the SDDOT (Bump, 1979), using the Linearly-Variable Displacement Transducer known to rock engineers. The device, known as the *electronic extensometer* is inexpensive, accurate, and capable of remote readout. The lower end of the device is anchored in stable bedrock and to a surface plate installed at the time of placement of the last embankment lift. The upper plate in the SDDOT device is an anchor for a black-pipe housing (3.18 cm; 1.25 in.) for the LVDT, which, in turn, is connected to a 6mm (0.25-in.) brass rod, providing the connective element between the LVDT and the lower-plate anchor. The brass rod is attached to the lower plate by way of a 6mm (0.25-in.) soft-iron inducing plug (rod) that is attached to the lower anchor plate. Settlement of the embankment alters the vertical position of the soft-iron plug and produces a directly-proportional response in the electromagnetic field induced in the LVTD.

G.5.4 Pore/Cleft Water Pressure

Piezometers are water-level measurement devices generally employed in the finer-grained soils, in which the coefficient of permeability is sufficiently low as to preclude rapid sensing of the level of free water in the soil. The technique was introduced to the United States by Karl Terzaghi (1938). Piezometers measure this water level at the point of the sensor which transmits its measurement by fluid pressure or by an electronic signal. The system must be designed so that its sensors are able to detect, by positioning, the water level (piezometric surface) in the embankment or foundation soil as well as the pore pressure (u) at key locations. Pore pressure is the main variable affecting shear strength of foundation soils and the system can be used to monitor conditions under which the embankment was designed with adequate margins of safety.

The *open system* of piezometers employs a porous stone water pressure equalizing device at the depth of concern. The porous stone is constructed so as to be annular and to house a moisture indicating device, or the stone is placed as a part of the housing containing

Table G-5
Instrumentation used to Detect Vertical Deformation/Swell/Heave

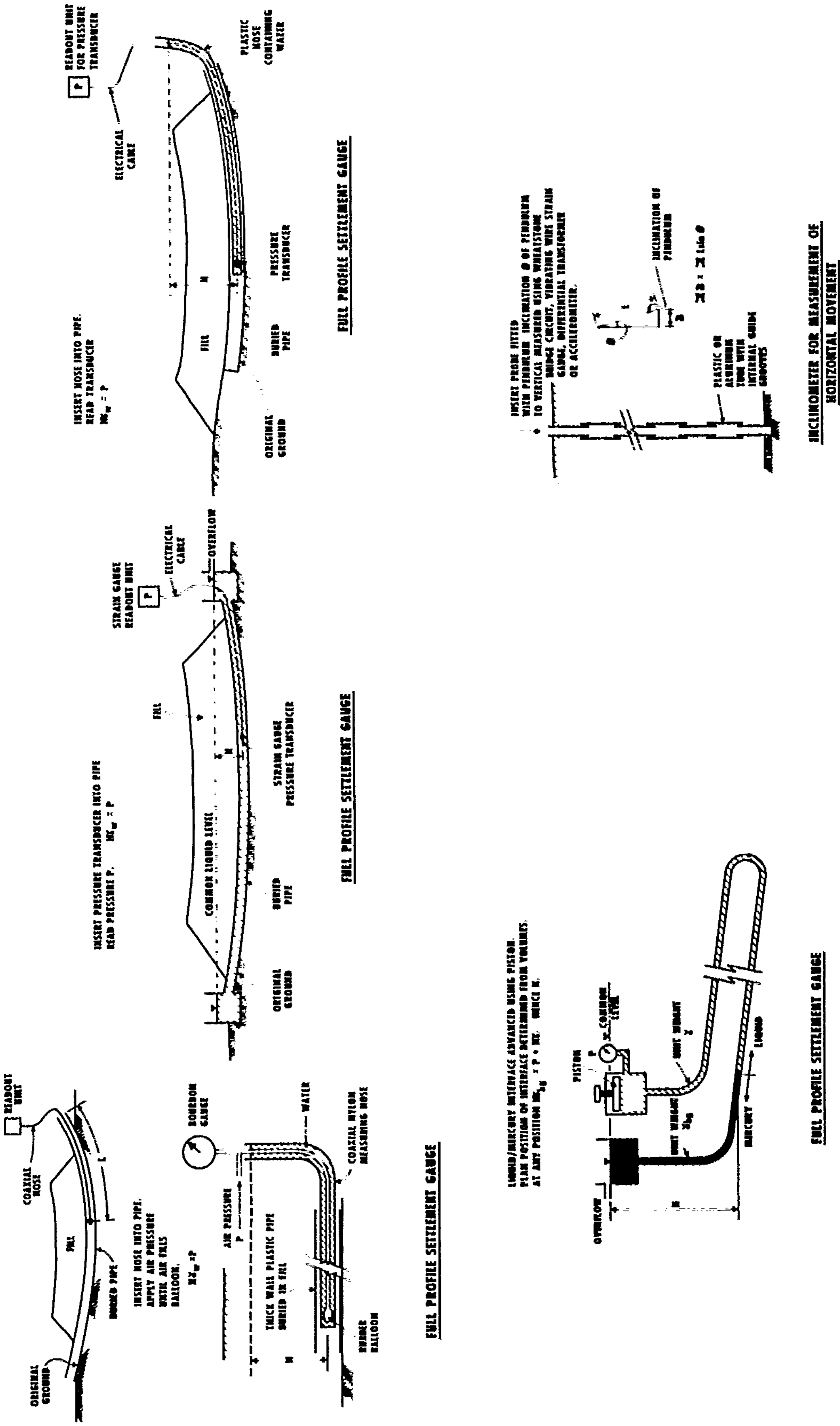
Instrumentation Type	Operating Principle	Transportation System Usage	Advantages	Limitations	Accuracy
Mechanically-sensed, telescoping tube	Improvement of USBR cross-arm device; telescoping tube segments are dragged downward as embankment settles	Embankments	Discriminates settlement to distinct telescopic intervals of embankment	Requires several cross arm intervals; best is at less than 3 m; gives no indication of tilt	±0.1 cm
Electrically-sensed telescoping tube	Sonde senses electrical field impedance imbalance at known induction coil depth locations; coils move with surrounding soil settlement	Embankments	Relatively inexpensive to install numerous measurement intervals; rapid readout		$\pm 0.5 \times 10^{-1}$ -1.0 cm
Piezometers; open and closed-system	Fluid level corresponds to change in elevation of base plate, acting as riser pipe anchor	Embankments	Rugged, reliable; can be multipoint	Requires external benchmark survey control; air bubbles can obstruct pressure transmission; somewhat sensitive to temperature differentials	$\pm 5 \times 10^{-2}$ to 5×10^{-3} cm
Electrical reed switch (U.K.) bldg. research establishment	Switch forced to close when settlement of casing and surrounding annular magnet are in proximity	Embankments	Offers multipoint discrimination	Requires multiple points to discriminate displacement	$\pm 2 \times 10^{-1 \mathrm{cm}}$

the moisture indicating device. The signal in these types of piezometers is electronic and is detected by calibrated ammeter readings.

The *closed system* of piezometers is generally employed in soils of relatively low coefficients of permeability, in which detection of the hydrostatic pressure of free pore water is difficult because of the reduced interconnectivity of the soil pore spaces. The point of interest is sealed from overlying soil strata compacted lifts, by use of less-permeable bentonitic clay slurries or poured concrete. The water tube often employs a sealed system of gas pressure and a gas pressure gauge to sense the fluctuations in pore pressure in the soil surrounding the tip and forcing changes in the observation tube water level. An observation well is generally installed in the same casing or in a nearby casing. With the closed system, it is possible to separate pore pressure from hydrostatic pressures due only to the presence of free ground water in the embankment or structural foundation.

Although most piezometer and observation well systems are visually recorded, it is possible to record the hydrostatic and piezometric components at predetermined frequencies by magnetic or digital tape or plotted graphically.

Diaphragm-type piezometers are the most expensive of the groundwater pressure detection devices, the most sensitive to changes, and the most responsive in terms of time lag in fine-grained soils and thinly-fractured rock. A housing isolated, by sealing, into the soil/rock stratum or other volume of rock senses pressure by way of a flexible diaphragm that is monitored by an internal fluid-actuated or electrical transducer (Figure G-12). The sensitivity of the system is governed by the characteristics of the diaphragm. Repeated measurements at each station,



SCHEMATIC ARRANGEMENTS OF VARIOUS TYPES OF SETTLEMENT AND HORIZONTAL MOVEMENT GAUGES

Figure G-7.

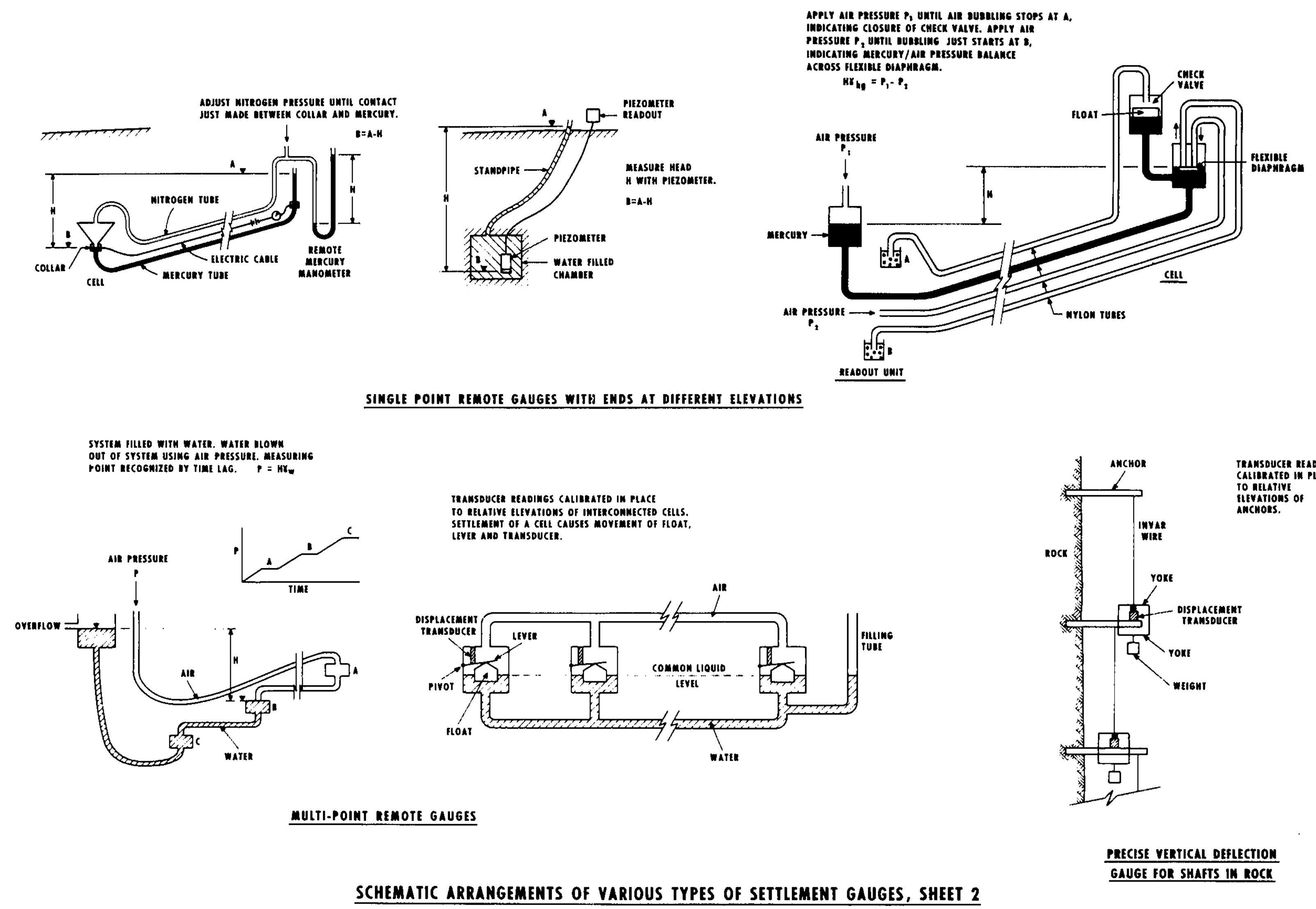


Figure G-8. Highway Focus, 1972 (USDOT)

(Soil and Rock Instrumentation, 1972)

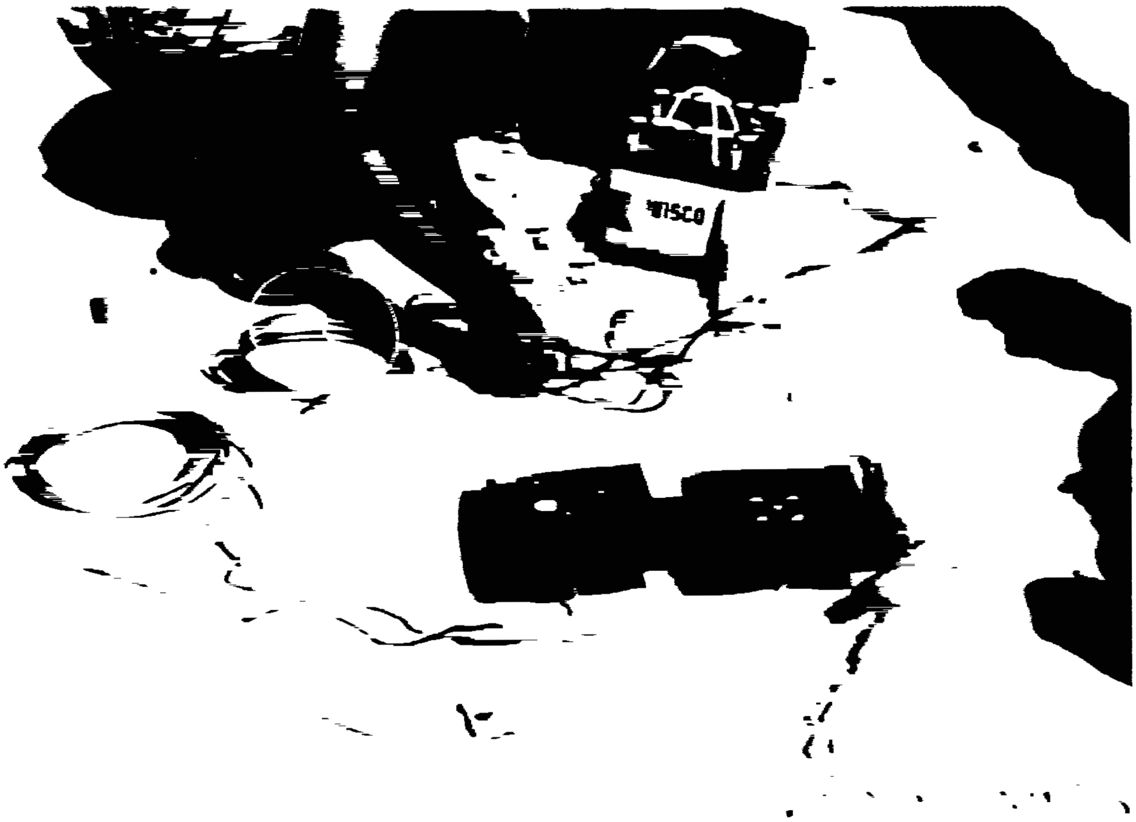


Figure G-9. Single-point vertical extensometer placed to measure foundation settlement (Photograph by S. T. Parkhill, Haley and Aldmch, Inc.)

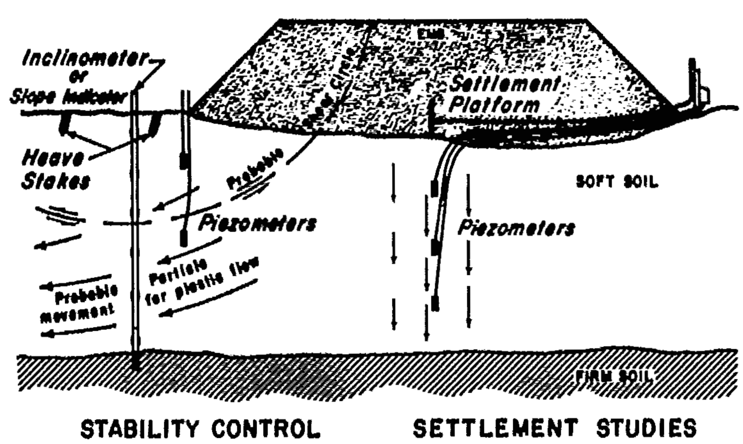


Figure G-10. A composite illustration of the use of various stability control and settlement detection devices in a typical, thick highway roadway embankment. Installations such as these are kept under surveillance, for such time as design engineers believe the embankment conditions may lead to roadway damage (From California Dept. of Transportation, 1973).

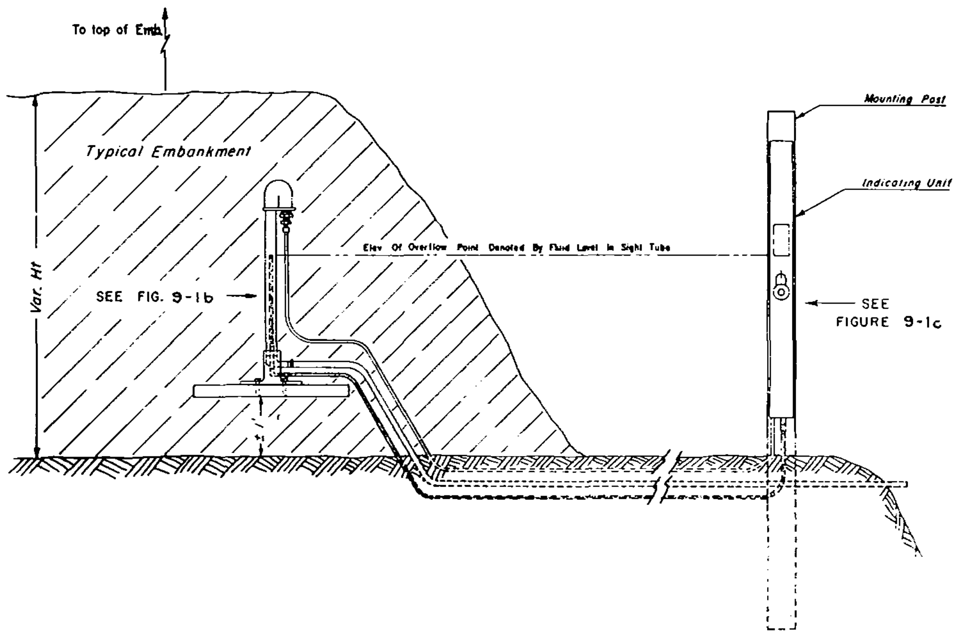


Figure G-11. **A sealed fluid level settlement detection device, as embedded in a typical roadway embankment and connected to a freestanding-manometer-type indicating device. The device provides critical information to geotechnical engineers monitoring settlement and stability behavior of high embankments, sometimes placed over compressible subsurface soil units (From California Dept. of Transportation, 1973).**

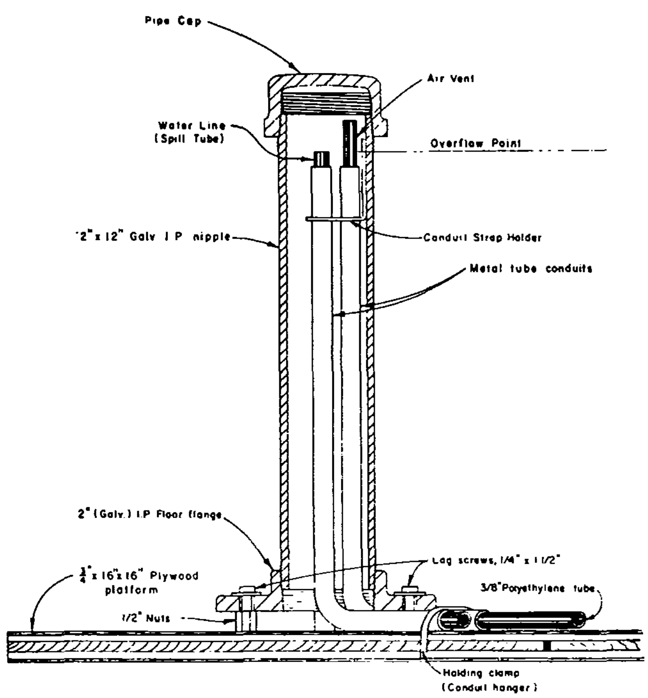
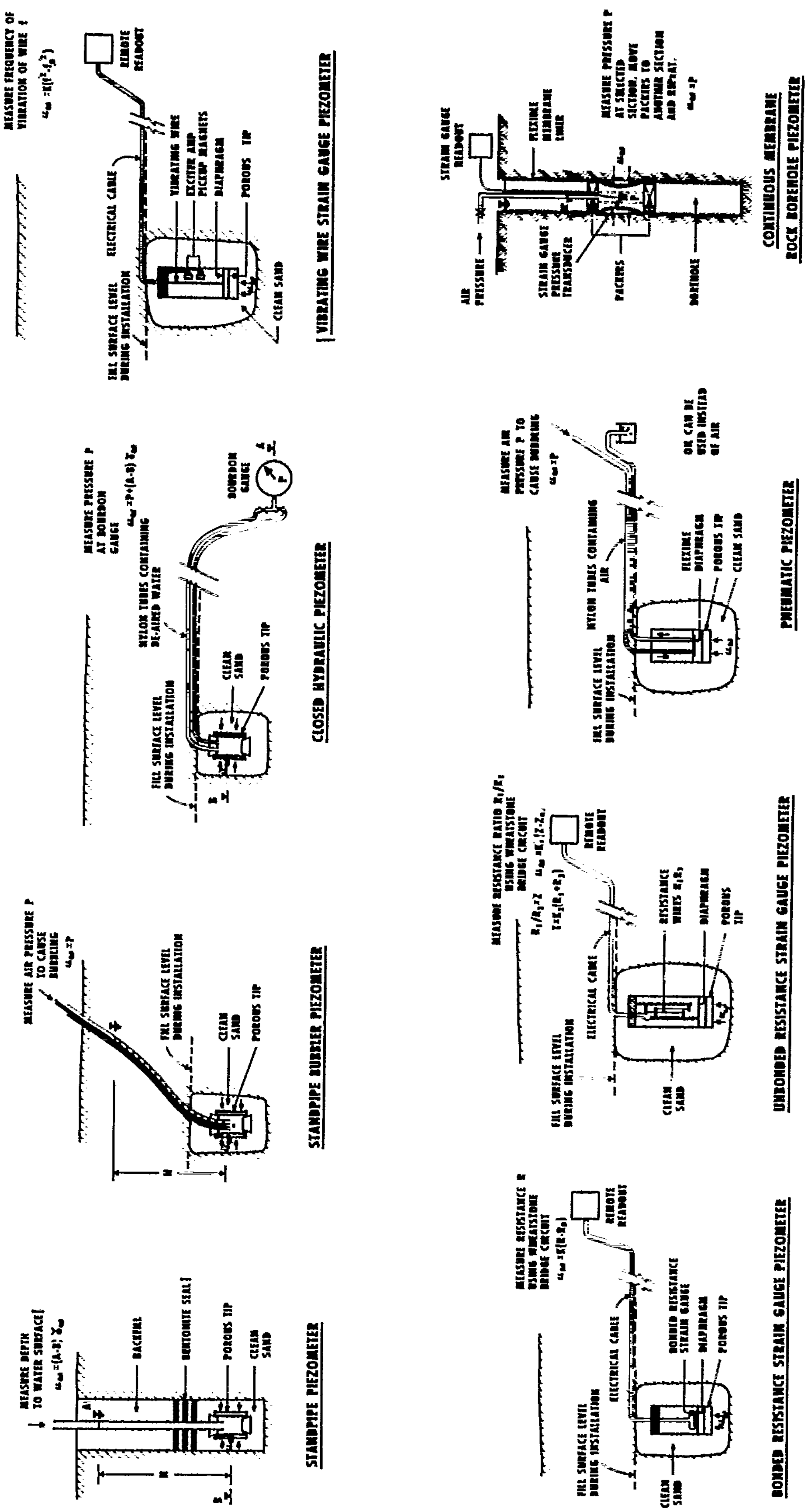


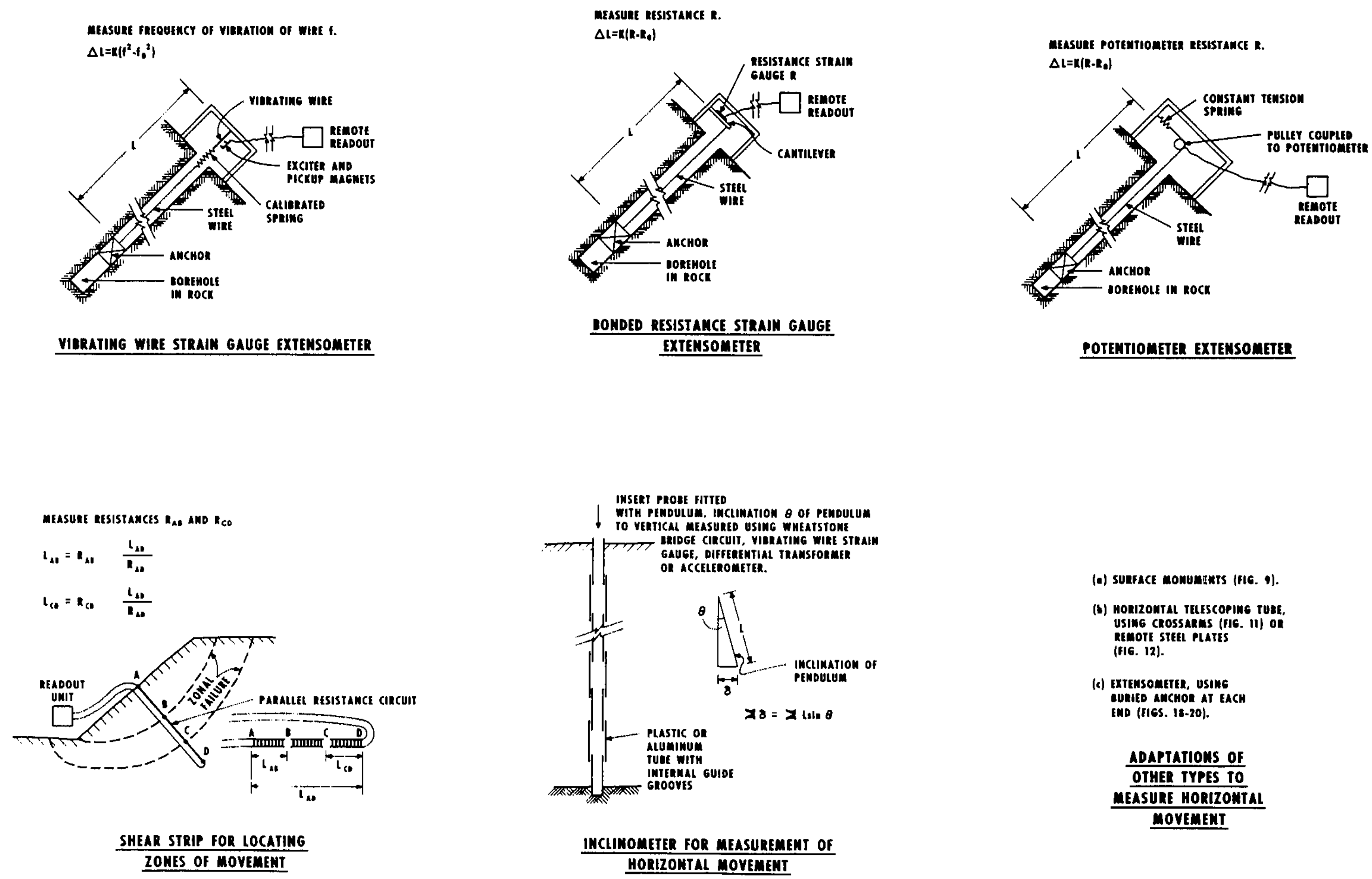
Figure G-11a. **A vented fluid level standpipe device used to detect embankment settlement. Units such as this are buried at various depths within the embankment and are hydraulically connected to an external, off-embankment, manometer-type, visual indicator device. Water contained within the system provides an indication of the relative change in the base elevation of the base platform as water is spilled out of the system at the overflow point, as the base settles (From California Dept. of Transportation, 1973).**



SCHEMATIC ARRANGEMENTS OF VARIOUS TYPES OF PIEZOMETERS

Figure G-12. Highway Focus, 1972 (USDOT)

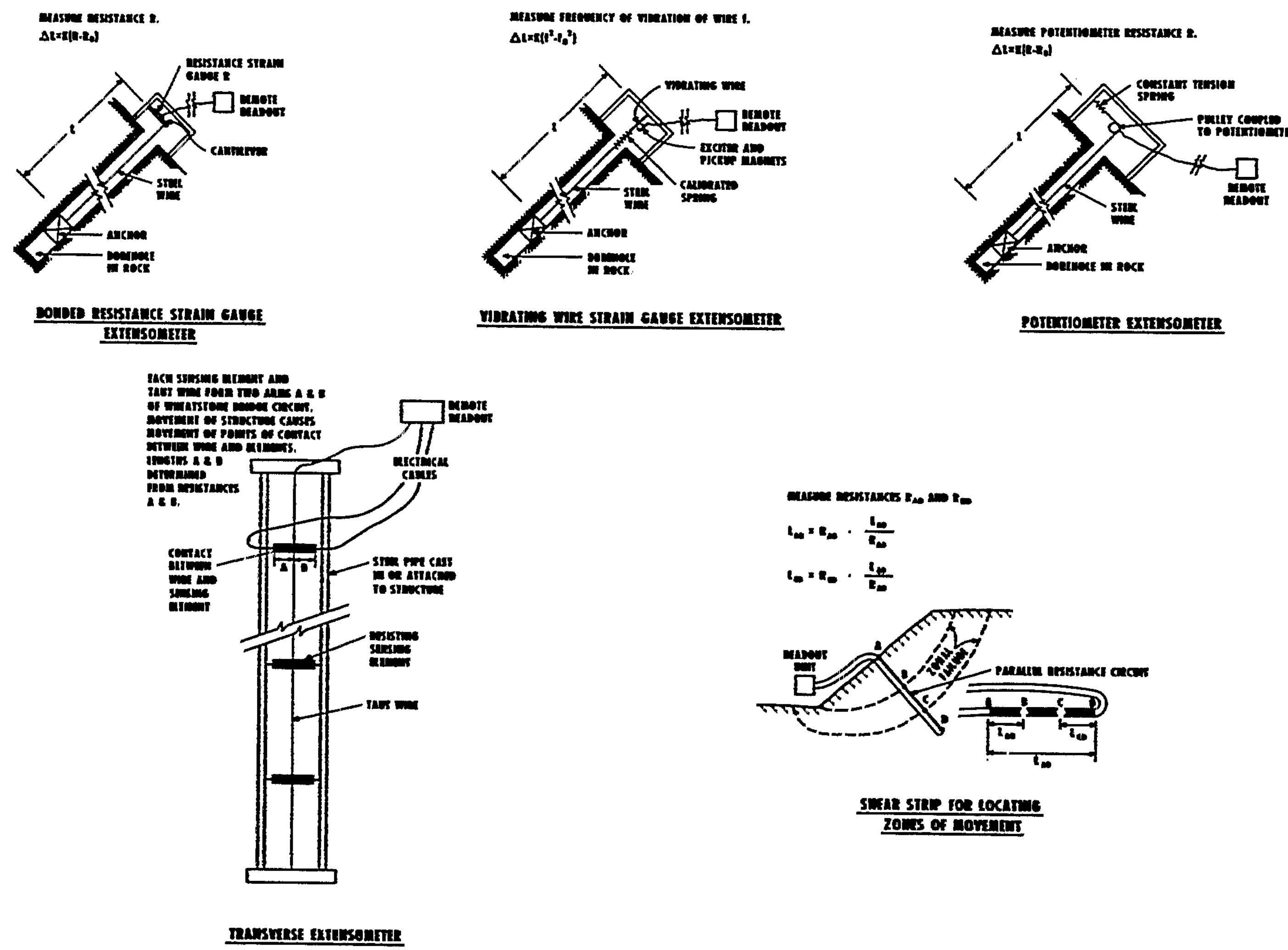
(Soil and Rock Instrumentation, 1972)



SCHEMATIC ARRANGEMENTS OF VARIOUS TYPES OF INCLINED AND HORIZONTAL MOVEMENT GAUGES

Figure G-13. Highway Focus 1972 (USDOT)

(Soil and Rock Instrumentation, 1972)



SCHEMATIC ARRANGEMENTS OF VARIOUS TYPES OF INCLINED AND HORIZONTAL MOVEMENT GAUGES

Figure G-14. Highway Focus 1972 (USDOT)

(Soil and Rock Instrumentation, 1972)

Table G-6
Instrumentation used to Detect Horizontal or Relative Deformation

Instrumentation Type	Operating Principle	Transportation System Usage	Advantages	Limitations	Accuracy
Tape Extensometer	Visual measurement between two opposing, exposed points	Tunnels and stations	Simple, inexpensive; numerous stations possible	Difficult to reproduce exact tape tension on measurement	$\pm 1 \times 10^{-3}$ cm
Rod Extensometer	Measures distance between converging stations on walls of underground opening	Tunnels and Stations	Simple; cheap installation costs; durable, functional to length of up to 200 m; multiple sensing stations	Temperature correction necessary; entirely manual operation	$\pm 2 \times 10^{-2}$ cm
Multiple-Position Borehole Extensometer (MPBX)	Measures incremental deformation along instrument axis; at specified stations; anchored at base of borehole	Tunnels, stations, and large cut faces in jointed rock	Reliable, long-term, incremental measurements; correct for temperature effect by dummy gauge	Expensive	$\pm 10^{-2} - 10^{-3}$ cm
Telescoping Tube (Borehole)	Torpedo sensor is moved to measure distance to buried plate at interior tip of instrument	Embankments Cut slopes Vertical excavations Tunnels and underground openings	Continuous deformation record along axis of measurement	Complicated installation; expensive	$\pm 5 \times 10^{-2}$ cm
Tensioned Wire (Borehole)	Wires attached to anchor plates along deformation-telescoping tubes		Only moderately complicated	Senses incremental deformation	± 1 cm
Transverse Extensometer (Borehole)	Tensioned wire anchored to base of boring; surrounding steel casing contains multiple resistance elements which move with deformation	Retaining structures	Readout by Wheatstone Bridge	Senses incremental deformation	$\pm 1 \times 10^{-2}$ cm
Vibrating Wire Strain Gauge (Borehole)	Measures strain of steel rods acting as retention support	Retaining structures	High sensitivity; long-term durability	Expensive; requires sealed housing, inert gas; high cost of electrical circuit reliability	$\pm 10^{-2}$ to 10^{-3} strain percent 2×10^{-3} to 1×10^{-2} strain percent
Simple Distance (Deformation) Gauge	Visual measurement between two opposing, exposed points	Tunnels and stations	Simple, inexpensive; numerous possible stations	Temperature influences; damage potential to exposed stations; total deformation only	$\pm 10^{-2} - 10^{-3}$ cm
Bonded Resistance Strain Gauge (Borehole)	Measures strain of steel rods acting as retention support	Retaining structures	Relatively cheap	Corrosion sensitive	$\pm 5 \times 10^{-2}$ to 10^{-1} strain percent

over a period of minutes, are generally required to insure functionality of the instrument.

All piezometers should be sealed into the rock of soil unit to be monitored. This is usually accomplished by carefully creating a hole for emplacement of the instrument, with sufficient annular space around the instrument for placement of a clean sand or fine gravel filter. The volume of rock above the filter is then isolated, usually with expanding clay pellets and may then be further sealed with placement of a plug of concrete or grout to prevent pore water from entering or leaving the filter body from other than the volume of soil or rock to be monitored.

G.5.5 Lateral Deformation/Movement

Embankment settlement is often accompanied by lateral deformation of the mass of engineered fill, as are many natural and cut slopes. Lateral deformation should be detected, recorded, analyzed and compared with stability computations for each slope, constructed or cut. When combined with piezometric measurements of pore pressures definite indications of approaching instability (limit equilibrium or Safety Factor approaching zero). Magnitudes and rates of deformation are the most important of these measurements, as well as seeking to gain volumetric coverage of the moving mass. A variety of sensing devices are available, their selection and employment is often complicated by the expense of coverage of significant volumes of cut or fill earth or rock (Table G-6).

Concern over the presence and magnitude of horizontal movements is usually expressed in vertical faces such as concrete retaining walls, sheet pile bulkheads, tieback anchored, vertical cuts (Fig. G-15), and high faces in underground structures constructed in jointed rock. For sheet pile bulkheads, the object is to verify the nature of the moment diagram and resulting elastic deformation (Fig. G-16); for concrete walls and slurry trench installations, the object is to establish an accurate vertical profile before overturning forces are released; for vertical cuts and faces, to detect and rate-monitor movement into the adjacent opening.

Most lateral deformation measurements are made to verify the stability of the face/wall or its internal or external support system reinforcing. In the case of some reinforcing systems, such as rock bolts, deformation measurements may be made to establish the need for tensioning required to bring the face back

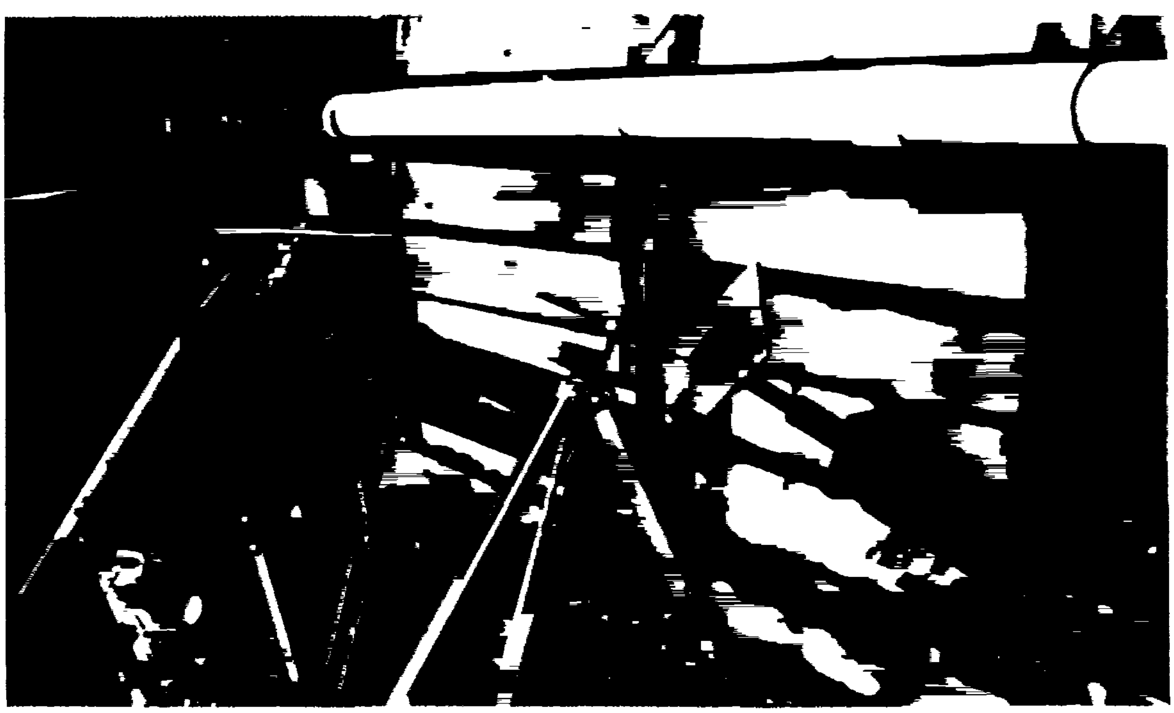


Figure G-15. Proof testing of angled and belled rod-type tieback anchors before final lockoff of the tensile stress placed on the anchors. Measurements include vertical extension of the rod (as determined at the tripods), tensile stress as applied by the rod-mounted hydraulic jacks, and time. Tieback anchors are generally placed in successive rows; this row represents the first two to be installed as the excavation opposing the lagged wall is deepened. (Photograph by J. R. Wheeler).

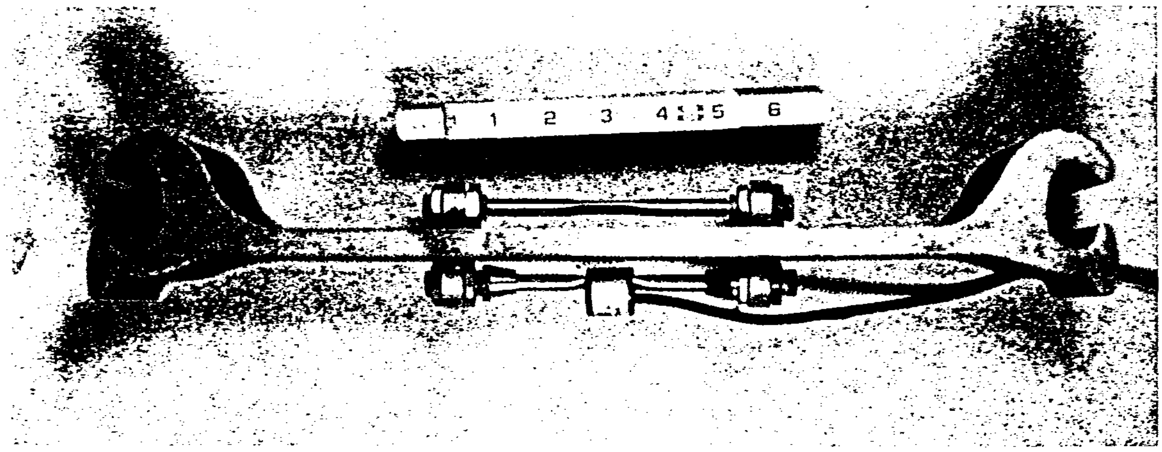


Figure G-16. Vibrating wire strain gages applied to PSX32 sheet pile section; used to monitor deformation of the sheet piles during changes in hydrostatic pressure on one side of a drydock wall. A bending moment diagram can be accurately constructed from data collected from sets of dual-position strain gages placed at varying elevations along the piles, before driving (Photograph by M. X. Haley).

into the desired state of shear strength activation. Separately-identified masses of joint-bounded rock can be monitored by horizontal movement-detecting devices, often remotely read, such as shown in Figure G-17.

High-speed rail transport requires rail embankment and bedding design capable of flexible response within well-defined limits of lateral deformation. Small horizontal components of rail tie push are measured by reaction-beam and dial gage and empirical relationships used to refine theoretical design relationships (Fig. G-18).

Figure G-17. Permanent instrumentation installations placed to monitor displacements of rock masses at large cut slopes can be collected and circuit-carried to readout boxes placed at accessible locations. The electrically-based readings are made by potted-plug connections, as shown here in an installation monitored by the New Hampshire Department of Public Works and Highways, at Woodstock, NH (Photograph by A. W. Hatheway).

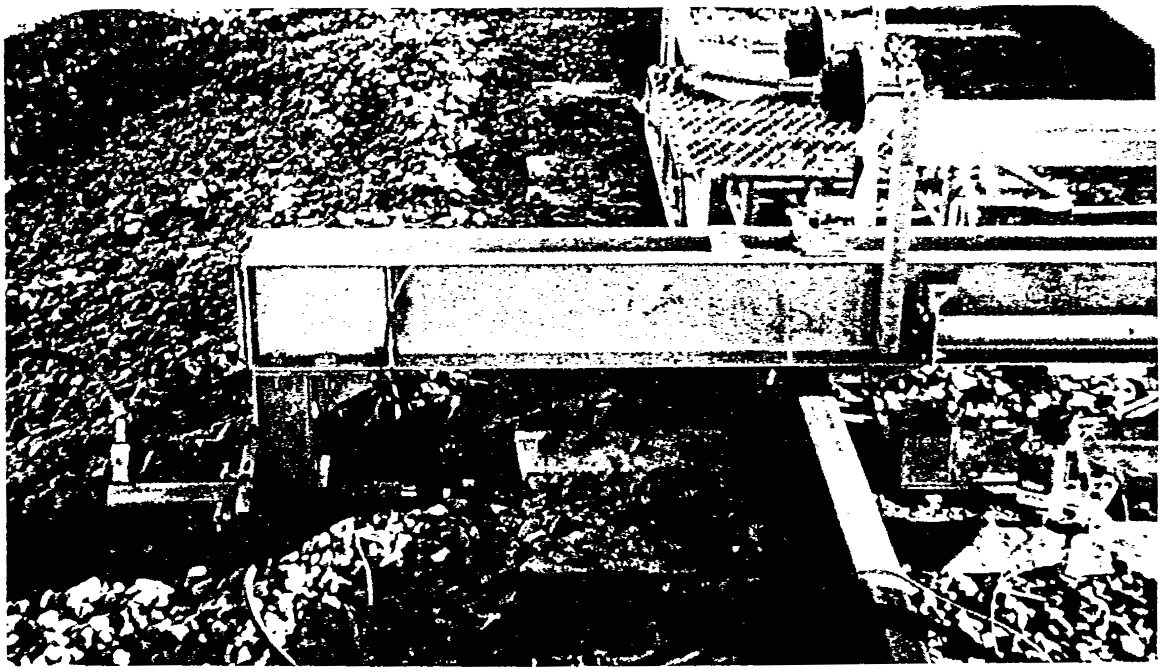


Figure G-18. Reaction beam and dial gage designed to measure one-time, maximum lateral tie push on rail embankments from passing trains (Photograph by J. R. Wheeler).

G.5.5.1 Extensometers. Extensometers represent a major category of instrument types that sense the pulling apart of particles or elements within a mass of earth or rock, or the separation of structural components of an engineered facility from their surrounding host rock or soil. Extensometers may be made of telescoping invar steel rods and placed between exposed measurement stations (e.g., the walls of a tunnel; Figs. G-19 and G-20) or may be made up of wires or strain-gauge-bonded steel rods anchored at depths beyond the zone of actively moving or deforming rock or soil. In any event, the goal is to create a means of sensing the total or incremental movement of a wall or mass of earth of rock into or toward an open face,

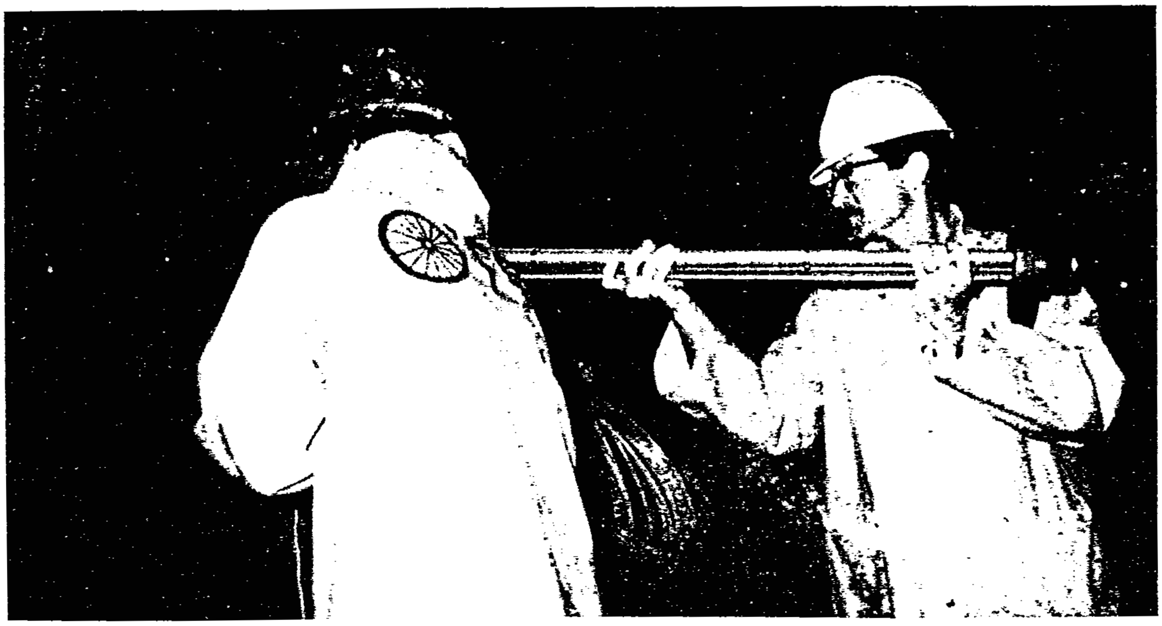


Figure G-19. Installation of a multiple-position borehole extensometer (MPBX), containing five measuring stations (Photograph by Alan L. Howard).

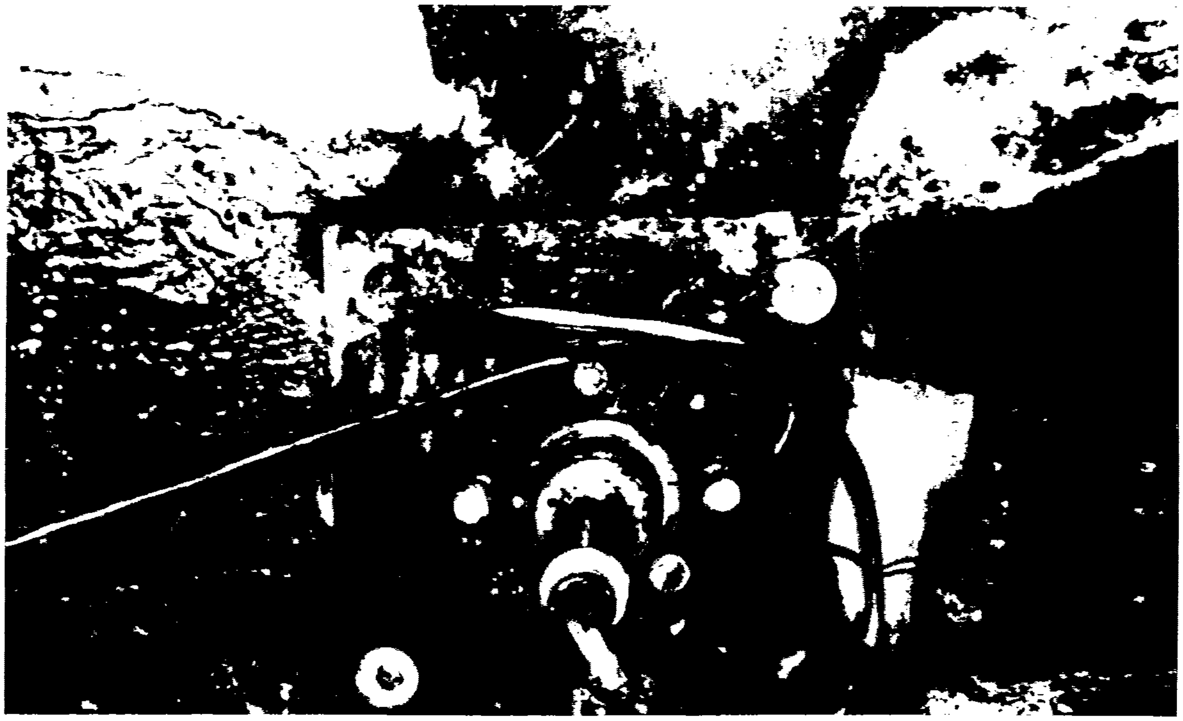


Figure G-20. A sonic-probe, multiple position borehole extensometer (MPBX) anchored to a concrete surface cast against a mass of jointed rock. The device measures radial extension or compression of the rock mass at five stations over a 5 m length. (Photography by Alan L. Howard).

excavation, or underground opening. The variety of extensometers employed can include measurements of structural support member deformation along with or separate from the deformation of the host rock or soil.

The level of accuracy needed in extensometer measurements is the basic controlling influence on the type of instrument utilized. Accuracy is mainly influenced by friction between wires, rods and used to transmit the sense of deformation and the encasing borehole, lining casing, and anchors used to hold the device at the far (interior) end of the installation. With the use of wire connectors between anchor and head, there is additional concern about the effects of corrosion, and temperature on the elastic (tensile) properties (stretch) of the wire.

Extensometers installed within soil masses provide deformation only for the vectors represented by their own axial alignments. In order to resolve the nature of three-dimensional strain fields, multiple extensometers are required, if the geometry of the instrumentation location is appropriate. If the site is a single, large face, this is understandably difficult; if the site is in a tunnel, a radial array of extensometers can be placed to give strain orientation in one or more planes. Fortunately, large cut faces are acted upon primarily by gravitational forces induced by the presence of the face itself; underground openings are commonly affected by *in situ* ground stresses which may be influenced by residual tectonic stresses or stresses created and concentrated by the act of mining or excavation associated with creation of the opening.

Extensometer readings define deformation trends with time along an axis, in a place (more than one installation) or in space (multiple installations; Fig. G-19 and G-20). The readings are essentially accurate until or unless the block or rock or support face/lining becomes separated from the mass, thus leaving the associated borehole positional gauges unanchored or leaving one end of a two-position underground opening gap distance unrecorded.

Wire-type gauges are susceptible to deformation along other than their own axes if the rock mass is jointed and movement occurs along another plane, thus displacing the axis of the wire. Rods are not as susceptible to this type of cross deformation due to their greater stiffness.

G.5.6 Tilt Indicators

Generally speaking, *inclinometers* provide the most definite means of detecting the approximate depth of the failure surface of a mobilized embankment or slope (Table G-7). Most inclinometers consist of

Table G-7
Instrumentation used to Detect Tilt

Instrumentation Type	Operating Principle	Transportation System Usage	Advantages	Limitations	Accuracy
Vertical Deflectometer	Tensioned wire anchored in vertical casing with pre-positioned knife-edged supports; sensor detects deflection from initial position at each support	Slopes, abutments, walls	Simple	Must have deformable casing to pass sense of movement to knife-edged supports	$\pm 10^{-2}$ cm
Groved inclinometer (multi-position inclinometer)	Gravitationally-activated pendulum records angular tilt from vertical, along vector established by positioning grooves in enclosing casing	Slopes, abutments, walls	Continuous record with depth; can be recorded on digital or magnetic tape for computer reduction	Requires careful installation and packing of casing	$\pm 10^{-3}$ horizontal cm in 30 m; or $\pm 10^{-4}$ radians
"Poor Man's" Tiltmeter	Deformable tubing installed in uncased borehole; series of variable-length rods lowered to point of construction; point of maximum curvature is computed	Slopes	Simple, inexpensive, rapid installation	Can detect only one (upper) deformation zone; difficult to detect rates and trends	±2 to 5 cm
Tiltmeter (single-position inclinometer)	Deformable tubing accumulates total deformation at collar	Tilt of existing structures; as affected by construction	Simple	May be affected by marine tidal forces in seacoast areas	$\pm 5 \times 10^{-1}$ cm

borehole-fitted plastic (polyvinyl chloride) or aluminum casing, installed vertically to a depth below that predicted to contain any possible slope failure surface. Casings as small as 1.9 to 2.5 cm are available and are assembled in standard lengths. As movement in the slope is activated, the verticality of the casing changes to a downslope tilt, which can be measured quite accurately to ± 0.001 cm. The first level of accuracy for inclinometers is the position or original of tilt, which is the slip surface of the mobilized mass. This is termed the "refusal position." The amount of tilt is usually measured by a sonde which is lowered into the casing and which sends an electronic signal to a digital readout (Wheatstone bridge) and is further converted manually to degrees of tilt from the vertical. The direction or vector of displacement is calculable also. Analyses make use of magnitudes of displacement versus depth, and such assessments are usually corroborated by field observations and displacements of such accessory devices as the heave stakes. An illustration of the use of inclinometers is shown in Figure G-21.

Inclinometers require specialty casings and readout equipment. One readout set will service all inclinometers on the project, probably with the desired frequency of readings. An alternate system, shown in Fig. G-22 and in routine use by South Dakota DOT, employs simple, unslotted vertical, deformable casing as installed in boreholes in the area suspected to be subject to movement. A sonde, consisting of only a metal weight in a series of about 30 cm to 60 cm (1 to 2 ft.) in length is lowered by cord or

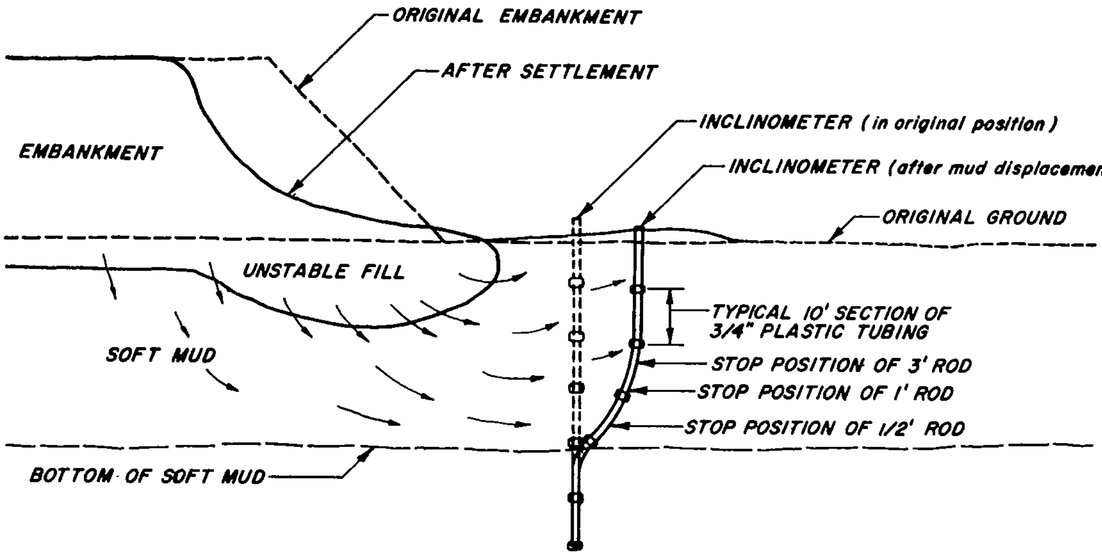


Figure G-21. An inclinometer installation placed to observe tilt associated with lateral deformation of the foundation soils underlying a roadway embankment. Inclinometer sensitivities are such that even minute movements can be closely associated with elevation and used to define the geometry and rate of deformation (From California Dept. of Transportation, 1973).

cable, and the position at which the successive lengths of sonde are restricted to further down-hole movement are recorded and plotted to determine the origin of the indicated radii of bedding representing deformation of the casing occurring at approximately the depth of the failure surface. The rod-type sondes are up to 45 cm (1.5 ft.) long by 0.95 cm (0.38 in.) in diameter, in SDDOT practice, and are suspended in 1.28 cm (0.5 in.) PVC pipe. These "poorman's" inclinometers are extremely useful at remote locations, for those projects for which inclinometers are not available at the time of need, or more simply, as an inexpensive expedient in damage-susceptable locations.

All forms of tiltmeters must be surveyed carefully as to lateral position and elevation of their exposed collars. The reference benchmark must be located outside of the presumed or identified zone of influence of the movement. Tiltmeter casings have been known to be dragged downward by the vertical frictional component of down-gradient slope movements.

Inclinometers are instruments which are generally capable of recording a vector of tilt. In order to accomplish this, most are mounted in grooved plastic or aluminum casing and the measurement sonde is dropped slowly down the casing in a one of a number of oriented combination of grooved tracks. Drift of the boring in which the casing is emplaced is important to the correct solution of vectored displacements as is the potential for twisting of the instrument casing in installations of greater depths than about 30 m. It is usually necessary to conduct a verticality and drift survey at the time of installation in order to establish the base position of the grooves and subsequent incremental measurements. Measurements can be made at any position along the grooves and are, therefore, capable of pinpointing zones of slip displacements that intersect the casing at oblique angles.

G.6 POSITIONAL SURVEYS AS INSTRUMENTATION TECHNIQUES

Traditional positional surveys are often disregarded as appropriate methods of instrumentation for deformation. However, as Wilson and Mikkelsen (1978) have pointed out, optical instrument surveys and tape measurements can be used to determine lateral and vertical movements, within certain ranges and accuracy. Table G-8 is modified from Cording, and others (1975) and shows the ways in which survey techniques can be utilized to detect deformation of earth and rock masses.

For slope movements, it is essential to set up a survey point network that extends into the affected area from stable ground. Figure G-23 illustrates a suggested scheme of survey point positions from Sowers and Royster (1978), incorporating enough basic stations to detect vectors of maximum displacement and to assist in chosing locations for supplemental survey stations. The survey net can be utilized to compile a hasty orthophotographic map of the slope,

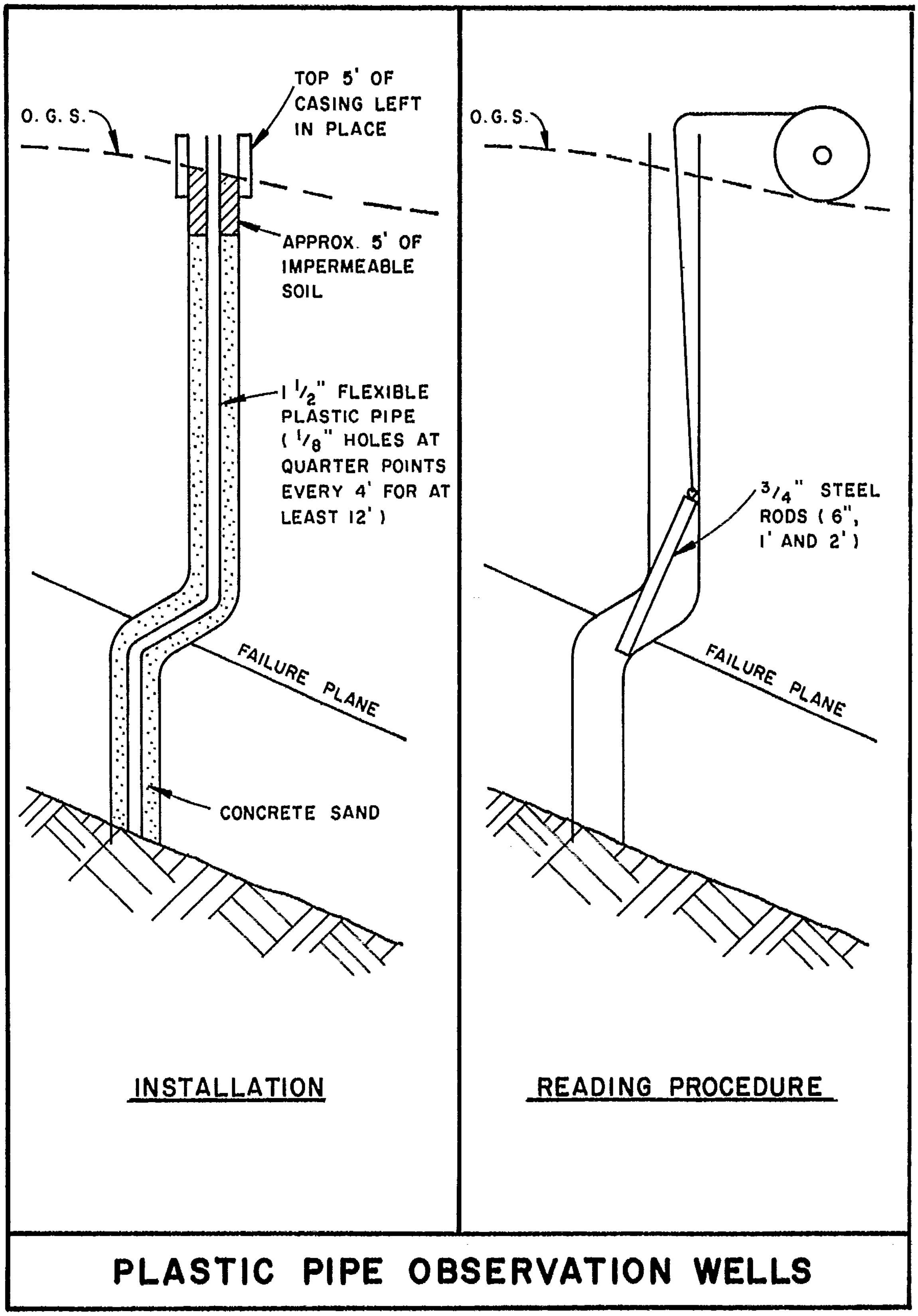


Figure G-22. South Dakota DOT

Table G-8
Positional Survey Methods Applied as Instrumentation Techniques[1]

Survey Method	Range	Accuracy	Advantages	Limitations and Precautions	Relative Reliability
Chaining					
Ordinary; 3rd order	Variable	$\pm 1/5000$ to $1/10,000$ of distance	Simple and inexpensive; direct observation	Requires line of sight between points; stable benchmarks	Excellent
Precise; 1st order		$\pm 1/20,000$ to $1/200,000$ of distance	Simple and inexpensive; direct observation	Corrections for temperature and slope; standard chain tension must be used	Excellent
Electronic distance measurement (EDM)	20 to 3000 m	$\pm$ 1/50,000 to 1/300,000 of distance	Precise, long-range, fast; usable over rough terrain	Accuracy influenced by atmospheric conditions; accuracy at short ranges (30 to 90 m) is curtailed for most instruments	Good
Optical leveling ordinary; 2nd–3rd order	3.0 to 100 m	± 3 to 5×10^{-2} cm	Simple, fast; particularly with self-leveling instruments	Has limited precision; requires good benchmark nearby	Excellent
Precise; 1st order	3.0 to 30 m	$\pm 1 \times 10^{-2}$ to 5×10^{-3} cm	Most precise	Requires good benchmark and procedures	Excellent
Offsets from baseline theodolite and scale	0 to 1.5 m	$\pm 0.5 \times 10^{-2}$ to $\pm 0.5 \times 10^{-3}$ cm	Simple; direct observation	Requires stable baseline; repeat the sight from opposite end of baseline	Excellent
Laser and photocell detector	0 to 1.5 m	$\pm 0.5 \times 10^{-2}$ cm	Faster than transit	Is seriously affected by atmospheric conditions	Good
Triangulation	Varies according to instrument quality and accuracy of baseline; best under 200 m	$\pm 1 \times 10^{-2}$ to 5×10^{-3} cm	Usable when direct measurements not possible; good for tying to external benchmarks	Precise measurement of base distance and angles; good benchmarks	Good
Photogrammetric	Virtually unlimited	$\pm 1/5000$ to $1/50,000$ of distance	Can record hundreds of potential movements at one time for determination of overall displacement pattern	Poor weather conditions degrade image quality and resolution of station position	Good

[1] Modified from Cording, and others, 1975.

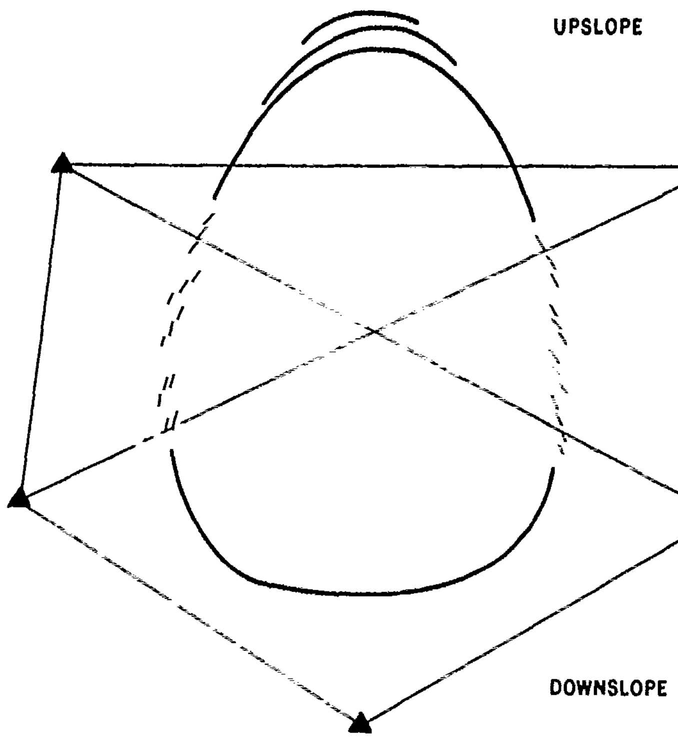


Figure G-23. A simple bench mark and triangulation leveling network for a known or suspected slope movement mass (From Sowers and Royster, 1978).

showing elevation contours and major topographic features superimposed on a screened-base photographic image of the area. This base should be utilized immediately to geologically map all features which are felt to represent the character of the portion of the slope under movement and its near environs.

Often two or three traverses down the long axis of the slope will be sufficient to establish the overall geometry of the displacing mass. Tensile fractures that will open from time to time on the slope should be monitored by mapping and placement of survey hubs (stakes) on opposing sides of the crack. These serve as the basis for frequent taped measurements of the growing displacements.

For larger slope movements with a nearby promontory of overall view, it may be possible to install a phototheodolite, for the purpose of monitoring displacements by photogrammetric computation to accuracies in the range of 5×10^{-2} to 1×10^{-1} cm.

Transit and theodolite surveys have also been useful to monitor the positional locations of fixed stations on otherwise rigid structures, such as retaining walls and sheet pile bulkheads.

Triangulation, linear offsets, and simple chaining can all be employed to detect deformations relative to a stable benchmark or bench line of about $\pm\ 5\times10^{-1}$ to 1.0 cm. The Bureau of Reclamation's *Earth Manual* Procedure E-32, (1974) and U.S. Army Engineer Manual 1110-2-1908 are standards for location and construction of survey monuments utilized for geotechnical instrumentation purposes.

South Dakota DOT (Bump, 1979) has developed a sighting device for installation at the collar of borehole deflection measurement casings. This device consists of a salvaged and refitted survey level instrument arranged for a tight fit into standard Slope Indicator Co. casing. The alignment device provides a precise, non-magnetic, tracking-groove alignment in areas influenced by large amounts of steel. The device shown was fabricated from an accident-damaged instrument and was fitted for use near a large steel girder bridge suffering abutment deformation.

G.7 SURVEY CONTROL FOR INSTRUMENTATION

Benchmarks and benchlines represent the usual method of establishing a stable or non-moving position with which to use in reference to on-site instrumentation. Benchmarks serve as individual, stable monuments, or the monuments marking the ends of a stable benchline. The benchmarks should be specified in contract documents and should be placed in an array that will represent a basis for independent review of the stability of its own alternate points, on the basis of positional measurements of its own turning points. The ideal benchmark consists of a central pipe or rod, anchored to depth and enclosed in an outer, friction-free casing arrangement, so that the interior rod is protected from possible down-drag of the surrounding soil or rock. Bond-breaking coatings such as oil-soaked waste or asphaltic compounds can be used to serve this purpose.

G.8 ACCURACY AS A CONSIDERATION IN INSTRUMENTATION

Most experts working with standard deformational phenomena, such as slope movements and foundation settlement, will be able to judge at about what level of deformation the project structures will be impacted negatively. The accuracy of the instrumentation should be high enough as to provide several to dozens of incremental readings in the range below that the undesirable level of deformation or stress accumulation. Long-span differential settlement of flexible foundation members can be detected by building-mounted devices or survey techniques to about $\pm\ 5 \times 10^{-1}$ cm; more rigid foundations such as mats will show differential settlements in the 1×10^{-1} cm level. Relatively large roadway embankments, based on the general experience of designers of earth dams (Gould and Dunnicliffe, 1971) shows that extension fractures tend to develop when longitudinal strain reaches about 0.1 to 0.3 percent; unless the roadway embankment is in fact a dam, it is likely that instrumentation will not be specified until and unless some visible indication of slope distress is noted.

Favorable conditions must be designed into the instrumentation program so that the devices are allowed to function solely for the purpose intended and are as free as possible from anticipated exterior influences. Table G-9 is a list of factors that should be considered in specifying the instrumentation and which will directly affect the accuracy of their readouts (modified from Gould and Dunnicliffe, 1971).

Table G-9
Factors Affecting Instrumentation Readout Accuracy

Instrument Design Features
- General sensing ability (level of discrimination)
- Readout sensitivity (human observer)
- Digital or tape recorded level of discrimination
- Durability (resistance to damage).

Installation Procedures
- Avoidance of damage
- Improper installation.

Exterior Environmental Influences
- Temperature
- Vibrations (ambient and transitory)
- Corrosion
- Moisture

Instrument/Host Medium Interaction
- Designed displacement or sensing function should be unimpaired by incremental host-medium deformation occuring between sensing points.
- Stress field should be disturbed as little as possible.

Observer Care
- Sensible observation procedure
- Specified data reduction procedure
- Calibration at timely intervals
- Method of conversion to reporting format.

G.9 INSTRUMENTATION FOR HAZARD WARNINGS

Transportation facilities constructed for extreme need in terrain affected by geologic or meterologic constraints (i.e., hazards) can be made more safe through the installation and monitoring of instrumented warning systems. The literature of natural event warning systems applied to transportation systems is not broad. Many experiments have been made, including instrumentation of the Fountain Slide on Interstate 80, some 105 km east of Portland, Oregon, along the Columbia River. The Fountain Slide (Munoz and Gano, 1974) was instrumented by 63 inclinometers and numerous associated piezometers and exploratory borings. The slope movement mass appears to be a debris slide of andesite and basalt blocks in a matrix of sandy silt and silty clay. Relatively large volumes of perched water are believed to be trapped in the slide mass which has also proved difficult to drain. The slide, which is some 2 km long (upslope) and 1 km in extent across the highway, has been in motion since early highway construction was attempted across its toe in the 1920's. An integrated warning system, developed on the basis of remote sensed readings from the instrumentation, was installed at a highway shoulder position away from the slide, but has not functioned satisfactorily due to electronic problems.

Hazard warning plans, based on instrumentation, can be ideally developed on the basis of the three stages of movement familiar to travelers; green (a *go* condition equivalent to safe); an *amber* condition equivalent to *unsafe condition probable);* to a *red* condition (a *no-go* condition equivalent to unsafe;

Table G-10
Natural Hazard Warning System Based on Instrumentation

Condition	Criteria	Analysis	Action
Green	Normal background	Analyze for improvement of background determination	Analyze, revise criteria as necessary.
Amber	Acceleration of any indicator, above determined safe threshholds.	Comparison of instrumented data, visual inspection, and external factors such as precipitation	Verbal report; site meeting; written report; decisions.
Red	Rapid acceleration of any indicator or acceleration of two indicators.	Comparison of dependency of indicators; inspection/visual monitoring of site.	Decision to clear the site of human activity; site meeting; remedial measures.

Franklin 1977

hazard occurrence eminent). Some of the philosophy for instrumentation of natural hazards warning systems will be found in Franklin, 1977. A synopsis of an instrumented warning system is contained in Table G-10. Such a warning system has been installed and favorably operated by the California DOT at one of the Malibu landslides on the Pacific Coast Highway.

G.10 CONTRACTS AND SPECIFICATIONS

As the case of other construction arrangements, instrumentation should be governed by specifications designed for the particular project and made part of a contract between the owner and the contractor. Cording, and others, 1975, have listed the essentials of good specifications for instrumentation contracts (Table G-11, as modified herein):

Draft specifications should be reviewed by an instrumentation specialist, for appropriateness and relavancy with the actual requirements of the project. Unless the owner or design engineer is absolutely certain about the necessity to use a particular manufacturer's equipment, it is wise to make the specifications open to competition from various suppliers subject to verification of reliability and subject to rejection of hardware which is felt to be substandard to the desired purpose.

The contract for instrumentation equipment, installation, and monitoring should be written so as to protect the owner's interests in securing timely and accurate instrumentation data. The contract should specify what organization will be making the data reduction and analyses, and the manner in which the reduced data, conclusions, recommendations are delivered to the owner and his representative. Minimally, the contract should insure the following:

- Description of the instrumentation, installation and monitoring.
- Delivery of acceptable installations and ensuing data.
- Minimize the owner's exposure to additional costs over and above that of the contract.

Table G-11
Essentials of Instrumentation Contract Specifications

- Statement of purpose of instrumentation
- Responsibilities of parties to the contract instrumentation.
- Contractor cooperation with other parties.
- Hardware and quality assurance requirements.
- Installation procedures, layout and schedule for compliance.
- Instrumentation contractor support services to the Owner.
- Maintenance and reading procedures and schedules
- Delivery of observation records
- Disposition of instruments and support equipment.
- Measurement of services and payment.

Modified from Cording, and others, 1975

- Establish the responsibilities of all parties working with the instrumentation or its derived or computed data.
- Specify the nature of a working relationship between all parties to instrumentation and its derived data.
- Establish a method of accommodating instrumentation-related changes in work as the project proceeds.

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APPENDIX H
Subsurface Investigations for Earthquake-Resistant Design

H.1 EARTHQUAKE DAMAGE TO TRANSPORTATION SYSTEMS

Earthquakes can affect transportation systems through one or more of the following factors:

- ground rupture from displacements along faults
- ground movements from slope instabilities, sloughing and lateral sliding
- reduction in soil strength as a result of vibratory loading
- seismically-induced soil settlements
- changes in lateral stresses on walls
- increased stresses in structural members caused by ground shaking

This Appendix is intended to describe the effects of earthquakes on transportation systems, and to present a discussion relative to subsurface investigations to aid in earthquake-resistant design.

H.1.1 Ground Rupture

A fault is a break in the earth's crust on which there is movement parallel to the surface along which the break occurs (Stokes and Varnes, 1955). Several types of fault movement have been observed to occur as described by Weigel and others (1970). Figure H-1 illustrates some common types of faults. The ground surface may shift vertically or horizontally, or in any combination of these, depending on the fault type. Fault movements during individual earthquakes may range from millimeters to meters (inches to feet) depending on the magnitude of the earthquake, the total fault length, the length of the fault along which movement occurs and other factors. If a fault exists beneath or close to a transportation structure, fault movement may result in differential movements within the structure, possibly leading to structural distress or even failure. For example, vertical fault displacment may make a highway impassable and horizontal or vertical displacements of a bridge support could cause varying amounts of damage or collapse. Fault displacement can produce severe loading on most structures, so recognition and identification of faults are critical to investigation and design.

H.1.2 Ground Shaking

The most widely felt effect of earthquakes is ground shaking or vibration. Ground shaking occurs in all directions, but for convenience the motion is resolved into three components, one vertical and two mutually-perpendicular horizontal motions. Frequently for geotechnical problems, the horizontal components are considered in design while the vertical component is ignored. Damage from ground shaking takes many forms as discussed in the following sections.

H.1.2.1 Liquefaction. Many soils tend to compact or densify when subjected to vibration. As a result of the rapid loading produced by earthquakes, pore water cannot drain from some saturated soils quickly enough to allow the soil to compact. This causes pressure in the pore water of the soil to increase. If the earthquake shaking is of sufficient severity and duration, the pore water pressures may become high enough such that the soil loses nearly all its shear strength and begins to behave as a viscous liquid. Saturated, loose, medium to fine sands are the soil types generally found to be most susceptible to this phenomenon called liquefaction. Foundations on liquefied soils can lose their support and experience large settlements. Conversely, buried tanks have been observed to "float" out of position from liquefaction. Slopes in liquefied soils can flow laterally until nearly level; movements of tens of meters (hundreds of feet) can result. Since liquefaction can cause

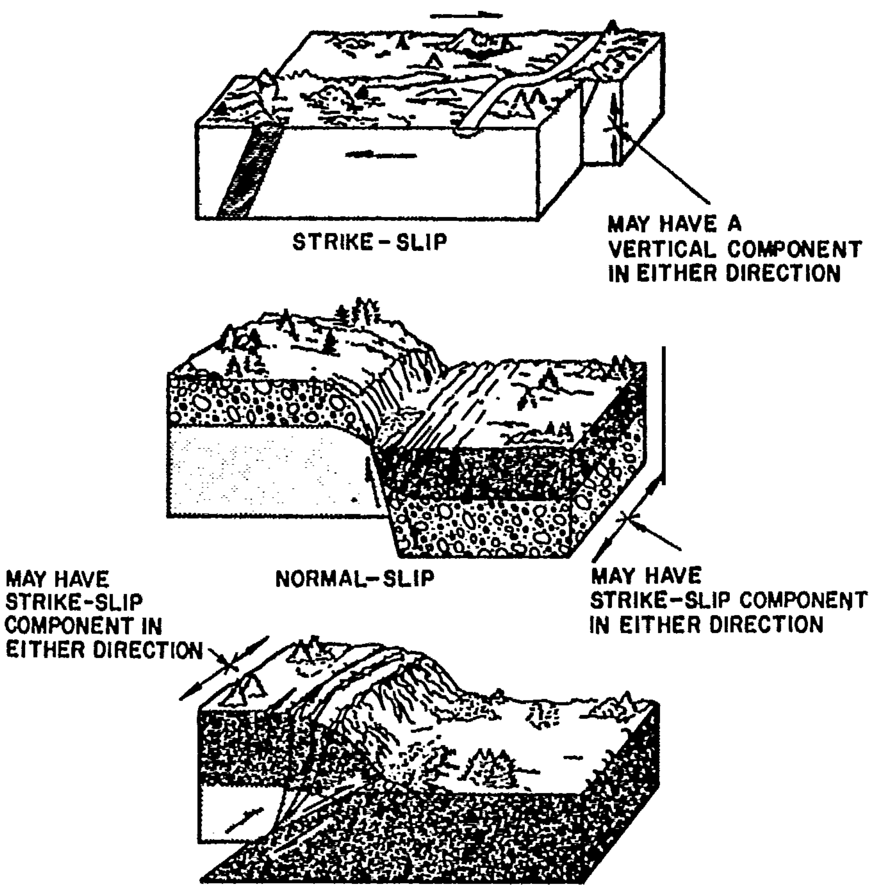

Figure H-1. Block diagrams showing effects of surface displacement along a strike-slip, normal-slip, and reverse-slip fault. Taylor and Cluff (1977).

significant damage to facilities, it is important to identify liquefaction-susceptible soils in subsurface investigations and to provide suitable designs to accommodate such conditions.

H.1.2.2 Slope Instability. Slope movements caused by liquefaction were discussed above. Seismically-induced slope instabilities are not confined to conditions of soil liquefaction. Seismic forces imposed on cut and fill slopes can result in overall instabilities from rotational or sliding movements of soil masses, sloughing, rock falls and debris slides. The damage which occurs is dependent upon the type of engineered structure, the type and magnitude of movement and the volume of material involved.

H.1.2.3 Settlement. Even in soils not susceptible to liquefaction, pore water pressures may build up during earthquake shaking. The magnitude of increased pore pressure depends on the soil properties and the severity of shaking. When the shaking lessens significantly, the pore water pressures begin to dissipate. Drainage of water from the soil occurs until the excess pressures are dissipated. This outflow of water often results in a decrease in soil volume and can lead to settlements of structures.

The tendency for dry or partially saturated cohesionless soils to settle during vibration is a well-known principle of soild compaction. Densification of such soils during earthquakes is a commonly observed phenomenon.

The amount of seismically-induced settlement may be enough to cause structural damage to bridges, retaining walls and other engineered structures. In addition, embankments may settle from densification of the fill and/or foundation soils.

H.1.2.4 Soil-Structure Interaction. The horizontal ground accelerations associated with earthquakes result in changes in horizontal forces on below-grade structures. These forces can reduce the stability of retaining walls and result in increased loadings on other structure types. Indeed, toppled retaining walls are among the most prominent results of earthquake shaking. Although the presence of loose or soft soil materials often contributes to instability of retaining walls, the presence of these soils is not a necessary condition.

H.1.2.5 Effect of Local Soil Conditions on Earthquake Motions. Local soil conditions at a site can alter the earthquake motions at or near ground surface relative to those which occur at the level of bedrock. The amplitude to motions can be increased or decreased and the frquency and duration of shaking altered. Specific changes which do occur are the result of many factors such as the thickness and properties of the soils, the intensity of shaking at the rock level, and distance from the earthquake epicenter. The resulting ground motion characteristics influence the stresses and displacements which develop in surface and below grade structures. Depending on the nature of the projects, it may be important to evaluate the effects that local soil conditions will have on resulting earthquake motions.

H.1.3 Summary

It is generally uneconomical to design transportation structures to be completely earthquake resistant. It is therefore important to be aware of the costs of preventive measures compared to the risk and cost of failure or severe damage from seismic loading. For example, if the only consequence of seismically-induced embankment settlement is relatively minor repaving, extensive analyses and preventive measures may be unwarranted. However, if the embankment also provided support for a bridge abutment, the same amount of settlement might cause significant bridge damage and would warrant more detailed design and special construction.

Hannon and Jackura (1978) provide a list of considerations for the seismic design of highway structures which include:

- Potential for loss of life
- Costs and difficulty of repair or reconstruction
- Availability of alternate routes or sufficient right-of-way to detour traffic in the event of damage
- Importance of facilities serviced by the structure
- Volume of traffic

The following section deals with approaches for subsurface investigations in connection with seismic phenomena. Evaluation of the above factors will provide perspective concerning the extent of seismic investigation and design required to minimize potential damage.

H.2 SUBSURFACE INVESTIGATIONS FOR SEISMIC CONDITIONS

This section is intended to summarize the geotechnical information needed from a subsurface investigation for seismic design, and some suggested methods to obtain the required data. The effort and cost for an actual seismic investigation should reflect the probability that significant levels of fault movement or earthquake shaking will occur, as well as the consequences of such occurrences.

H.2.1 Faulting

An investigation of the effects of fault displacement on a transportation facility should incorporate a geologic investigation of the fault including (Figure H-2):

- Determining the type of fault and its orientation
- Determining its history of activity including time period between past fault movements, length of fault rupture and amount of fault displacement
- Assessing the width of the disturbed zone across the fault

These steps can be accomplished through the use of airphotos, test borings, geologic mapping, test pits and trenches plus other specialized methods such as radioactive carbon dating of materials in the fault zone. There is a great deal of information in the literature concerning the assessment of faults. Sherard, Cluff and Allen (1974) and Taylor and Cluff (1977) provide good summaries of investigations.

H.2.2 Liquefaction

The state-of-the-art in assessing liquefaction potential is still in the relative infancy and evolving. Considerable engineering judgement is required in evaluating the liquefaction susceptibility of soils at a site. The items presented here should be used with an understanding of this present state of knowledge. The primary factors affecting the liquefaction susceptibility of soils are:

1. The degree of saturation
2. The weight of the overlying soils (overburden pressure)
3. The soil grain size and gradation
4. The degree of compactness or relative density of the soils
5. The intensity of the earthquake shaking

The goal of the subsurface exploration program is to establish factors 1 through 5 above such the potential

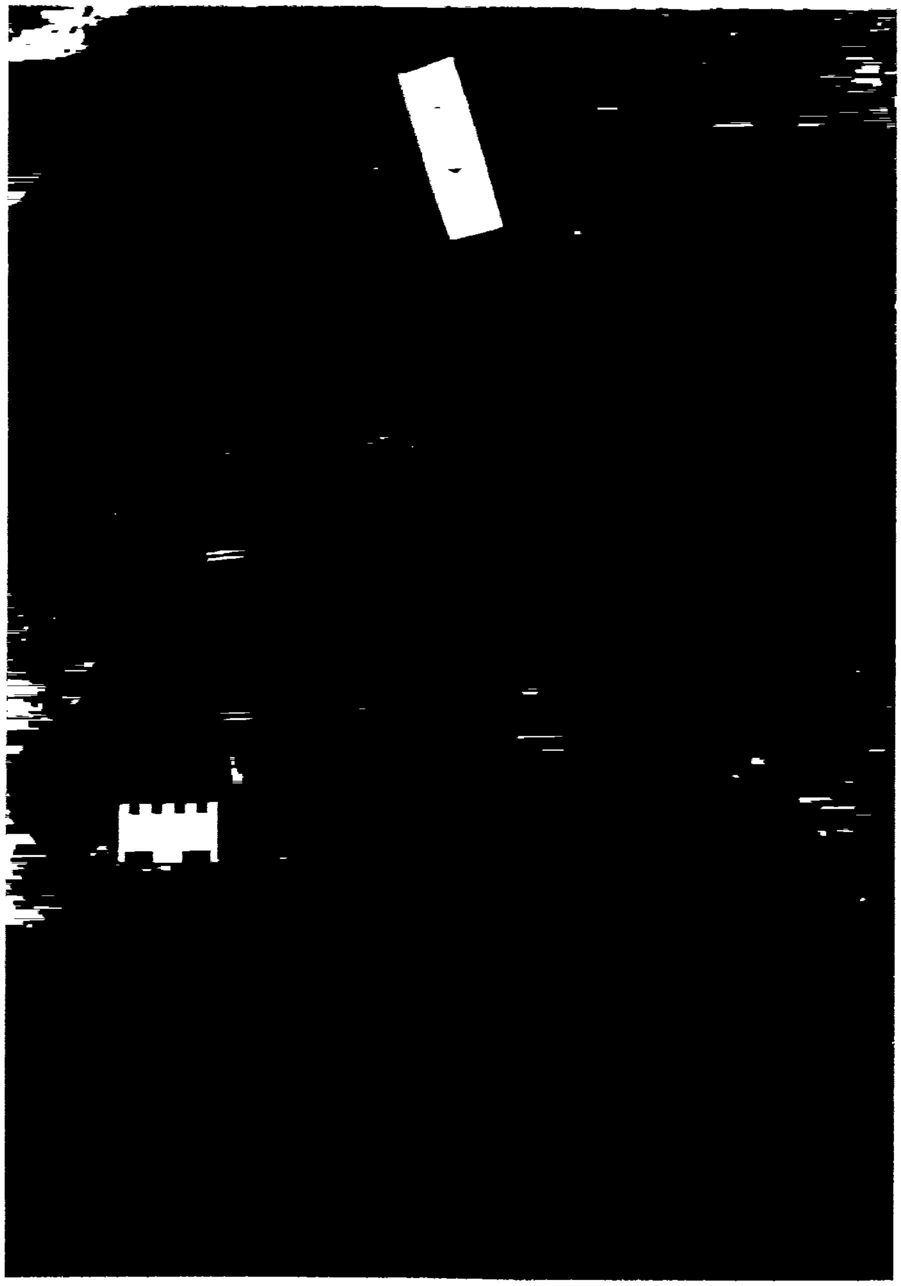

Figure H-2. A close look at the actual break-surface of the notorious San Andreas Fault. From an exploratory trench at Pallet Creek, California. (A.W. Hatheway)

for liquefaction can be evaluated with sufficient reliability.

H.2.2.1 Saturation. For pore water pressure to build in the soil to a level such that liquefaction could occur, the soil must be saturated. Saturation can be evaluated by locating the groundwater table at the site. Methods which can be used to determine the groundwater level include:

- Study of available data on geology and groundwater levels
- Site visits to observe wetlands, streams or standing surface water
- Test pits (for shallow ground water levels)
- Observation of water levels in test borings
- Installation of observation wells and piezometers

H.2.2.2 Overburden Pressure. The overburden pressure or effective vertical stress can be established on the basis of soil total unit weight and groundwater level. For most soils, the total unit weight can be established on the basis of experience and available published correlations. In special cases such as for lightweight volcanic soils it may be necessary to obtain undisturbed samples for actual measurement of total unit weight in the laboratory.

H.2.2.3 Grain Size and Gradation. The soils considered to be most susceptible to liquefaction are clean, medium to fine sands. Silty or gravelly sands are generally considered to be somewhat less susceptible to liquefaction. Silts and clays may liquefy under certain conditions, but are generally considered to be more resistant to liquefaction than sands. Samples of soil should be recovered to permit visual classification and for laboratory testing by sieve and/or hydrometer methods. Atterberg limits tests on soils exhibiting plasticity are frequently an aid in assessing liquefaction susceptibility.

H.2.2.4 Relative Density-Cohesionless Soils. The most difficult factor to determine with reliability in the subsurface investigation for liquefaction potential is the relative density of the soil. Loose or soft soils are most readily liquefiable; dense or stiff soils are generally considered to be non-liquefiable. The most frequently used subsurface investigation technique in the United States for the assessment of the *in situ* relative density of cohesionless soils is the Standard Penetration Test (SPT; see Appendix B.4.1).

Other *in situ* relative density measurement techniques are often used (for example, the static Cone Penetration Test or CPT; see Appendix B.3), but the SPT is most popular for two main reasons:

- A soil sample is obtained which can be visually classified and tested in the laboratory for grain size and plasticity characteristics.
- As a result of the widespread use of the test, considerable empirical data now exist for instances of liquefaction and non-liquefaction from past earthquakes. These data can be used as a guide for assessing liquefaction potential.

An example of SPT correlations is provided in Figure H-3 where data from the Niigata, Japan earthquake of 16 June 1964 are summarized from Seed (1979). Please note that this figure is only applicable to the specific subsurface conditions of the Niigata area and for the intensity of ground shaking produced by that particular earthquake.

The use of the SPT for assessing liquefaction susceptibility is not without drawbacks. The SPT has been criticized by some workers for its reliability as an indicator of low relative density soils which may be susceptible to liquefaction. Variations in drilling operations and deviations from prescribed procedures have unknown and possibly significant effects on the results. Additionally, there are many aspects of soil structure and fabric which may influence liquefaction susceptibility, but which cannot be individually determined by the test. However, the quantity of available data and the apparent consistencies in correlations of SPT resistance with actual cases of liquefaction make this the most well-documented procedure currently available.

The static CPT Resistance may be correlated with the SPT blow count to allow for the use of the SPT liquefaction correlations. General correlations between the SPT and CPT are available, but a site-specific or region-specific correlation is preferable where possible.

Correlations relating relative density to liquefaction potential are available using relative density as defined by:

$$D_r\,(\%) = \frac{e_{max} - e_o}{e_{max} - e_{min}} \times 100$$

in which e_o is the *in situ* void ratio, e_{min} is the minimum void ratio and e_{max} is the maximum void ratio. The use of such correlations msut be accomplished with considerable care because of potential errors associated with measurement of void ratio.

H.2.2.5 Liquefaction of Silts and Clays. The liquefaction potential of silts and clays is not as well-documented as it is for cohesionless soils. Static liquefaction of sensitive clays in slopes has been observed in Scandinavia and North America, but the potential for liquefaction of these materials under seismic conditions is uncertain.

The only way currently available to assess liquefaction potential in silts and clays is by recovering undisturbed samples for laboratory testing. Atterberg limits, consolidation and strength tests can be performed to assess the overconsolidation ratio, sensitivity and shear strength. Cyclic triaxial or simple shear tests can be performed to evaluate soil properties under repeated loading.

H.2.2.6 Laboratory Testing for Liquefaction Susceptibility. As an alternative to the SPT, CPT or other *in situ* approaches, undisturbed samples of soils may be recovered for cyclic laboratory testing. The samples can be subjected to confining pressures of the same magnitude as existing in the field, and subjected to cyclic loading patterns in the triaxial cell or simple shear device (See Section 9) designed to model earthquake effects.

- At least some of the structure, and all of the grain size characteristics, of the actual soil is retained in the tested sample

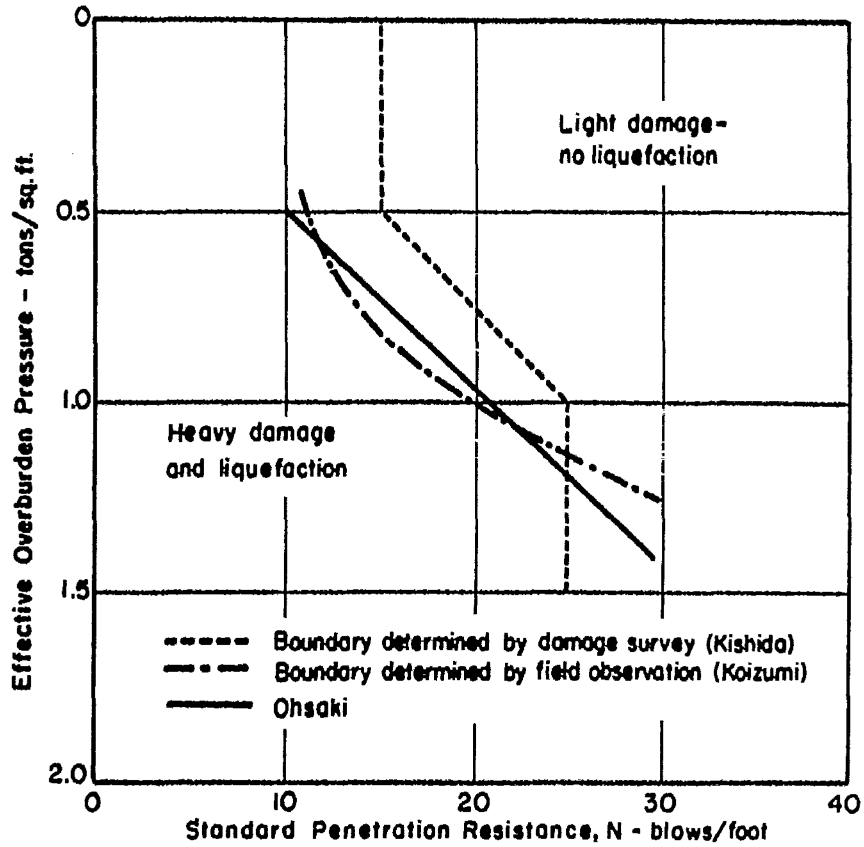

Figure H-3. Correlation of SPT Resistance with Liquefaction for 1964 Niigata, Japan Earthquake (from Seed, 1979).

- The stress conditions in the test can be controlled
- The results of the test are visually observable

The disadvantages are:

- The tests are expensive to conduct
- Obtaining undisturbed samples of cohesionless soils is very difficult
- The accuracy of the cyclic loading in modeling seismic conditions is uncertain
- The results have not been verified by extensive field observations.

For silts and clays, however, and possibly under sloped ground conditions, laboratory testing is the only means currently available for assessing liquefaction potential.

H.2.3 Slope Stability Under Seismic Conditions

Typical subsurface investigations for assessing cut or fill slope stability under seismic conditions are not greatly different from investigations used for a static slope stability assessment. The important factors to be determined are the type and extent of the soils, the groundwater conditions, and the soil unit weight and shear strength. *In situ* methods of determining shear strength can be used (See Appendix B) and disturbed or undisturbed samples can be recovered for laboratory strength testing (See Section 9). An investigation of the liquefaction potential of the natural soils within a cut slope or within the fill or foundation soils of an embankment should be conducted using field and laboratory procedures outlined in Section H.2.2. Cyclic laboratory tests on undisturbed samples may be necessary in critical situations to account for the higher static shear stresses in a slope as opposed to the level ground condition. Even if liquefaction is ruled out as a possibility, it may be necessary to evaluate the potential for soil strength loss during or after earthquakes because of pore water pressure build-up. Cyclic triaxial and simple shear tests can be used to develop such data on pore pressure development.

H.2.4 Seismically-Induced Settlement

The features to be determined in a subsurface investigation for assessing the potential for seismically-induced settlement are very similar to those required for liquefaction susceptibility. The important quantities are the grain size, gradation, and the relative density. The location of the groundwater table is also needed, but saturation of soils is not a necessary condition for seismic settlements to occur. As with liquefaction, loose, cohesionless soils are most susceptible to densification by earthquake vibrations. It is unlikely that significant seismically-induced settlements could occur in clay under level ground. Subsurface investigations should make use of the techniques discussed in Section H.2.2 for assessing the potential for seismic settlement.

H.2.5 Dynamic Earth Pressures on Walls and Other Below-Grade Facilities

A subsurface investigation performed for assessing soil-structure interation should be aimed at determining the strength and stress-strain characteristics of the soils in the vicinity of the structure. *In situ* or laboratory strength tests on undisturbed soil samples can be used for determining soil strength. The stress-strain behavior of the soil can be investigated through *in situ* geophysical tests such as the downhole, uphole or crosshole methods (see Section 6), or by cyclic laboratory tests such as resonant column, cyclic triaxial, or simple shear tests, performed on undisturbed samples.

H.2.6 Effect of Local Soil Conditions on Earthquake Motions

The thickness and unit weight of each soil layer present at a site and the shear modulus or shear wave velocity and damping ratio of each stratum are required if the analysis of the effects of local soil conditions on earthquake motions of a site is to be accomplished. Layer thickness and unit weight determinations are easily accomplished in standard subsurface explorations. Shear wave velocity can be measured using geophysical methods or laboratory tests on undisturbed samples, as discussed in H.2.5. Alternatively, the shear modulus of the soil can be measured in a cyclic simple shear device or can be calculated from the results of strain-controlled cyclic triaxial tests (See Section 9). Soil damping characteristics can be obtained from cyclic laboratory tests or from published correlations.

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APPENDIX I
Geotechnical Contributions to Environmental Reports

Transportation systems in the United States are constructed only after completion of environmental impact analyses. Such analyses were initiated in 1969 with passage of the National Environmental Policy Act (NEPA) and have been further amplified and defined by other Federal and state legislation. Although Agency geotechnical and geological personnel are seldom placed in charge of developing the various required reports and statements, their speciality data often represent some of the most important portions of such reports. The major pieces of Federal implementing legislation and directives are as follow:

- National Environmental Policy Act (NEPA) of 1969
- Federal Highway Administration (FHWA) Implementation Procedures; Code of Federal Regulations, Title 23
- US Department of Transportation, Urban Mass Transit Authority (UMTA) Implementation Procedures; Code of Federal Regulations, Title 49
- US Department of Transportation, Procedures for Considering Environmental Impacts, Order DOT 5610.1C
- Federal Safe Drinking Water Act, Section 1424(e); administered by the US Environmental Protection Agency (USEPA)

Many of the states have enacted supplemental legislation governing the nature and content of environmental impact studies and the manner in which they are undertaken. Generally, the state regulations are supplemental to the degree that they either accommodate or incorporate the Federal acts, often the state requirements are more stringent.

I.1 INTENT OF ENVIRONMENTAL IMPACT ANALYSES

The process of environmental impact analysis is not always well understood. The laws and regulations specifying the analyses are numerous and complex. Officials placed in charge of developing the analyses are faced with the question of how much of an effort will be sufficient to analyse and report on the environmental impacts of a proposed project. Many times the costs associated with compilation, assessment, review and presentation of environmental analyses represents a significant portion of the funding for a given project. While the implementing legislation did not intend for the analyses to represent such outlays, often the studies do grow beyond reasonable limits. The basic intent of the analyses, as stated in Section 120(2)(C) of NEPA is to produce a "detailed statement" describing the following relationships of the proposed project and the environment:

- the environmental impact of the proposed action;
- any adverse environmental effects which cannot be avoided should the proposal be implemented;
- alternatives to the proposed action;
- the relationship between local short-term uses of man's environment and the maintenance and enhancement of long-term productivity;
- any irreversible and irretrievable commitments of resources which would be involved in the proposed action should it be implemented.

Geologists and geotechnical engineers are often most closely associated with many of the most important impacts associated with construction and operation of transportation systems. The contributions of these professionals are important from the standpoint of clarity and accuracy, and more importantly, geologists and geotechnical engineers are in a position to make environmental impact assessments at the most favorable terms of costs and time consumed in the process.

I.2 GENERALIZED PROCEDURE OF ENVIRONMENTAL IMPACT ASSESSMENT

The manner in which environmental impact assessments are compiled, reviewed, presented and analysed varies from Agency to Agency, from State to State. The general procedure however, is one of progression from the general to the more detailed and from the promulgating agency to the public. What is not really clearly defined by the various regulations is the real detail of each element of the assessment. Through a good understanding of the general process, the Agency process, and the basic intent of the implementing regulations, those who are charged with developing portions of impact assessments should be able to offer sound advice not only in terms of environmental impact, but in terms of the costs in funding and time required to produce a suitable end product.

In broad terms, the US DOT has prescribed a policy (USDOT, Order 5610.1C) which calls for the integration of national environmental objectives into its activities and those State programs which it funds. These objectives are as follows:

- Avoid or minimize adverse effects wherever possible;
- Restore or enhance environmental quality to the fullest extent practicable;
- Preserve the natural beauty of the countryside and public park and recreation lands, wildlife and waterfowl refuges, and historic sites;
- Preserve, restore and improve wetlands;
- Improve the urban physical, social and economic environment; increase access to opportunities for disadvantaged persons;
- Utilize a systematic, interdisciplinary approach in planning and decision making for projects which may have an impact on the environment.

Transportation Agency personnel will naturally wish to accommodate these objectives in the least possible time and at the most reasonable expenditure. The generalized process of environmental impact assessment for transportation system construction and improvement occurs in approximately the following sequence of events.

I.2.1 Planning and Early Coordination

Planning and early coordination is conducted at the conceptual level of system development. The assessment team should evaluate the general scope of the proposed project and develop a list of potential environmental impacts (such as shown in Table I-1). A list of Federal, State, Regional and local agencies and activities which may hold important data or which should be involved in some way in the review process should be compiled. The need for this action is coordination which generally results in early discovery of important contributory data and which also tends to ease potential conflicts and to make the impact assessment process flow more optimally.

I.2.2 Scoping the Level of Assessment

With all contributing experts available, the system planners should indicate the basic objectives of the project and the design guidance under which they are operating. The experts should be given an opportunity to evaluate the proposed development concept in terms of what each knows of the region in which the project is contemplated. It has been demonstrated repeatedly that the most difficult of actual or perceived environmental impacts deal with sociological factors, historic sites, areas of rare or unusual natural scenic value, the habitat of endangered forms of wildlife or the occurrence of groundwater. Many times the value of the environmental element is viewed by the public on a purely emotional basis. Extreme efforts must be undertaken by scientific and engineering professionals to determine the actual impact of the project, and to explain the impacts in clear and graphic terms to the public or its special-interest protection groups. The main problem in this communication is to portray technical information and concepts to non-technically-trained citizens. The relative ease or difficulty of such undertakings will probably become apparent at the scoping stage of the project.

I.2.3 Initiation of the Environmental Assessment

At the termination of the scoping stage, the environmental impact assessment team should have been formed and a list of assigned topics of investigation and submission deadlines determined, as well as coordination of responsibilities. The scoping, assignments and deadlines should be constructed in a two-phased manner, so that initial findings and assessments can

Table I-1
Geological and Geotechnical Contributions to Environmental Reports

ENVIRONMENTAL PARAMETER	POTENTIAL NEGATIVE IMPACTS	CAUSATIVE ACTIVITY	MEANS OF ESTIMATION	PRIME INDICATORS	TYPE OF QUANTIFICATION USUALLY POSSIBLE
LAND SURFACE	EROSION	Grading; surface disturbance and disposition of excavated materials	Geologic reconnaissance; interpretation of USDA County Soil Survey Maps and geologic maps	Expected physical properties of each soil or rock unit	According to U.S. S.C.S. Standards
	CUTS	Grading; surface disturbance and disposition of excavated materials	Preliminary slope stability analysis; suggests stable slope angle; hence extent of cut	Expected physical properties of each soil or rock unit	High degree of definition of areal extent of cut
	FILLS	Alteration of land surface by placement of earthworks	Preliminary slope stability analysis; suggests stable slope angle; hence extent of fill	Expected physical properties of each soil or rock unit	High degree of definition of areal extent of fill
	SLOPE MOVEMENTS	Inadvertent activation of ancient/dormant unstable slopes	Detection of existing unstable masses	Aerial photographic interpretation	±20 percent by area
SURFICIAL SOIL	DENIAL OF AGRICULTURAL ACCESS	Placement of earthwork and structures; determination of extent of cut/fill slopes; delimitation of right-of-way	Preliminary geological and geotechnical findings are incorporated by highway engineers; minor explorations may be required	Overlap of construction features on USDA soil series of agricultural value	According to U.S. S.C.S. Standards
	QUALITY	Surface disturbance of land, for marshalling, maintenance and access; subsequent release from Agency jurisdiction as surplus to needs	Planner's stated requirements	Exposure of poor quality soil/rock units	According to U.S. S.C.S. Standards
SURFACE WATER	WATER QUALITY	Grading; relocation of drainage courses and waterways	Field reconnaissance Topographic interpretation	Exposure of unwanted soil/rock minerals/groundwater; uncontrolled erosion and sedimentation	Mainly presence; volume estimations to ±20–40 percent by volume
	WATER QUANTITY	Grading and diversion	Field reconnaissance Topographic interpretation	Diversion measures	±10–40 percent by volume
GROUND WATER	WATER QUALITY	Structures which promote infiltration of roadway runoff within or in near vicinity of right-of-way	Well survey Chemical analyses	Chemical analyses and knowledge of hydrogeologic regime	Estimated conformance/nonconformance with Federal Drinking Water Standards
	WATER QUANTITY	Earthworks; cuts which intercept piezometric surface; fills and paved surfaces which obstruct infiltration	Well survey Geologic reconnaissance Geologic interpretation	Water levels Stratigraphy Topography Construction elevations	Estimated gross raising or lowering of water table; estimted "shadow" (area of extent) of this effect
AIR QUALITY	FUGITIVE DUST	Grading, earthwork placement; open-air blasting	Analysis of nature of soil/rock expected to be encountered	Lithology, grain-size, hardness, compressive strength, degree of weathering/alteration, rock structure	General estimate of probable occurrence
	BLASTING AND FUMES	Open-air blasting	Analysis of nature of soil/rock expected to be encountered	Nature of expected explosive	Specifications can be set for minimal or acceptable human impact; probable duration noted
SOCIOLOGIC	NOISE	Blasting	Comparison of nature of rock with established empirical relationships	Lithology, grain-size, hardness, compressive strength, degree of weathering/alteration, rock structure	Specifications can be set for minimal or acceptable human impact; probable duration noted
	VIBRATION	Blasting	Comparison of nature of rock with established empirical relationships	Lithology, grain-size, hardness, compressive strength, degree of weathering/alteration, rock structure	Specifications can be set for minimal or acceptable human impact; probable duration noted
	AIR PRESSURE	Blasting	Comparison of nature of rock with established empirical relationships	Lithology, grain-size, hardness, compressive strength, degree of weathering/alteration, rock structure	Specifications can be set for minimal or acceptable human impact; probable duration noted

be reviewed internally for the significance of potential impacts. If early returns indicate that the level of adverse impact is not significant, then steps may be taken to formulate a *Finding of No Significant Impact* (FONSI).

With completion of the FONSI, Agency personnel may terminate the assessment process and submit the finding through appropriate channels and hold the required public hearings. The object of this early determination is to save unnecessary expenditures of time and funding and to expedite completion of the proposed project.

The FONSI should be attached to a formalized *Environmental Impact Report* (EIR) or combined with the Report as a single document. The level of geological and geotechnical data contained in the EIR/FONSI should be what would be normally collected, analyzed and interpreted as a contribution to initial project planning. Field explorations such as drilling, test pitting, or detailed geologic mapping, are generally not required. Field geologic reconnaissance or photogeologic interpretations may be necessary if the project lies in a region in which geologic mapping is lacking.

I.2.4 Compilation of the Environmental Impact Report

The working document recording the process of collecting and analysis of environmental impact is the *Environmental Impact Report* (EIR). The EIR is edited into the *Environmental Impact Statement* (EIS), which is issued in draft form (DEIS) to reviewing agencies (State and Federal) and interested citizens. Hearings are held and comments of concerned agencies and citizens are addressed. The Final Environmental Impact Statement (FEIS) is the document summarizing all facts and deliberations concering the project. Generally speaking, in the case of Federally-funded projects, the proponent State Agency will prepare an Environmental Impact Report in all of the necessary final detail and the Regional Office of USDOT will review, edit, comment and release the document as its FEIS and DEIS for the project. If the project is predominantly funded by the State, then the State DOT will issue the EIS.

Detail to which the EIR and draft EIS is compiled is entirely subject to the judgment of the assessment team. In the natural course of events in environmental impact assessment, some compilers, reviewers, and intervenors increase the level of investigations used to determine the nature and extent of environmental impacts. Some of the most equivocable assessments are those with geological bases because the depth, areal extent and physical characteristics and engineering properties of natural materials are extremely variable.

I.2.5 Format of the Environmental Impact Report/Statement

For purposes of standardization in review, the US Council on Environmental Quality (USCEQ) has recommended in its 40 CFR Title 1502.10 regulations, the following format for EIR and EIS documents:

- Cover Sheet
- Summary
- Table of Contents
- Purpose and Need for the Project
- Alterantives Including the Proposed Project
- Environmental Consequences
- List of Preparers
- List of Notified Agencies, Organizations, and Persons
- Index
- Appendices

Considerable latitude is given to the assessment team in structuring the detailed content and layout of the report or statement.

I.2.6 Comments and Interaction

Inherent in the intent of the environmental impact assessment process is an open presentation to interested agencies and individuals. For transportation agencies, this list normally includes USDOT, USEPA and other State and Federal agencies having jurisdiction over lands or environmentally-sensitive aspects in the area occupied by or traversed by the proposed transportation project. Draft and final EIS document availability is required to be announced to the general public through the various communication media.

The EIR normally circulates only to the Agencies, other interested agencies and various consultants employed in compiling the report. Some of the issues may be discussed at public meetings held in the area of the project. The EIS, in draft and final form, is usually the first document released for general public scrutiny. EIS team members should strive to properly identify the environmental impacts and to treat each of them with as much data collection effort and analysis as is reasonable to identify the impacts, real or potential. Team members should carefully assess reviewer's concerns and should undertake such additional field work, such as may be reasonably required to clarify any missing aspects of the assessment.

I.2.7 Final Environmental Impact Statement

Final Environmental Impact Statements (FEIS) are prepared by the regional office of the Federal Agency providing principal funding for Federally-financed projects. An Agency team will prepare the FEIS from a complete final EIR provided by the State DOT or its consultant, or it will be compiled from a final EIR provided by an Agency team. Specifically, the EIS team will evaluate the document for consideration of the following aspects:

- Adequacy of coordination with all appropriate Federal State and local governments and regional commissions;
- Adequacy of the DEIS and supplements in identifying and defining environmental impacts and of presenting reasonable alternatives to the proposed project; and
- Adequacy in treating reasonable concerns identified by Agencies and individuals communicating with the Agency EIS team, and as voiced at public hearings.

I.3 CONDUCT OF STUDIES

The geotechnical parts of most Environmental Impact Reports can be compiled in phase with preliminary route selection and feasibility studies. Most of the identified environmental concerns of a geological or geotechnical nature also present questions of interest to project planning and design. As identified on Table I-1 and I-2, a variety of environmental parameters should be considered for potential impacts on most types: those that occur within the right-of-way (ROW), and those that are external to it. Most of the impacts that can be associated with the ROW itself, can be accurately established in terms of depth, areal extent and degree. The impacts are also generally one-time occurrences and will not vary essentially with time. Those impacts that can possibly affect the surrounding terrain are generally associated with surface water or groundwater or with slope movement masses which might be activated through construction related to the project.

Design features which may alter the general nature of surface drainage should be reviewed for potential impacts generated by the redistribution of flow; generally in the form of overbank flow, decreased discharge which does not now meet various former water supply demands, and increased erosion and sedimentation. Impacts are seldom transferred outside of the ROW except by flowing water, wind-dislodged surface particles of unstabilized earth, or by slope movements such as are generally classified as types of landslides. The most difficult of potential impacts to determine with specific accuracy are those relating to disturbance or alteration of the groundwater regime through which the ROW or project passes. Typical examples of effects of highway construction on local groundwater conditions are discussed and illustrated in Section 8.

Most potential groundwater impacts can be adequately assessed in at least a semi-qualitative fashion by careful review of the field observations that are normally required in preliminary or feasibility level geological investigations. Special field explorations such as installation of groundwater observation wells and conduct of pump tests hould ordinarily be avoided as being costly in excess of the returns. Since groundwater constitutes an emotional issue to many individuals whose residences, farms or business are in the near vicinity of a ROW, it is often difficult to produce absolute evidence from field explorations that will insure a definite level of impact or nonimpact.

Baseline studies are an important factor for consideration in highway planning. Later changes in surface and groundwater conditions are generally ascribed by abutters to the presence of the transportation project, when, in reality, they may caused by a number of other factors. Baselines are usually effectively determined by asking abutters for permission to record several waterlevel readings and a groundwater sample to be analyzed for the constituents covered in the Federal drinking water standards. The location and distance from the project center line for wells or other water sources to be so surveyed should be determined by a hydrogeologist. An observation period of one year is generally the minimal record length required to determine a representative seasonal fluctuation in groundwater levels.

Potential slope movements triggered or actuated by construction of transportation projects are of extreme importance to the cost and functionality of any such project. Most potentially unstable masses of rock or soil are identifiable by geopmorphic indicators which can be seen in field reconnaissance or by photogeologic interpretation (see Section 5). Other forms of slope movements can be created by transportation system component structures when an otherwise stable geologic condition is overloaded in terms of embankment or cut construction, when masses of soil or rock are isolated without restraint or gravitational load, or when groundwater or surface water conditions are altered so as to represent a potential increase of pore water (soil) or cleft water (rock) pressure. As in the case of groundwater impacts, data required for estimation of most slope movement activation poten-

Table I-2
Potentially-Negative Construction-Related Environmental Impacts

ENVIRONMENTAL PARAMETER	POTENTIAL NEGATIVE IMPACTS	BASELINE DESIRABLE OR POSSIBLE	APPLICABLE STAGE OF PROJECT PLANNING OR DEVELOPMENT: GRADING	UNDERGROUND BLASTING (TUNNELS)	SURFACE BLASTING	EXCAVATION OF CUTS	EMBANKMENT CONSTRUCTION	OPERATION	REMARKS
LAND SURFACE	EROSION	Estimate/define present susceptibility to erosion; use photography	•			•			Data collected in the course of normal preliminary field investigations should suffice
	CUTS	Not indicated			•	•			
	FILLS	Not indicated					•		
	SLOPE MOVEMENTS	Photogeologic interpretation of ancient unstable slope masses	•			•	•		
SURFICIAL SOIL	DENIAL OF AGRICULTURAL ACCESS	Map the extent of existing soil units	•				•		Data collected in the course of normal preliminary field investigations should suffice
	QUALITY	Perform index tests of soil agronomic properties	•			•	•		
SURFACE WATER	WATER QUALITY	Chemical analysis prior to construction	•			•	•	•	Observational approach can be applied toward accurate estimation
	WATER QUANTITY	Visual estimate of seasonal discharge	•			•	•	•	
GROUND WATER	WATER QUALITY	Chemical analysis prior to construction				•		•	Most difficult impact of those listed, to quantify; requires careful attention to indicators; in situ hydrogeologic tests are expensive and sometimes equivocal
	WATER QUANTITY	Visual estimate of seasonal well spring discharge and water level prior to construction				•		•	
AIR QUALITY	FUGITIVE DUST	On-site monitoring at intervals during construction	•						Can be controlled by Contract Specifications
	BLASTING AND FUMES	On-site monitoring at intervals during construction			•				
SOCIOLOGIC	NOISE	On-site monitoring at intervals during construction			•				Can be controlled by Contract Specifications
	VIBRATION	On-site monitoring at intervals during construction			•				
	AIR PRESSURE	On-site monitoring at intervals during construction			•				

(•) denotes that the stated parameter applies to the particular stage tial are available as a direct result of the standard geologic investigations.

I.4 IMPACT ON ABUTTERS

The extent to which an EIS should cover the area surrounding a project or to either side of a ROW should be determined primarily on the basis of geologic and hydrologic knowledge of the project area. Geologic estimates can be used to outline the areal extent of the various geologic constraints that may be activated by construction of the project, as well as the groundwater shadows produced by cuts along the ROW. The team hydrologic expert will be able to define the potential impact coverage related to modifications of those portions of individual watersheds through which the project passes and which are subject to modification by construction of the project.

I.5 PRESENTATION

As noted previously, overambitious or overdetailed studies to define potential environmental impacts are probably neither desirable or effective. The most important aspects of such studies are collection of essential information of the location and areal extent of specific impacts and descriptions of the rates and fluctuations expected to govern the nature of the impacts. Photographs and diagrams are essential for portraying the expected nature of the impact and maps should always be considered essential in explaining impact concepts to others. Existing US Geological Survey topographic maps, project-developed photogrammetric maps, or simple aerial photographic enlargements make excellent and suitable bases for representing the details of identified impacts. Data developed on any of these bases can also be transferred to screened bases used for other EIS purposes. Screened base maps are doubly effective due to the fact that topographic and planimetric detail become subservient in view of the geological detail developed developed during the EIS study. When reproduced in the EIS documents, care should be taken that the topographic and planimetric details remain legible and are not rendered undetectible through photographic copying or reduction.

I.6 REFERENCES

"Wyoming Highway Department Engineering Geology Procedures Manual, 1983." Cheyenne, Wyoming: Wyoming State Highway Department, 1983.

Zettinger, J. M., and Pendrell, D. L. "Design of Containment—Treatment System For Contaminated Groundwater Northwest Boundary, Rocky Mountain Arsenal, Colorado," Army Corps of Engineers, *Twentieth Annual Engineering Geology and Soils Engineering Symposium Proceedings,* pp. 63–82, Idaho Department of Highways, Boise, Idaho, 1983.