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SUSITNA HYDROELECTRIC PROJECT

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WATANA DEVELOPMENT

FIELD MANUAL

FOR

GEOTECHNICAL EXPLORATION OF THE FINS, FINGERBUSTER, POWERHOUSE AREAS

APRIL 1984

HARZA-EBASCO
SUSITNA JOINT VENTURE

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

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1.0 PURPOSE AND SCOPE

Field subsurface exploration programs are required in order to obtain detailed information on soil and rock conditions as a part of the Harza-Ebasco (H/E) effort to design the Susitna Hydroelectric Project (Exhibit 1) for the Alaska Power Authority.

This manual supplements the drilling contract plans and specifications by outlining field techniques and procedures which have been developed through years of trial and error by many different organizations experienced in underground explorations. These procedures are intended as a guide for H/E personnel who will monitor and supervise the Watana field explorations. All participating H/E personnel will be furnished a copy of the manual and the specifications. All participants are required to be familiar with their contents.

The objective of this manual is to outline the field exploratory techniques that are required to provide information for the design of the Watana Dam. In particular, the exploratory drilling program will provide information for evaluating the "Fins", "Fingerbuster", and powerhouse areas.

2.0 EXPLORATION PROGRAM

During the August 1983 site visit by the Federal Energy Regulatory Commission (FERC) Engineers, concern was expressed over the potential erodibility and seepage potential of the "Fins" feature. This feature is a zone of shears, fractures, and alteration located on the rock bluff (north bank of the Susitna River) just upstream of the intake for the diversion tunnels. The FERC Engineers reviewed rock cores and expressed particular concern over the disintegrated rock in the (COE) drill hole, DR-20.

Following their Anchorage visit, FERC formally expressed their concerns in regard to the "Fins" area in a letter dated September 23, 1983. Potential impacts on licensing were also discussed and further geotechnical exploration programs were recommended.

In a letter dated December 30, 1983 to FERC, the Power Authority outlined a Spring 1984 Geotechnical Exploration Program. This program is designed to provide feasibility level geotechnical information to support the license for the "Fins" area, as well as providing data in the area of the "Fingerbuster" and the underground powerhouse. The program consists of overburden and rock drilling, sampling, pressure testing, down-hole geophysics, and instrumentation installation in approximately 10 angle holes ranging in depth between 100 and 850 feet. The approximate location and depths for the boreholes are shown in Exhibit 2 and outlined on Exhibit 3. The exploration program is scheduled to commence on April 30, 1984 and conclude no later than June 15. A bar graph schedule outlining the pertinent program milestones is shown on Exhibit 4.

Borings will generally be drilled to the indicated depths using three rigs, each working two 12-hour shifts, daily. In certain instances, based on the information gained during development of the program, it may be possible to drill shallower or deeper than the indicated depths, and such holes may be varied at the direction of the Resident Geologist. Every effort will be made to carry the holes to the desired depths. The drilling contractor will be equipped for both overburden and bedrock drilling in various sizes, along with suitable casing, core barrels, and a variety of bits. The contractor is responsible for advancing the hole by the method he feels will bring the best results. If, however, unsuitable recovery or progress is being made, the contractor will be directed to use alternative methods of drilling or testing in order to obtain the best information under the prevailing circumstance.

3.0 STAFFING

3.1 GENERAL

During the implementation of the investigation program, an organization of personnel will function as indicated in Exhibit 5. This organization will monitor and guide the work, collect data, document the program activity and administer to the contract.

4.0 FIELD STAFF RESPONSIBILITIES

4.1 GENERAL

The H/E field staff (Exhibit 5) will consist of a Resident Engineer and a Resident Geologist and up to ten support personnel including one geophysicist. The Resident Geologist directs and the technical aspects of the program. The Resident Engineer coordinates contractor field activities with other field operations, and maintains contact with the Anchorage office contract administrative personnel. The Field Geologist will assist the Resident Geologist by reviewing all core and logs for content, accuracy and consistency. The Rig Geologist guides the drill rig operation to optimize the collection of data, directs sampling testing and instrumentation installation and prepares borehole log and shift reports.

4.2 RESIDENT ENGINEER

The following statements outline some general responsibilities and field procedures of the Resident Engineer:

A. Responsible for all field contractual aspects of the Watana Geotechnical Explanation Program. Maintain coordination with Lead Geotechnical Engineer on contractual matters.

B. Monitor and guide operational aspects of the field program. Coordinate with the contractor, government agencies and/or individuals to administer safety, permitting, and scheduling activities.

C. Coordinate with the camp manager to ensure that logistical support between H/E Anchorage office and field personnel is provided and maintained.

D. Utilize all available communication systems and provide reports to the H/E Anchorage office in order for them to keep current with the field operations from a contract standpoint.

E. Review and evaluate contractor time sheets and shift reports.

F. Maintain a field office which will have complete up-to-date progress records readily available for reference.

G. Maintain the drilling progress and costs.

H. Supports the Resident Geologist.

I. Will keep all personnel informed of any changes in the drilling contract as far as payment is concerned.

J. Coordinate with Anchorage Geotechnical Engineer in regard to requirements for supplies, project memoranda and personnel mail.

K. Coordinate the transfer of the rock core from the site to Anchorage by the drilling contractor.

4.3 RESIDENT GEOLOGIST

The following statements outline the general responsibilities of the Resident Geologist:

A. Responsibile for all technical aspects of the field activities associated with the Watana Geotechnical Exploration Program. Maintain technical coordination with the Lead Geotechnical Engineer.

B. Supervise and coordinate the activities of the field geologist, rig geologists and the geophysicist to insure an efficient flow of information from all the involved technical elements. Approve all borehole locations; all drilling, sampling, and borehole testing procedures.

C. Review and distribute logs and any other pertinent information necessary to keep the Anchorage design element up-to-date with the program.

D. Work with Resident Engineer to insure that proper storage facilities are provided for samples and/or core.

E. Visit the rigs in the field as frequently as necessary in order to become familiar with the project from the field standpoint as well as to provide assistance to the rig geologists.

F. Prepare a factual report (as the work progresses) outlining results of the program. The report will contain sufficient drawings and write-up so that upon completion of the program, the draft along with appropriate drawings can be quickly finalized in the Anchorage office.

G. Construct and maintain field drawings and X-sections as the work progresses.

H. Support the Resident Engineer in contractual matters.

I. Available at any time for consultation with Rig Geologists concerning current progress of hole being drilled.

4.4 RIG GEOLOGIST

The following statements outline the general responsibilities of the rig geologist:

A. The primary responsibility of the Rig Geologist (drill inspector) is to attain the best possible samples or core from the encountered horizons and to maintain them in the best possible condition to allow the maximum amount of design data to be extracted.

B. All geological data will be logged at the test hole site. The logs (example shown on Exhibit 6) should be complete, accurate and legible with copies submitted to the Resident Geologist at the conclusion of each shift. Items also included are observation device installation details, daily groundwater measurements etc.

C. Guide the drilling operations through coordination with the driller. The driller's experience and knowledge will be a definite asset and should be considered when decisions are being made. Although such things as daily drilling supply needs, spare parts requirements, sampling equipment, etc., are the driller's responsibilities, it behooves the Rig Geologist to be familiar with these matters in order to accomplish the goals of the drilling operation. It is, however, the responsibility of the Rig Geologist to bring any deficiencies to the attention of the driller as well as the Resident Engineer.

D. Safety on a drill rig is the concern of all individuals assigned to the unit. However, the Rig Geologist is generally in the best position to observe the overall operation and therefore, should be instrumental in instituting safety procedures on the drill rig. Safety cannot be overemphasized wherever people and machines are working together.

E. Cooperate with the contractor's representative in moving and locating equipment that will result in minimum disturbance to the surrounding environment.

F. Maintain continual contact with the Resident Geologist in order to determine if the maximum amount of engineering and geological data are being obtained at each hole location. Any surface or subsurface features at or near the drill site which will contribute information toward a complete evaluation of the particular area or project should be noted on the log.

G. Take daily notes in a field book of drilling activities including manpower, equipment and drilling operations utilized.

H. Thoroughly familiar with the drilling contract and pay items, as field notes in regard to the drilling activities will be the basis for determining payment of the contractor.

4.5 FIELD GEOLOGIST

The following statements generally outline the responsibilities of the Field Geologist:

A. Receive samples or core upon completion of shift and review all boring logs (Exhibit 6) for completeness and legibility.

B. Using copies of the Rig Geologists boring logs, review recovered core for continuity and accuracy. Differences with the field log will be noted in red pencil and passed on to the Resident Geologist and the Rig Geologist.

C. Compilation and interpretation of all geotechnical data to include prepartion of drill hole test data and geologic profiles. Summarize the program results as the work progresses.

D. Photograph the core (with scale).

E. Assist the Resident Geologist in the preparation of the "Exploration Report".

4.6 GEOPHYSICIST

Down-hole geophysical techniques may be used to maximize subsurface information. Geophysical applications will be programmed by the Resident Geologist as the field work progresses.

A. Direct and perform the geophysical logging of the drillholes.

B. Assist in the preparation of the Exploration Report.

C. Perform other geological duties as assigned by the Resident Geologist.

5.0 BOREHOLE LOGGING TECHNIQUES AND PROCEDURES

5.1 GENERAL

The purpose of the borehole log is to enable the design engineer interpreting the log to visualize the core samples (as seen by the compiler of the log) and hence to draw inferences on the likely behavior of the actual foundation mass. Only those parameters which are significant to the rock mass or give a better understanding of the general geology of the site should be recorded.

Whenever possible, classifications should be defined using simple field tests with equipment commonly carried by the core logger (knife, geological pick, etc.).

Most of the boreholes start in soil or pass through soil strata. Descriptions of these soil horizons should be made from the disturbed but intact core samples to form on intergal part of the borehole log.

The bedrock core description includes not only the description of the rock mass but great emphasis should be given to the discontinuity surfaces that occur through it and the fracture filling materials. It is recognized that while discontinuity surfaces and their filling are of great importance to rock mass behavior they are often given a secondary role in the rock core description. A core description comprising four parts is therefore necessary:

(1) The primary descripton is that of the parameters affecting the basic rock mass properties.

(2) To supplement the above a description of the discontinuity surfaces should be given.

(3) A description of any fracture filling should be indicated.

(4) Drilling information; abrupt change in rate of penetration, loss of drilling fluid, equipment reaction, etc.

The log should be a factual description of the samples or core. Interpretation or assessments on the part of the core logger should be made on the log form in the remarks column only.

It is recognized that assessments and interpretations are best made when actually looking at the samples or core in its fresh and least disturbed state. The logger is therefore often in a very favorable position to make such assessments and interpretations and to exclude these from the borehole log is to reduce the value of the information. Such interpretations or assessments should be included in parentheses, in the remarks column of the log.

5.2 LOG DESCRIPTIONS

A legible, concise, and complete record of all significant information pertaining to the drilling and sampling operations within each borehole must be maintained concurrent with the advancement of the hole by the Rig Geologist assigned to guide the operation. The record must contain all information available to define the subsurface geology, groundwater, and thermal conditions. The log (Exhibit 4) should be a complete record of the drilling and sampling operations. Among the information to be recorded is the following:

A. Reference information comprising the project number, title, and location; the borehole designation; the exploraton location by coordinates, the inclination of the boring and, the bearing or azimuth of the hole; the reference elevation, that is the elevation from which all depth measurements are made;

B. Personnel information including the name of the drilling contractor, the driller, and the rig geologist. Notes on the log should include dates hole started and date complete., depth and date at shift change, etc.

C. Equipment data consisting of the manufacturer's name and model designation for the drill rig, and pressure testing equipment; method of drilling, core diameter etc.

D. Sampling and coring information consisting of the following:

(1) For all sampling or coring operations: the sample type and number; the depth at the start and at the completion of the coring "run"; the length of bedrock sample or core recovered; recovery is defined as the ratio, expressed in percent, of the length of rock sample or core recovered; to the length of the sampling drive or push or coring "run"; and a complete visual descripton of each sample or core including color, type of material, density, or consistency of soil, hardness of rocks, stratification, rock structure, moisture conditions, etc. The description should be made immediately following the retrieval of the sample or core so that it represents the "as retrived" classification. This is particularly important when sampling materials which tend to break down on exposure.

(2) For soil or rock sampling, insure that the driller records average rotational speed and downward hydraulic pressure of the core barrel and the average rate of penetration (run length/time) should be noted.

(3) Description of material penetrated but not sampled as determined from drilling or chopping action or changes in the color of the drill water or equipment reactions.

(4) Insure that the driller records casing information consisting of the size of the casing; the depth at which casing was added; the length of casing added; the final depth of the bottom of the casing, etc.

(5) Pressure test information comprising the depths at which tests were performed and the time required for each test. The actual test data is recorded on forms for that purpose. (Exhibit 15)

(6) Groundwater information consisting of the depth to the water surface in the hole, recorded daily, at the start and close of each shift. These readings should be continued after completion of the hole until the water level in the hole has stabilized.

(7) Artesian pressure information including the depths at which artesian pressures were encountered during drilling, the measured heads, and the time at which each measurement was made.

(8) Elevation of the top and bottom of the hole and the top of the bedrock.

(9) Insure that the driller records date and time of operations and delays including, but not limited to, drilling, sampling, permeability and pressure testing, artesian pressure measurement, machine breakdown, injuries, etc.

(10) Miscellaneous information which may aid in the interpretation of subsurface conditions. This would include the depth at which drill water is lost or regained, the amount and color of the return water, and the depth at which a change in drilling action occurs. The latter would include the depth at which rod vibration starts or stops, the depth at which the rate of penetration or ease of penetration changes, etc.

(11) Any additional information which the driller, or Rig Geologist considers pertinent to the interpretation of subsurface conditions.

E. Core Logging Sequence

(1) Roughly piece together the core while still in the liner and mark all mechanical breaks and the end of the run with a magic marker (the end of the run may not be evident until the next run is recovered).

(2) When logging the bedrock core include lithologic and structural descriptions; determine recovery and RQD values for the run.

(3) Remove core from liner and place in core box.

(4) Mark with a magic marker all hammer breaks in bedrock.

(5) Place run, depth and core loss blocks where appropriate.

(6) Mark core box identifying depth of core and box number.

F. Miscellaneous

A very complete description of any existing groundwater is imperative. A written description indicating whether or not it is a true or perched water table, how much water there is, any artesian characteristics or any other useful groundwater information should be included. The presence or absence of water in soils is extremely important when evaluating engineering data.

Such general statements regarding caving problems, drill penetration rate (drill easy, hard, very hard, etc), reason for no sample recovery, hydraulic pressure used for bit penetration into the ground, drill reaction (drills rough, drills smooth), bit usage, termination of hole for unusual circumstances or any other details pertinent to a complete evaluation of the test hole, should be written down.

The design personnel who make the final evaluation of any project have to rely upon clear, concise, accurate and complete written information. Memories of a geologist they may never see or talk to are of no value when the final decisions and recommendations are made.

5.3 BEDROCK CLASSIFICATION AND DESCRIPTION

This section discusses the procedures for identifying rock type and describing rock characteristics. In general, the bedrock description should cover rock type, weathering, fracturing, hardness, color, grain size, and strength.

5.3.1 Bedrock Characteristics

Based on previous geotechnical explorations for the project, the geology at the site is comprised primarily of a diorite pluton and an andesite porphyry flow. These rocks have been intruded by relatively narrow felsic and occasionally mafic dikes. The rock is generally hard, strong, and fresh except within shear, fracture, or alteration zones (see Appendix H).

A - Hardness

Hardness, which is a function of lithology, is an estimation of the strength of intact rock. Generally, hardness is not as important as the discontinuities within the rock mass; however, radical variations in hardness from typical values can significantly affect the behavior of the rock. Descriptive terms (such as hard, soft, etc.), rather than numerical values, are used in the field to describe rock strength. A relative scale of hardness is shown in Exhibit 8. This scale is based on simple field tools: a knife blade and a geologist's hammer.

B - Weathering and Alteration

Weathering (Exhibit 9) is the process by which rocks are changed in character (color, texture, composition, hardness, and or form) through mechanical, and/or chemical action. Mechanical weathering results in the opening of discontinuities. Chemical weathering (generally the result of rainwater) leads to chemical changes or solutioning of the original minerals and often results in discoloration (for example, iron oxide staining) of the rock material.

Alteration (Exhibit 9) is the process by which changes in the chemical or minerological composition of a rock are produced by weathering or hydrothermal solutions. Like chemical weathering, alteration may cause a discoloration of the rock mass. Kaolinization (altering of feldspar to clay) is a typical example of alteration.

It is often difficult to distinguish between the effects of weathering and alteration. Both may affect rock strength and discontinuity spacings. As with rock hardness, it is the relative degree of weathering or alteration which should be noted. A list of descriptive terms for identifying the degrees of relative weathering or alteration is shown in Exhibit 7.

C - Discontinuity Surfaces

The engineering behavior of rock masses is often controlled by the discontinuity surfaces which occur within them. Discontinuity surface frequency or spacing is often the most effective feature to convey the effect of the discontinuities on rock mass behavior. It is therefore selected for inclusion in the rock mass description. It is not necessarily the most significant feature controlling rock mass behavior of any particular type. The extent of joints and their separation may control seepage capability while orientation and fracture filling may be more significant to shear strength.

The physical properties of the discontinuity is described in the core log as follows:

1) Separation
2) Filling
3) Roughness
4) Orientation
5) Fracture Filling

In view of the disturbance of fracture filling and core orientation in the drilling process and the limited extent of any discontinuity surface exposed in the core, the accuracy or validity of the discontinuity surface descriptions made from core is often dubious. The use of special drilling techniques (core orientation) or exploratory methods (down-the-hole periscope or cameras) may be warranted.

The primary description of the rock mass properties may be followed by a description of the fracture filling. Where individual discontinuity surfaces are recognizable as significant features, such as faults or shear and fracture zones, they may be described singly. Where they are observed to fall into distinct groups, that is, joint sets, with distinctly different group properties, each group may be described separately.

The extent and complexity of the discontinuity surface description is at the discretion of the core logger. Sufficient detail should be included to enable a valid assessment of the rock mass behavior to be made for the specific engineering problem at hand, but unnecessary and often costly detail should be avoided. It should be born in mind that the cost of logging is small by comparison with the cost of recovering the core and that the core log will probably be utilized as a reference by design engineers rather than the core.

Discontinuity surfaces are any surfaces across which there is a discontinuity of physical properties. Only those surfaces which have occurred as a result of natural geological processes are described. Fractures resulting from the drilling process or subsequent to core removal from the borehole are not described, but are marked with a magic marker. Mark the break with a line connecting the two pieces after the core has been pieced together.

The properties of a discontinuity surface (extent, separation, fillings, roughness, waviness and orientation) are often characteristic of its origin. Recognition of surface type in terms of its origin can therefore be extremely useful. For example, bedding planes clearly have a much greater extent than cross joints in the same sedimentary rock.

(1) Separation

The separation between fracture surfaces controls the extent to which the opposing surfaces can interlock. In the absence of interlocking of the fracture walls, the fracture filling controls entirely the shear strength along the fracture. As the fracture separation decreases the rough edges of the fracture walls tend to become more interlocked and both the filling and rock material contribute to the shear strength. The shear strength along the fracture is therefore dependant on the degree of separation, presence or absence of filling materials, nature of asperities (roughness of the fracture walls) and the type of filling material.

The effect of degree of separation and filling can be classified as follows:

(a) Tight - With no separation (no filling) the sliding plane passes entirely through or along wall rock: the shear strength is entirely dependant on the properties of the wall rock.

(b) Slight separation (filling appears as a stain): The filling or separation is considered only as modifying the friction angle. (tight)

(c) Part Open - Appreciable separation (filling of measureable thickness) but still appreciable interlocking of the wall asperitics: The shear strength will be a complex combination of filling and wall rock material strengths.

(d) Open - Complete separation with no interlock of wall asperities: the filling material determines the fracture shear strength.

(2) Fracture filling (Presence or Absence)

All materials occurring between the fracture walls are referred to as fracture filling. The term includes in-situ weathered materials, fault gouge and breccia and foreign materials either deposited or intruded between the fracture surfaces.

Only the presence or absence of fracture filling is noted in the discontinuity surface description. Where applicable a separate description of the fracture filling is given after the discontinuity surface description.

(3) Roughness

Rough edges which occur on fracture walls interlock, if the fractures are clean and closed, and inhibit shear movement along the mean fracture surface. This restraint on movement is of two types. Small high angle asperities are sheared off during shear displacement and effectively increase the peak shear strength of the fracture (asperities are termed roughness). Large low angle asperities cannot be sheared off and "ride" over one another during shear displacement, changing the initial direction of shear displacement. Such large order asperities are termed waviness and cannot be reliably measured in core. A classification of roughness is presented in Exhibit 10.

Where slickensides are observed the direction of the slickensides should be recorded after the standard discontinuity surface description.

(4) Orientation

There are at present a number of specialized methods that can be used to obtain the dip and dip direction of discontinuity surfaces in drill core. One method is to remove an orientated core from the rock mass using a special core orienter barrel. Alternatively the discontinuity surface orientation can be measured in the wall of the borehole using an orientated borehole camera capable of photographing the borehole sides. Where a feature of known dip and dip direction, i.e. bedding, intersects the core at an angle, this may be used to orient the core. A further method requires the presence of at least on easily identificable marked band and the use of a minimum of three boreholes. This latter method enables three dimensional geometry, usually aided by sterographic projection, to be used to establish the attitude of the marker horizons. The above methods are costly and are only employed where the attitude of the discontinuity is critical to the solution of the problem.

The dip direction is the compass bearing, from true north, of the direction of maximum dip and is recorded in degrees (e.g. 045° and no NE) measured clockwise from north.

A complete definition of the orientation of any one surface is given by recording the dip and dip direction, for example: 30° at 036° would indicate a surface dipping at 30° form the horizontal with the direction of maximum dip having a orientation of 36° measured clockwise from true north.

Discontinuity surfaces usually occur in sets. It is the definition of a number of sets and their relationship to each other that is necessary for design purposes. The definition of these sets and their orientation is simplified if all the field readings are plotted on a stereogram. The sterographic plot enables the distribution of individual discontinuity surfaces to be seen and permits the definition of discontinuity surface sets and their orientation.

Examples of discontinuity surface descriptions are as follows:

(a) Bedding joints are part open, fe oxide stained, slightly rough; dipping 30° at 145°.

(b) Set "A" cross joints are tight and clean, rough, dipping 10° at 270°.

(5) Description of the Fracture Filling

The influence of fracture filling is two-fold.

(a) Depending on the thickness, the filling prevents the interlocking of the fracture asperities (see roughness Exhibit 10)

(b) It possesses its own characteristic properties, that is, shear strength, seepage characteristics and deformational characteristics.

To determine the effect of the fracture filling on the rock mass the moisture content, color, consistency of hardness, rock type and origin of the infilling materials should be adequately described.

It should be remembered that the drilling technique employed to recover rock cores may not be suited to the recovery of relatively thin bands of softer material within the rock mass. Recovery of fracture filling may therefore be only partial and recovered material may be disturbed. Where drilling muds or fluids are used, the filling materials may be contaminated and the moisture conditions could be altered.

5.3.2 Bedrock Quality (RQD)

The Rock Quality Determination (Deere, et al., 1969) method of determining rock quality is as follows:

Sum up the total length of core recovered in each run, but count only those pieces of core which are four inches (10 cm) in length or longer and which are hard and sound. The sum is then represented as a percentage over the length of the run. If the core is broken by handling or by the drilling process, the fresh broken pieces are fitted together, marked with a magic marker, and counted as one piece provided that they form the requisite length of four inches (10 cm).

Relation of RQD and Rock Quality

RQD (%)	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

5.3.3 Bedrock Logging Terms

The order of rock descriptions should be the same throughout a log and follow the sequence presented in the example log in Exhibit 6. The description is comprised of two parts; the lithology and physical properties of the rock; and the structural properties of the rock.

In describing bedrock, the footage interval of the rock unit is identified followed by the rock name which is capitalized. The description of the physical properties; should be presented in the following order: weathering/alternation, hardnesss, strength, texture, color, mineralogical composition (%), and alteration products.

A blank space should be left between the lithologic and structural descriptions. The structural description will start with the degree of fracturing for the overall interval, and be followed by descriptions of the spacing of discontinuities, filling or stain, separation, roughness, angle of discontinuities to core axis (range), and maximum, minimum, and average size pieces. Common abbreviations used in logging are presented in Exhibit 7.

In addition, shear and fracture zones within an interval will be "called out" on the log, using indented paragraphs. The description will identify the footage interval, type of discontinuity (capitalized), composition (gouge, breccia, slickensides, etc), weathering or alteration, fracture spacing; moisture density, and plasticity if applicable, and color. See Exhibit 8 and 9. General comments should include amount and type of ice present and any notes on the groundwater conditions if applicable.

5.4 SOIL CLASSIFICATION

A. Introduction

Varying thicknesses of overburden will be encountered in most of the proposed boreholes. Soil or soil like materials will be penetrated in the process of completing any or all of the boreholes.

The identification and description of soils will be based on visual inspection of the retrieved samples using the Unified Soil Classification System. These procedures will be followed by the Rig Geologist and incorporated with any other pertinent field information such as amount of cobbles and boulders, zoning, layering, etc. The field logs (Exhibit 6) will be verified by laboratory tests as required, and corrections and additions will be made to the field logs by the Field Geologist prior to final preparation. All identifications and descriptions should be as comprehensive and as precise as possible under field conditions. A correct overall impression of the soil should be conveyed without excessive emphasis on insignificant details.

B. Unified Soil Classification System

The Unified Soil Classification System takes into account the engineering properties of soils; it is descriptive and easy to associate with actual soils; and it has the flexibility of being adaptable both to the field and to the laboratory. Probably its greatest advantage is that a soil can be classified readily by visual and manual examination without the necessity for laboratory testing. The Unified Soil Classification System is based on the size of the particles, the percentage of the various sizes, and the characteristics of the very fine grained material. The overall or average characteristics of soils, defined in terms of gradation and plasticity, are represented in the Unified Soil Classification System as shown in Exhibits 11 through 13. A very comprehensive coverage of this subject is contained in the USBR Earth Manual (Appendix D)

The first step in classification is to determine whether a soil is either predominantly coarse-grained or predominantly fine-grained. A coarse-grained soil will have greater than half of the material visible as individual grains to the unaided eye. A fine-grained soil will have greater than half of the material not visible as individual grains to the unaided eye. The No. 200 sieve size is about the smallest particle vissible to the unaided eye. For all classifications of soils, no particles larger than three inches in size are included. Once the soil is classified as coarse-grained or fine-grained, the classification proceeds as per Exhibit 11. For fine-grained soils, the toughness, dilatancy and dry strength are determined as shown in Exhibit 12.

C. Soil Description

The in-place condition of soil assumes primary importance in soil classification. The description must present a complete word picture of the soil as it exists in the foundation, in addition to assigning a name and proper group symbol. The soil is again divided as coarse-grained soils and fine-grained soils as required by the Unified Classification System.

- Coarse-Grained Soils

Coarse-grained soils should be described using the items in Exhibit 11 when applicable. This information should be recorded on the boring logs.

- Fine-Grained Soils

Fine-grained soils should be described using the items in Exhibits 12 and 13 when applicable. This information should be recorded on the boring logs. The consistency of cohesive soils may be determined in accordance with the identification procedure given in Exhibit 11.

5.4.1 Overburden Sampling

The existance of very dense soil, boulders, and fragmented rock anticipated to exist in the overburden angle hole drilling precludes the use of normal soil sampling methods. Therefore double tube core barrel techniques will be used in the investigations for obtaining soil samples. Advancement of the cased holes through overburden will be accomplished by continuous sampling to bedrock prior to rock core drilling. If soil core samples are washed away by the drilling process, limited identification of soils can be made from wash cuttings. Fine materials in the cuttings may be lost by integration with the water for circulation and coarse materials will experience a considerable time lag before reaching the ground surface even when heavy mud is used as the drill fluid. Therefore the in-situ depth of such materials is always suspect. Nevertheless, information can be obtained by careful sampling of the return wash water and monitoring of the cutting action and rate of bit penetration. Every attempt will be made to optimize information from the overburden drilling operations.

5.4.2 Soil Logging Procedure

All data related to drilling and sampling should be recorded on the drill log. (Exhibit 6) These forms are to be filled in completely, as indicated in the procedure below:

Before drilling of a hole begins, all information at the top of the page should be completed, such as project, site, hole no, weather, contractor, etc. Depths are to be marked in the appropriate column. The actual soil description is logged in the following order: first, the major soil type, i.e., sand, silt, clay, etc,. In coarse-grained soils, the particle size should be included, i.e., coarse sand, or fine to medium gravel.

For coarse grained soil types, the lesser constituents should be added as a prefix for percentage of fines less than 12% and for coarse fractions greater than 25%.

Once the soil components are identified, other descriptions such as relative density or consistency, moisture, color, structure, cementation, geological origin or local name (Exhibit 14 and 15). The Unified Soil

Classification system Group Symbols are to be included as described previously, only when the visual classification has been verified by laboratory tests.

Example:
Lean Clay, v. dense, stiff, moist, Brown, 10% Gravel

6.0 GROUNDWATER SAMPLING

6.1 GENERAL

Groundwater samples will be taken for chemical analyses for the purpose of ascertaining the distribution, movement, and overall groundwater conditions. Water quality testing of the samples will include conductivity, pH, total dissolved solids, and the concentrations of Na, Ca, Mg, K, CO_3, HCO_3, SO_4, Cl, NO_3 and B. Samples of the drill water shall be taken each time a groundwater sample is obtained.

Water samples will be taken after completion of the hole and prior to hydraulic pressure testing. The drill holes shall be bailed or pumped with water to remove any drill fluids from the hole which may affect the permeability of the rock. The bailer should be flushed with distilled water prior to obtaining the sample. The samples shall be collected and transported to the laboratory in one quart polyethylene bottles.

The bottles shall be cleaned with distilled water prior to use. All bottles should be completely filled to minimize the amount of air in each bottle. When taking a sample, personnel shall wear disposable polyethylene gloves in order to minimize contamination.

The samples will be transported to the camp by the first available means, and refrigerated until they are transported to a laboratory.

7.0 HYDRAULIC PRESSURE TESTS

7.1 GENERAL

Tests in which water under pressure is forced into rock through the walls of boreholes provide a means of determining seepage characteristics of the bedrock mass.

7.2 APPARATUS

The apparatus used for pressure tests in rock is illustrated schematically in Exhibit 16. It comprises a water pump, a manually adjusted automatic pressure relief valve, pressure gauges, a water meter, and a packer assembly. The packers, (Exhibit 16) which provide a means of sealing off a limited section of borehole for testing, should have a minimum length five times the diameter of the hole. They may be of the pneumatically or mechanically expandable type. The former are preferred since they adapt to an oversized hole whereas the latter may not. The piping of the packer assembly is designed to permit testing of either the portion of the hole between the packers or the portion below the lower packer. The packers are normally set 2, 5, or 10 feet apart and it is common to provide flexibility in testing by having assemblies with different packer spacings available, thereby permitting the testing of different lengths of the hole. The wider spacings are used for rock which is more uniform; the short spacing is used to test individual joints which may be the cause of high water loss in otherwise tight rock.

7.3 TEST PROCEDURE

The test procedure used depends upon the condition of the rock. The packer spacing will be determined after review of th log and core. In rock which is not subject to cave-in, the following method is in general use. After the borehole has been completed it is filled with clear water, surged, and washed out. The test apparatus is then inserted to the bottom of the hole and upon completion of the test, the apparatus is raised a distance equal to the test interval and the test is repeated. If the rock in which the hole is being drilled is subject to cave-in, the packer system should be lowered to the bottom of the hole through the drill rods. The drill rods should be extracted up to the maximum permissible unsupported length of hole on a distance equal to the test interval, whichever is less. When abnormal gain or loss of drill water is observed, or caving of the hole occurs during drilling, it may be required that drilling be discontinued and the hole be pressure tested. Back pressure should be determined on the basis of depth below the water table, sealing pressure and maximum injection pressure (see example on Exhibit 17).

Regardless of which procedure is used a minimum of three pressures should be used for each section tested. The magnitude of these pressures is commonly 15, 30, and 45 psi above the natural piezometric level. However, in no case should the excess pressure above the natural piezometric level be greater than 1 psi per foot of soil and rock overburden above the upper packer. This limitation is imposed to insure against possible heaving and damage to the foundation. In general, each of the above pressures should be maintained for 10 minutes or until a uniform rate of flow is attained, whichever is longer. If a uniform rate of flow is not reached in a reasonable time, the geologist must use his discretion in terminating the test. The quantity of flow for each pressure should be recorded at 1, 2, and 5 minutes and for each 5-minute interval therafter. Upon completion of the tests at 15, 30, and 45 psi the pressure should be reduced to 30 and 15 psi, respectively, and the rate of flow and elapsed time should once more be recorded in a similar manner. Should leakage occure (poor seal of packers) the packer should be adjusted 1-2 ft to achieve a better seal.

Observation of the water take with increasing and decreasing pressure permits evaluation of the nature of the opening in the rock. For example, a linear variation of flow with pressure indicates an opening which neither increases or decreases in size. If the curve of flow versus pressure is concave upward it indicates the openings are enlarging; if convex, the openings are becoming plugged. Additional data required for each test are as follows: (1) depth of hole at time of each test ; (2) depth to bottom of top packer; (3) depth to top of bottom packer, (4) depth to water level in borehole at frequent intervals; (5)

elevation of piezometric level; (6) length of test section; (7) radius of hole; (8) length of packer; (9) height of pressure gauge above ground surface; (10) height of water swivel above ground surface; and (11) description of material tested. Test data in bedrock will be reported using Lugeon Units (see Appendix G)

Hydraulic pressure test data should be summarized on forms as outlined in Exhibit 18.

8.0 DRILL HOLE PERMEABILITY TESTING IN OVERBURDEN

8.1 GENERAL

There are two basic types of permeability tests which can be performed in boreholes in soil. The first type is the pumping-in type test; based on measuring the amount of water accepted by the ground through the open bottom of a pipe or uncased section of hole. Types of pumping-in tests commonly performed include falling head tests. The second basic type of permeability test is the pumping-out test; based on measuring the amount of water flowing into the hole through the open bottom of a pipe or uncased section of the hole. The most common pumping-out test is the rising head permeability test.

If drilling muds, fluids, or other additives have been used in advancing a hole to be tested, the hole will have to be flushed and thoroughly cleaned prior to the start of the field test. Clear water should always be used. The presence of even small amounts of silt and clay in the added water will result in plugging of the tested area and give questionable results.

8.2 FALLING HEAD TEST

The falling head permeability test is a pumping-in type test. This test may be performed in a casing sealed at the bottom of a hole or in an open hole. The flow of water into the hole, typically measured by the drop of water level in the casing or open hole, is monitored over a period of time. The permeability may then be readily calculated from the formulas in Exhibit 19 and 20 NAV-FAC DM7.

In each case a hole is advanced through the soil to the desired depth. A casing may then be driven and seated into the soil at the bottom of the hole to prevent seepage along the casing. The inside of the casing should be carefully cleaned out to remove any cuttings. To perform the falling head test, the hole is filled with clean water typically to the top of ground or top of casing and is then monitored over a period of time. At the beginning of the test, when the rate of the water level drop is usually fastest, the level is measured at small time-intervals, usually 30 seconds or 1 minute. As the rate of the water level slows, the time interval between measurements is gradually increased until the water level has stablized.

8.3 RISING HEAD TEST

The rising head permeability test is a pumping-out type test. It is very similar to the falling head test in that it may be performed in a casing sealed at the bottom of the hole or in an open hole. The difference is that the rising head permeability test must be performed below the ground water table. The testing procedure is as follows. After the hole is excavated and the casing, if any, is seated, the static ground water level is allowed to stablize in the hole. After the level has stablized, the initial depth to water is measured below some reference, usually the top of casing or the top of ground. The water level in the hole is then lowered by pumping or bailing to a level as low as can be reasonably achieved. The water level immediately after pumping or bailing is measured, and the rise of the water level in the hole is then continually monitored. In the beginning of the test, when the rate of water level rise is usually fastest, a close time-interval is used between readings. The faster the rate of rise, the closer the time intervals, usually from 10 seconds to 1 minute. As the water level rises and the rate of rise slows, the time interval between readings is increased until the water level reaches the initial level before pumping or bailing.

9.0 OBSERVATION DEVIDE INSTALLATION

9.1 GENERAL

Two types of instrumentation are installed in the boreholes to monitor the hydrologic and thermal subsurface conditions of the soil and rock. Groundwater conditions may be monitored using either, a standpipe or remote sensing piezometer. The thermal conditions of the ground may be monitored by using thermal probes or thermistor strings. Decisions will be made in regard to type and placement of observation devices based on an appraisal of the drill hole data by Anchorage design engineers. For this program, groundwater will measured by a dual installation of a standpipe and remote sensing piezometers.

9.2 STANDPIPE PIEZOMETER

The following operational procedures should be used in the installation of standpipe piezometers in open boreholes. Exhibit 19 indicates a typical installation for a standpipe or open tube piezometer.

When possible, only clear water should be used when drilling in the vicinity of the hole depth that will contain a piezometer. Care should be taken to avoid the introduction of oils, greases, or drilling muds as their presence can change the permeability of the in-situ soils.

Generally, the riser pipe is supplied in 10-foot sections that are assembled as the string is lowered into the hole. The pipe joints will be screwed together by the use of metal couplings. Teflon tape will be used over the male pipe threads. Prior to and during assembly, the rods will be tied off to the rig to hang suspended above the bottom of the hole to avoid damage or contamination of the tip.

The next step in the installation is the tremie placement of a "sand pack" around the tip (area of perforations). This is done by adding the specified sand (usually No. 40 Ottawa sand) to the hole as the tremie pipe is raised allowing the sand to settle around the tip. This operation should be done very slowly so that "bridging" of the sand in the tremie pipe does not occur. Adding water with the sand is oftentimes beneficial to facilitate a smooth flow of sand into the hole.

After the sand has been placed, bentonite pellets should be very slowly added to the hole by a tremie pipe which is constantly raised above the placement elevation. Usually, a 2-foot-thick plug is satisfactory, but more can be used to insure a pressure-proof plug. Allow a reasonable length of time before grouting to let the bentonite pellets swell and seal the hole. Note that no tamping is done for either the sand or bentonite pellets. A 1:1 grout mix (cement to water) should be placed in the hole from the bottom up (tremied).

A. Fittings

Recommended fittings for the plastic standpipe piezometer pipe are made of brass or stainless steel. These fittings can be made up slightly more than hand tight and will be leakproof when joined to the plastic pipe with teflon tape. A surface casing and cap is installed at the surface to mark the hole and protect the instrumentation.

B. Anti-Freeze

Because of the cold air temperatures and the existance of permafrost at the site, antifreeze is needed in standpipe piezometers if groundwater levels are at or near the ground surface. This is one of the most trouble causing items in the open tube standpipe system. Kerosene when used as antifreeze in a standpipe system containing a pressure gage (artisian condition), produces, totally unreliable readings. This is due to absorption of the kerosene by the plastic pipe. A vacuum or a lower positive reading is produced in the system depending on the length of the pipe.

In a standpipe system, without a pressure gage, kerosene is suitable because it is open to atmospheric pressure and the amount absorbed by the pipe is not critical for a reasonable period of time. Kerosene has a specific gravity approximately eight-tenths that of water and the differential must be computed to accurately estimate the piezometric level. It is essential to know the amount of kerosene in the vertical system, otherwise readings will be invalid. Kerosene can be put in a standpipe piezometer by:

> A length of PVC plastic pipe is used to displace the water to a depth at least fifteen feet below the ground surface within the riser pipe. After it is inserted into the standpipe displacing the water, it is then withdrawn. An M-scope ohmeter is then used to determine the water level and the kerosene added. This gives the elevation of the water and kerosene line.

Methanol (wood alcohol) anti-freeze has also proved to be satisfactory. It shows no significant reaction with plastic pipe. Mixing about one-third methanol and two-thirds water gives a freezing point of -9°. With this mixture the specific gravity is close enough to water to be ignored. For lower freezing points the methanol and water can be mixed as desired.

In using antifreeze, the following points should be remembered:

- If the piezometers are allowed to overflow, the anti-freeze will be lost and freezing will occur.
- Methanol is poisonous! Never use mouth suction.
- Water level measurement. Groundwater measurements in standpipe piezometers should be made with an M-scope and recorded on a form as outlined in Exhibit 22.

9.3 CLCSED SYSTEM PIEZOMETER (Remote Sensing)

Advancement of the hole will be accomplished by the same drilling methods outlined for the standpipe piezometer. Once the hole has been drilled to the desired depth without the use of drill mud or other contaminents, the closed system pressure cell (Exhibit 23) may be installed using the following procedure:

1. Tremie enough bentonite pellets to equal ± 2 feet at the location of the installation.
2. Measure depth to top of pellets after waiting for them to settle.
3. With tremie pipe still in the hole lower pressure cell to a point 2 feet above bentonite.
4. Measure enough Ottawa sand to fill about 3 feet of hole. Pour this sand while raising the tremie pipe and allow to settle around the pressure cell and above.
5. The upper seal is now made by using the tremie pipe to place approximately 2 feet of bentonite pellets. (These pellets will drop like marbles before expanding in the water to form a tight seal without tamping.)
6. The grout backfill can now be raised to the top of the hole or to a point where another pressure cell (Exhibit 23) is to be placed.
7. Note that no tamping is performed in any of the installation sequencies for a closed system piezometer.
8. A surface casing with a cover is installed at the surface which will protect the instrumentation.
9. A typical multiple installation for remote sensing piezometers is shown in Exhibit 24.

9.4 THERMAL PROBE IN SHALLOW BOREHOLES

Installation of Pipe

Materials Required for Installation:

- o Sand backfill and glycol antifreeze
- o Adequate length of 1 inch P.V.C. Pipe, in ten foot (10') sections, caps and couplings for the depth indicated
- o Adequate #1 Solvent and #2 Cement for the Above
- o Hacksaw
- o Pliers
- o One can of orange spray paint

Assembly Instruction:

1. Clean one end of tubing with #1 Solvent, then liberally wet that end with #2 cement and place a cap over end with a twisting motion. (The joints must be kept warm)

2. Follow instructions for joining P.V.C. pipe with couplings.

3. Whenever joining sections of pipe into twenty foot (20) or longer sections, allow at least (5) five minutes undisturbed drying time for each coupling. (Joint must be kept warm while drying)

4. Two (2), ten (10') foot sections of pipe can often be assembled before the pipe is installed in the hole.

5. If the length of pipe will exceed twenty (20') feet, a light nylon cord must be attached to the bottom of the first section by several turns of black electrical tape. This line is paid out as sections are added to facilitate the joining of additional sections and to reduce the stress on previously cemented couplings which may not have developed sufficient strength to sustain the weight of the pipe while it is being installed.

6. Allow approximately three feet of one inch P.V.C pipe to extend above ground surface or within 6 inches of the top of the surface casing.

7. Cut a notch in the top of the pipe which will allow a thermistor probe to be led out below the cap.

8. Fit an uncemented cap over the exposed end of the pipe.

9. Fill the annular space between the P.V.C. pipe and the drill hole evenly with a dry fine-grained (#10 screen) sand which is tremied into hole.

10. A surface casing with a cover is installed at the surface to protect the instrumentation and painted with orange point, this completes the installation. It is important that a good record be made of each installation including such items as location, test hole

number, date, depth and stickup of the PVC pipe and any other data which may be helpful in interpreting the data from that particular installation. The borehole thermal profile can now be monitored using a thermistor probe which is lowered and read at 5.0 foot intervals from the ground surface.

9.5 THERMISTOR STRINGS IN DEEP INCLINED BOREHOLES

Tools Required:

- o Measuring tape (10-foot)
- o Hand Drill and twist bits
- o Hacksaw and blades
- o Knife
- o Pipe clamp for 3/4" O.D. PVC pipe
- o 20-foot tripod to aid installation
- o Pulley for top of tripod
- o 60 feet 1/2 inch rope
- o Grout pump
- o Grout hoses and hose clamps
- o Grout mixing tools (shovels clamps)
- o Grout tub to mix grout in

Supplies:

- o Thermistor string
- o 3M electrical tape
- o 3/4 inch O.D. PVC pipe (threaded joints)
- o PVC cleaning solvent
- o PVC Cement
- o Portland cement if no permafrost is expected
- o Halliburton Permafrost Cement if permafrost expected
- o Water to mix with cement

Assembly Instructions:

1. Clean and join the PVC pipe with the solvent and glue. Make pipe in sections (the joints must be kept warm). Tape the bottom of the thermistor string to the proper location on the PVC pipe (measure from the bottom of the PVC to be sure the string is located properly.)

2. If installed in bedrock, drill about 10 holes in the bottom two feet of the PVC to allow grout to get out of the PVC pipe. (Muck in the bottom of the hole could plug the pipe so the extra holes are necessary.

3. Lower the first section into the hole and clamp in place with a pipe clamp. Join the second section using the cleaner and glue.

4. Wait 15 minutes for the glue to cure before proceeding. (If low temperature exist, glue will not work and threaded pipe with metal couplings will have to be used)

5. Lower second section, taping thermistor string on as you go.

6. Continue as above until the bottom of the hole is reached. Measure the amount of stickup to find the exact location of the thermistors downhole.

7. If installed in bedrock, mix grout and pump it down the PVC to fill the hole from the bottom. If open fractures occur down the hole, the grout may not fill the hole. In this case, finish filling the hole later from the surface. If installed in overburden, backfill the hole with soil, placing a bentonite seal at the ground surface.

8. A protective casing is installed on the surface to house the cable and fitting of the thermistor string.

This completes the installation. It is important that a good record be made of each installation including such items as location, test hole number, date, depth of the top thermistor, thermister string number, and any other data which may be helpful in interpreting the data from that particular installation.

10.0 RECORDKEEPING PROCEDURES

10.1 GENERAL

Because of the number and variety of field activities underway during the investigation program, certain records must be kept in an up-to-date fashion by field personnel. These records will be used for data analysis, contract administration, progress and dissemination reporting purposes. It is the responsibility of field personnel to obtain and submit the records for which they are responsible.

10.2 FIELD BOOK

Each geologist assigned to a rig will carry a waterproof field book at all times. This book will serve as a personal record of events on a daily basis. Information to be recorded in the field book includes field data, technical information, and memoranda. Information contained in the field books should include all drilling related activities which will be used in filling out the Daily Drilling Report; sufficient notes must be taken to adequately compile the overall data as it will be the bases for paying the drilling contractor. The field books are to be retained in the geotechnical files upon completion of the program.

10.3 RESIDENT ENGINEER'S REPORT

The Resident Engineer will submit to the Engineering Operations Manager, on a daily basis, a report detailing the activities of the day for all phases of work. This report will be used as a basis for the progress report and to document contract administration. A sample of the form to be used is shown in Exhibit 25.

Details to be reported on this form include all activities of the day: specifically, down time, weather delays, drilling time, and any special activities.

10.4 DAILY DRILLING ACTIVITIES

The geologist on each drilling rig will complete a shift report form as shown on Exhibit 26 at the end of each shift. This form will list drilling activities undertaken during the course of the shift and the quantities of each.

Upon completion of activities for the day, the completed form will be signed by the Harza-Ebasco geologist and the driller. One copy of the form will be given to the drilling contractor, one copy to the Resident Geologist, and one copy kept for the Resident Engineer's records.

10.5 PROGRESS CHARTS

The Resident Engineer will develop and maintain charts indicating expendatures-to-date as well as charts indicating drilling progress.

At the completion of each day, the Resident Engineer will mark the borehole progress by filling in the depth in red on the drilling progress wall chart and marking percent completion in the progress table.

10.6 WEEKLY REPORT

The Resident Engineer will compile a weekly report at the end of each work week. This report will indicate the progress and activities of all field investigations including but not limited to the following:

- Drilling;
- Geophysics;
- Contractor Staffing;
- Pay Items;
- Expenses;
- Other

Daily reports submitted by the rig geologists, together with camp records, will form the basis for the weekly report. Copies of the report will be distributed to the following:

- Resident Geologist
- Lead Geotechnical Engineer
- Engineering Operations Manager
- Anchorage Contract Administration
- Field Posted Copy

Copies of technical data obtained during the week will be furnished by the Resident Geologist and forwarded as attachments to the Weekly Reports.

10.7 CONTRACT RECORD OF PAY ITEMS

A continuous record will be kept by the Resident Engineer for the purpose of contract administration. This record will indicate all contract pay items on a daily basis with extensions for weekly totals, monthly totals, and cumulative totals to date. These records will be kept primarily for contract administration of the drilling contract and will be forwarded to the Anchorage Contract Administrative personnel as an attachment to the weekly report. The records will form the basis for payment under the drilling contract.

The Resident Engineer will document on a daily basis all activities of the day concerning contractual performance. Copies of the rig geologist's Shift Reports, (Exhibit 26) and contractor Work Reports will be reviewed and evaluated relative to contractual requirements and results recorded in the residents daily report. Items such as special drilling procedures, hourly work, delays, downtime etc. will be discussed in detail.

10.8 SHIFT ROTATION AND R&R

The Lead Geotechnical Engineer will develop, with the assistance of the Resident Geologist and Resident Engineer, a H/E shift rotation schedule including R&R and home visitation. A copy of the latest schedule will be posted.

10.9 DEBRIEFING MEMORANDUM

All field personnel are required to write a brief memo about the way in which future site explorations could be improved and any comments on the technical or interpretive aspects of the investigation. The memorandum should address but should not be limited to comments on the field manual, drilling contract, sampling on coring methods, camp facilities, etc. The memorandum should be directed to the Lead Geotechnical Engineer who will insure that the information will be reviewed and placed in the Task 5 files in Anchorage.

11.0 FIELD OPERATIONS

11.1 BASE CAMP

All field activities will be undertaken from the base camp located near the Watana damsite. Food and lodging, office space, and recreational facilities are provided at the camp. Bedding, linen, and towels are also provided, as are laundry facilities for personal clothing. Meals are served at regular times during the day. Instructions covering camp rules and procedures will be issued to all personnel at the camp.

11.2 TRANSPORTATION

Transportation between Anchorage and the staging area at Talkeetna (Exhibit 1) is either by auto or by fixed-wing aircraft. Transportation between Talkeetna and the camp is provided by helicopter service. Arrangements must be made in advance with the camp manager for transportation between Talkeetna and the camp.

Transportation between the camp and worksites is provided by helicopter as required. Arrangements must be made in advance with the camp logistics manager for all transportation. Because of helicopter usage demands and availability, time must be allocated based on existing priorities. Therefore, it is the responsibility of all field personnel to properly arrange both departure and pick-up time. Helicopter pilots will not wait for late arrivers.

Helicopters are equipped for slinging operations. In addition, a Bell 205 aircraft is available for larger slinging operations. Because of the nature of the terrain in the powerhouse - "Fingerbuster" area all large or heavy equipment must be slung in. Harza-Ebasco field personnel may be required to assist with radio guidance for slinging from time to time. All safety rules should be followed. (See Appendix I) Prior arrangements must be made for slinging operations.

12.0 EQUIPMENT FOR FIELD PERSONNEL

A. Field Equipment

All personnel working in the field shoulu take the following work related equipment with them to the site:

- Pens, pencils (mechanical), rulers, protractors, and colored pencils (supplied through Anchorage);
- Clipboard - with a cover to protect the logs in the rain;
- Field book (supplied through Anchorage);
- Hand lens (10x);
- Calculator with power pack and/or extra batteries

The remainder of the office supplies and field equipment are available at the camp.

B. Personal Requirements

The following items should be taken in suitable quantities:

- Personal hygiene items - bath soap, shaving cream, etc.;
- * Clothing for field work suitable for arctic temperatures;
- Field boots and
- Rain gear;
- Rubber boots;
- Any prescription medications   * supplied

13.0 COMMUNICATIONS

Two-way FM radios will be issued to the field staff for communication between the worksite, camp base, and helicopters. These radios are to be used for arranging transportation, logistics work information, and emergencies only. Use for casual conversation, etc., is not permitted.

A telephone is provided at the base camp for outside communications and emergencies. The telephone numbers at the site are 733-2450, 733-2295, and 733-2296. (2296 is reserved for emergency communications)

14.0 SAFETY

14.1 GENERAL

Reference is made to the Project Safety Manual. This manual, available at the site, outlines practically all aspects of prudent safety practices. It is up to each individual to promote safety at all times. Don't take chances with your safety or the safety of others! The following policies are intended to assist you in this effort, they are not all-inclusive. Your suggestions concerning safety matters are solicited. Remember - safety is everybody's job!

14.2 OPERATION OF ON-SITE EQUIPMENT

All vehicles will be operated in a safe, legal and courteous manner.

No one will operate a vehicle while under the influence of alcohol or drugs.

Misuse of vehicles or disregard for safety regulations will subject the employee responsible to disciplinary action or dismissal.

14.3 USE OF INTOXICANTS, STIMULANTS OR DEPRESSANTS

Use of intoxicants, stimulants, or depressants on Harza-Ebasco contracted project work sites will not be allowed during working hours. All field personnel are expected to observe all base camp rules and regulations at living quarters in off duty hours.

Safety requires that anyone under the influence of alcohol or drugs be removed from the work site immediately.

14.4 FIREARMS

All Harza-Ebasco personnel will comply with base camp firearm policies in all cases. No personal firearms will be allowed on any job site without prior permission of the Logistics Director. The description, intent of use, and propriety of all circumstances involved shall dictate whether permission will or will not be granted.

14.5 DRILLING OPERATIONS

Normal rules of safety will apply on all drilling operations in compliance with OSHA requirements.

14.6 HELICOPTER

Travel and logistic support for the field operations will be provided by helicopter or ground transportation as appropriate. All personnel new to the site will be briefed on helicopter safety prior to entering the aircraft by a pilot. The following basic rules will be complied with when using helicopters:

- The Pilot is in command of the aircraft. He decides weather limits, flights, paths, allowable sling loads, etc.

- Seat belts will be worn at all times when riding in the aircraft;

- No passengers are allowed in a helicopter carrying a sling load;

- Never approach or leave a helicopter from or towards the tail. Always allow yourself to be seen by the pilot.

- Never walk uphill away from a helicopter.

The following rules apply especially to load-slinging operations:

- A clearance of 50 feet diameter must be provided for the helicopter rotors when landing;

- Only one person should give signals to the pilot when setting down a load. The signaling should be done at a safe distance from the pilot's side of the aircraft. No personnel will be required to unhook the load as there is a release mechanism in the aircraft;

- When hooking up a load to be moved, care must be taken to avoid grounding the aircraft through your body. This can be done by using rubber gloves, rubberized hooks, or deliberate grounding by the pilot prior to hookup. Techniques will vary with equipment and personnel. Specific instructions will be provided onsite.

14.7 WEATHER - TEMPERATURE

Air temperature is an important factor in all drilling operations and it is frequently critical in the winter. While it is not unusual to find a person who is unaware of the importance of the wind in calculating the effective temperature, it is of value to be familiar with this chart with this chart relating speed and air temperature to the "equivalent chill temperature." The mean month temperature over the past three years during May ranged from 2.3°C to 7.6°C with a low of -8.0°C. Please study the chart (Exhibit 27) and refer to it frequently; it may help you avoid frost bite.

14.8 EMERGENCY PROCEDURES & AIR OPERATIONS MANUAL

The subject manual authored by the Susitna Logistics Director will be made available to all participants in the Watana exploration programs.

It will provide field personnel with a set of guidelines which will assist them in responding to our emergency situation.

As outlined in the manual "The frequent use of aircraft and boats, the nature of the terrain, the weather and the isolation of the site area all tend to require a constant sense of alertness and preparedness to prevent any accident from occuring."

15.0 PERMIT CONDITIONS

15.1 GENERAL

The Watana field investigation takes place on land belonging to the public and to native corporations or managed by state and Federal agencies.

Before any field activities or programs commence on these lands, approved state and federal permits as applicable to the nature of the work, or the land on which work is to proceed are obtained and copies should be in the hands of the Harza-Ebasco Resident Engineer in the field. In addition it is vital that the contractor have a copy of the permit and also be aware of any and all provisions of the permit. A permit training seminar will be scheduled prior to start of work.

The permits delineate what is to be done, to a certain extent how it is to be done, and often list special conditions, instructions, or restrictions to be followed in implementing the program.

The permit, in its final as-issued form, is not a "blank check" to perform a field program. Rather, the total permit package, the permit application plus any supporting attachments or agency supplements, may affect the field work at three levels:

A. Supplying specific details as to how the work is to be conducted; and/or

B. Establishing general criteria for how the work is to be conducted; and/or

C. Establishing the need for on-site authorization by an agency field representative for particular aspects of the work.

15.2 CONDUCT OF THE WORK

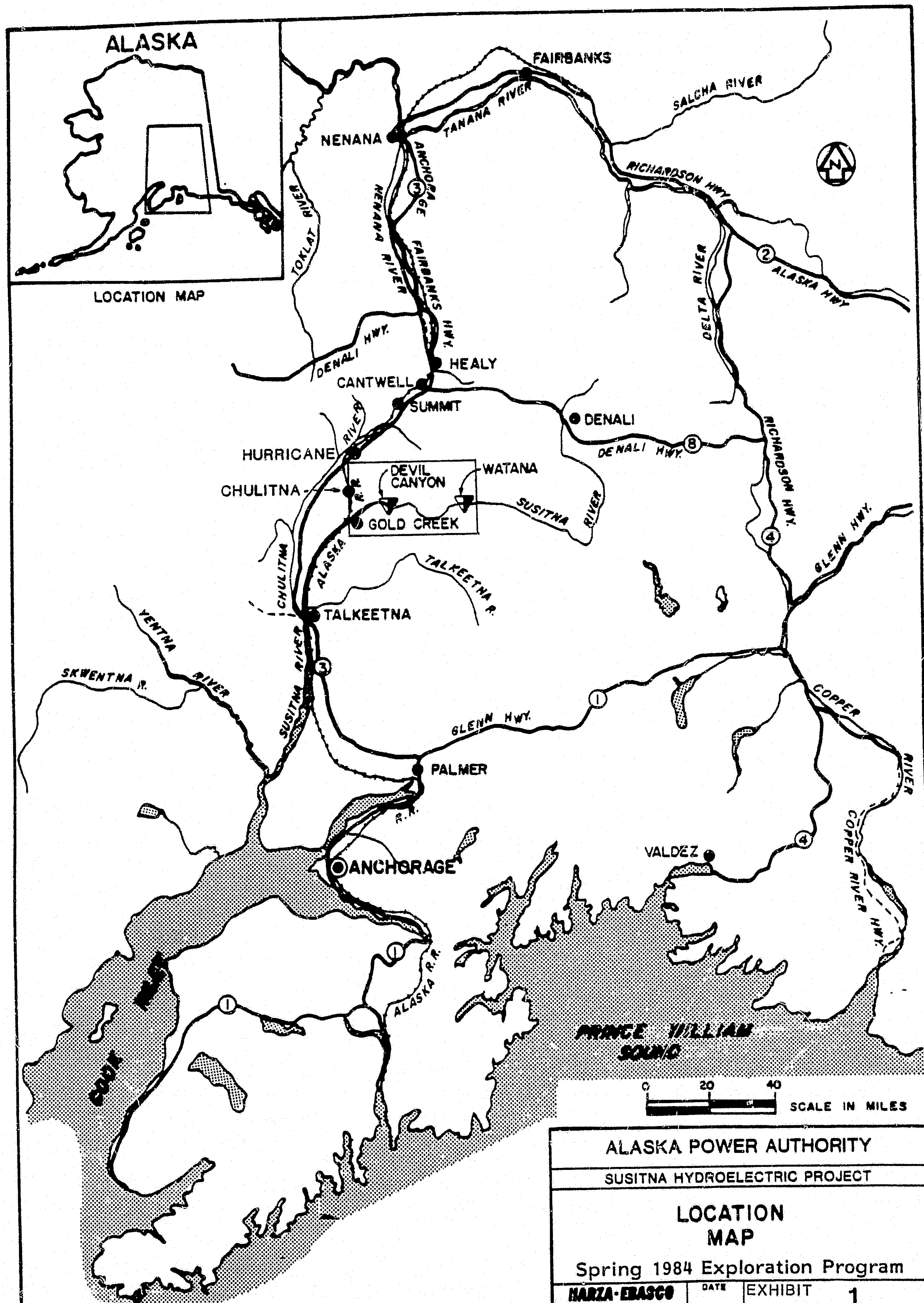
Harza-Ebasco, its employees, contractors and subcontractors are subject to existing state and federal regulations governing the particular activities in question as well as specific permit conditions.

Adherence to the following guidelines will ensure smooth progress of the work, and facilitate and expedite permission for future field activities.

- All field activities should be conducted so as to minimize any adverse impact on air, land, and water quality.
- Disturbance of existing conditions should be kept to a minimum; and any disturbed work areas should be left as near a to the original condition as is practically possible.
- Disturbance of vegetation, including the organic mat, must be kept to a minimum consistent with accomplishing the work.

- Feeding animals: Since July 1, 1977, it is a misdemeanor to feed bears, foxed, wolves, and wolverine in Alaska. In addition to direct feeding, it is also illegal to leave food or garbage in a manner that attracts animals. This law is to protect both humans (from physical damage and disease, i.e., rabies) and the animals.
- Camp fires: Field personnel should be aware of local fire conditions and exercise due caution with hot equipment, open flames, and cigarettes while in the field. Personnel should promptly notify BLM Fire Control of all fires, and take measures necessary or appropriate for the prevention and suppression of fires. Check the permit for all fire restrictions.
- Debris and litter: All construction debris and litter is to be disposed of properly, and work sites are to be left in a clean and stable condition.
- Hunting and fishing: Spare time hunting and fishing is out of the scope of project activities. However, field personnel wishing to engage in these activities should be aware of the state and federal regulations in their area. Seasons, limits, and regulations vary by area and the applicable regulation booklets are available from Alaska Department of Fish and Game offices.

EXHIBITS



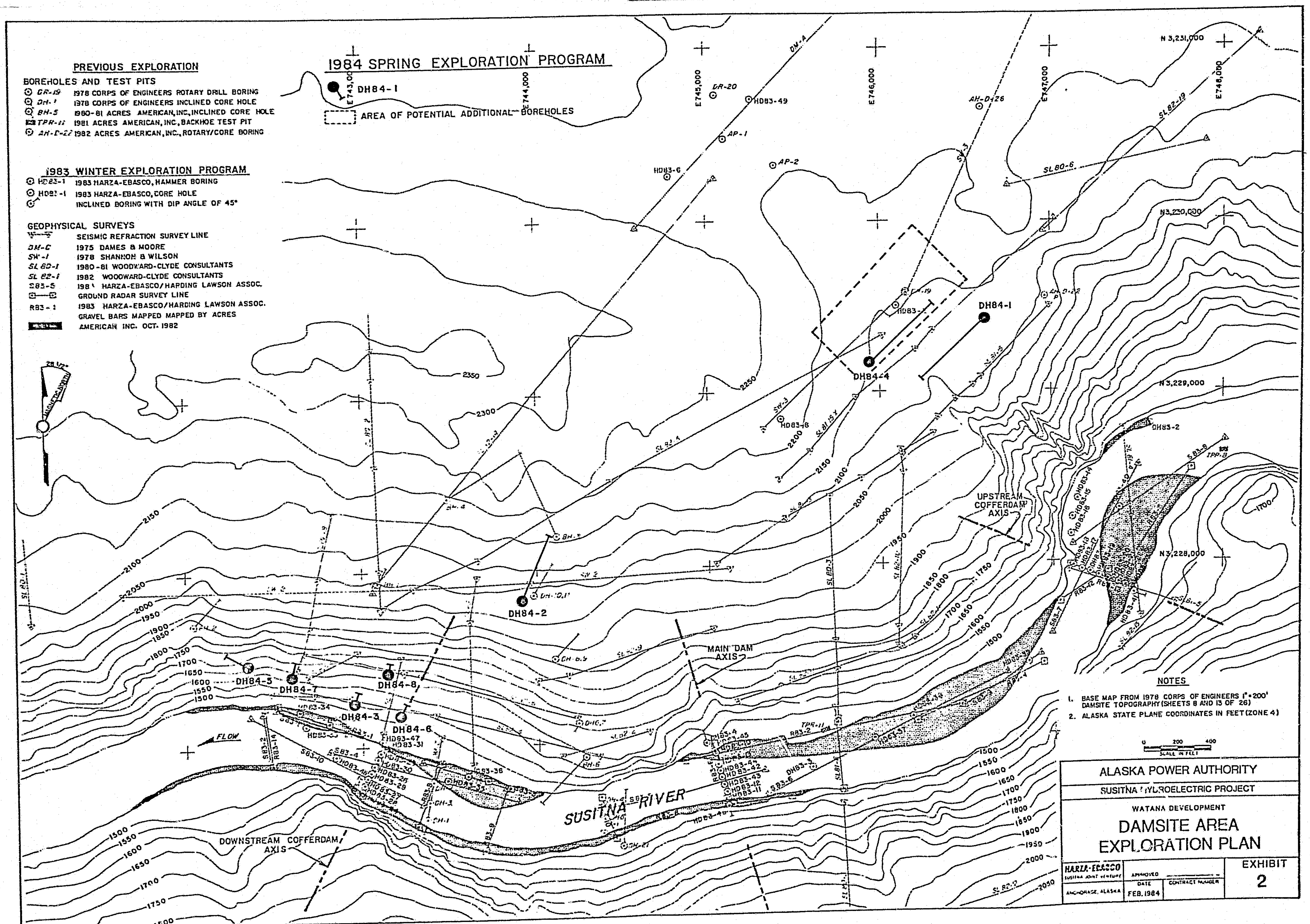
ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

LOCATION MAP

Spring 1984 Exploration Program

HARZA-EBASCO SUSITNA JOINT VENTURE | DATE | EXHIBIT 1



PREVIOUS EXPLORATION
BOREHOLES AND TEST PITS
DR-19 1978 CORPS OF ENGINEERS ROTARY DRILL BORING
DH-1 1978 CORPS OF ENGINEERS INCLINED CORE HOLE
BH-5 1980-81 ACRES AMERICAN, INC. INCLINED CORE HOLE
TPR-11 1981 ACRES AMERICAN, INC., BACKHOE TEST PIT
1982 ACRES AMERICAN, INC., ROTARY/CORE BORING
1983 WINTER EXPLORATION PROGRAM
1983 HARZA-EBASCO, HAMMER BORING
1983 HARZA-EBASCO, CORE HOLE
INCLINED BORING WITH DIP ANGLE OF 45°
GEOPHYSICAL SURVEYS
SEISMIC REFRACTION SURVEY LINE
DM-C 1975 DAMES & MOORE
SW-1 1978 SHANNON & WILSON
SL 80-1 1980-81 WOODWARD-CLYDE CONSULTANTS
SL 82-1 1982 WOODWARD-CLYDE CONSULTANTS
HARZA-EBASCO/HARDING LAWSON ASSOC.
GROUND RADAR SURVEY LINE
R83-1 1983 HARZA-EBASCO/HARDING LAWSON ASSOC.
GRAVEL BARS MAPPED MAPPED BY ACRES AMERICAN INC. OCT. 1982
1984 SPRING EXPLORATION PROGRAM
DH84-1
AREA OF POTENTIAL ADDITIONAL BOREHOLES
DH84-1
DH84-2
DH84-3
DH84-4
DH84-5
DH84-6
DH84-7
DH84-8
UPSTREAM COFFERDAM AXIS
MAIN DAM AXIS
DOWNSTREAM COFFERDAM AXIS
SUSITNA RIVER
FLOW
N 3,231,000
N 3,230,000
N 3,229,000
N 3,228,000
E 743,000
E 744,000
E 745,000
E 746,000
E 747,000
E 748,000
NOTES
1. BASE MAP FROM 1978 CORPS OF ENGINEERS 1"=200' DAMSITE TOPOGRAPHY (SHEETS 8 AND 13 OF 26)
2. ALASKA STATE PLANE COORDINATES IN FEET (ZONE 4)
SCALE IN FEET
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
WATANA DEVELOPMENT
DAMSITE AREA
EXPLORATION PLAN
EXHIBIT
2
HARZA-EBASCO
ANCHORAGE, ALASKA
FEB. 1984

WATANA DAMSITE DRILLING PROGRAM DATA SUMMARY

Drillhole[1] Number	Total Length, L.F. Overburden	Rock	Inclination (from Vertical)	Approximate Surface Elevation
DH84-1	35	815	35°[2]	2125
DH84-2	25	875	30°	2025
DH84-3	15	135	30°	1550
DH84-4	35	815	35°[2]	2150
DH84-5	15	250	30°	1650
DH84-6	15	135	30°	1525
DH84-7	15	210	30°	1600
DH84-8	15	85	30°	1675
OPTIONAL	165	245	30°	
	335	3565		

1 Drilling sequence is by drillhole number, but is subject to change at the discretion of the Resident Geologist.

2 This is a general angle specification which can be increased to a maximum of 40° in the field by the Resident Geologist.

SCHEDULE

3/2/84

FINS, FINGERBUSTER, POWERHOUSE AREA
EXPLORATION PROGRAM
WATANA SITE

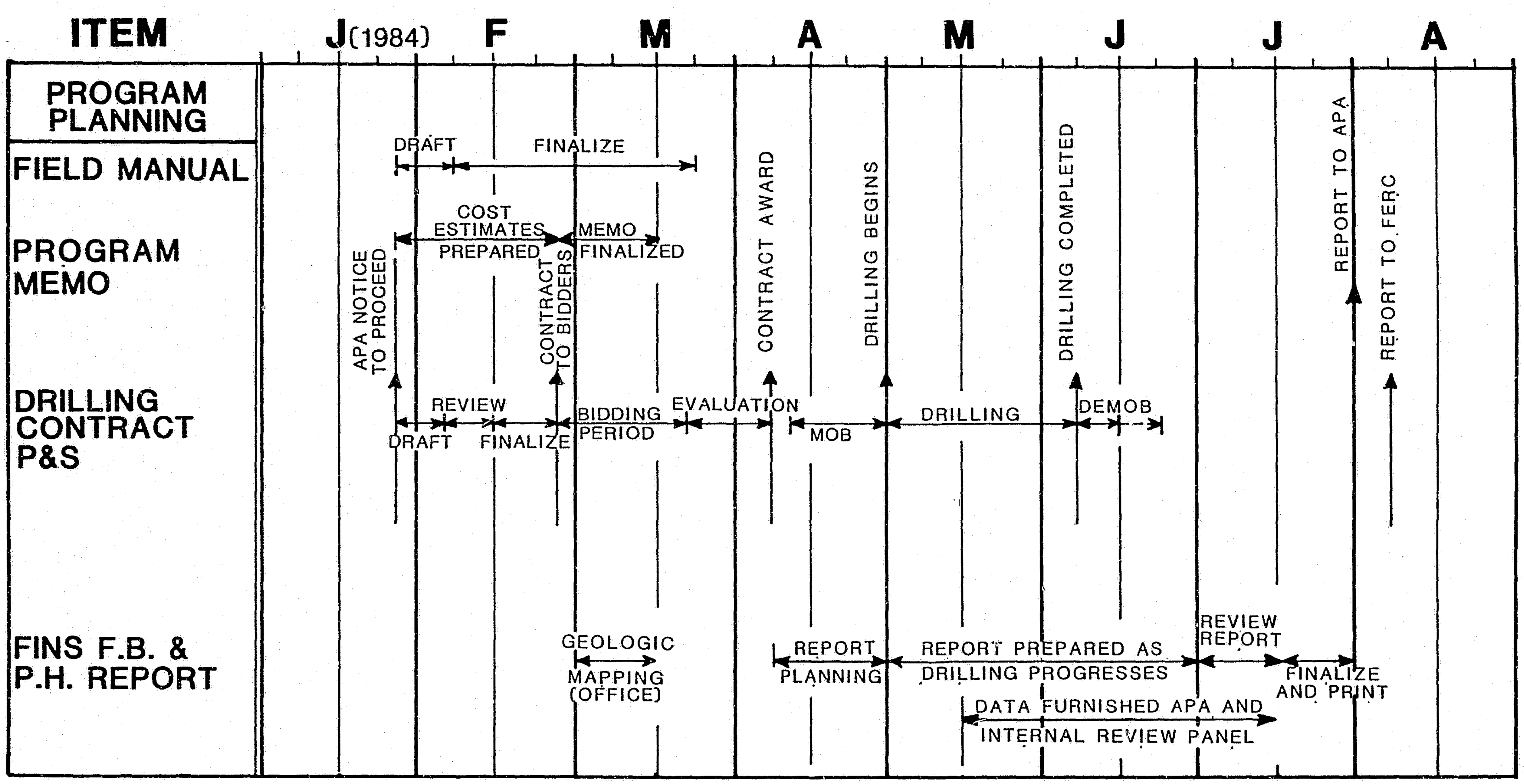
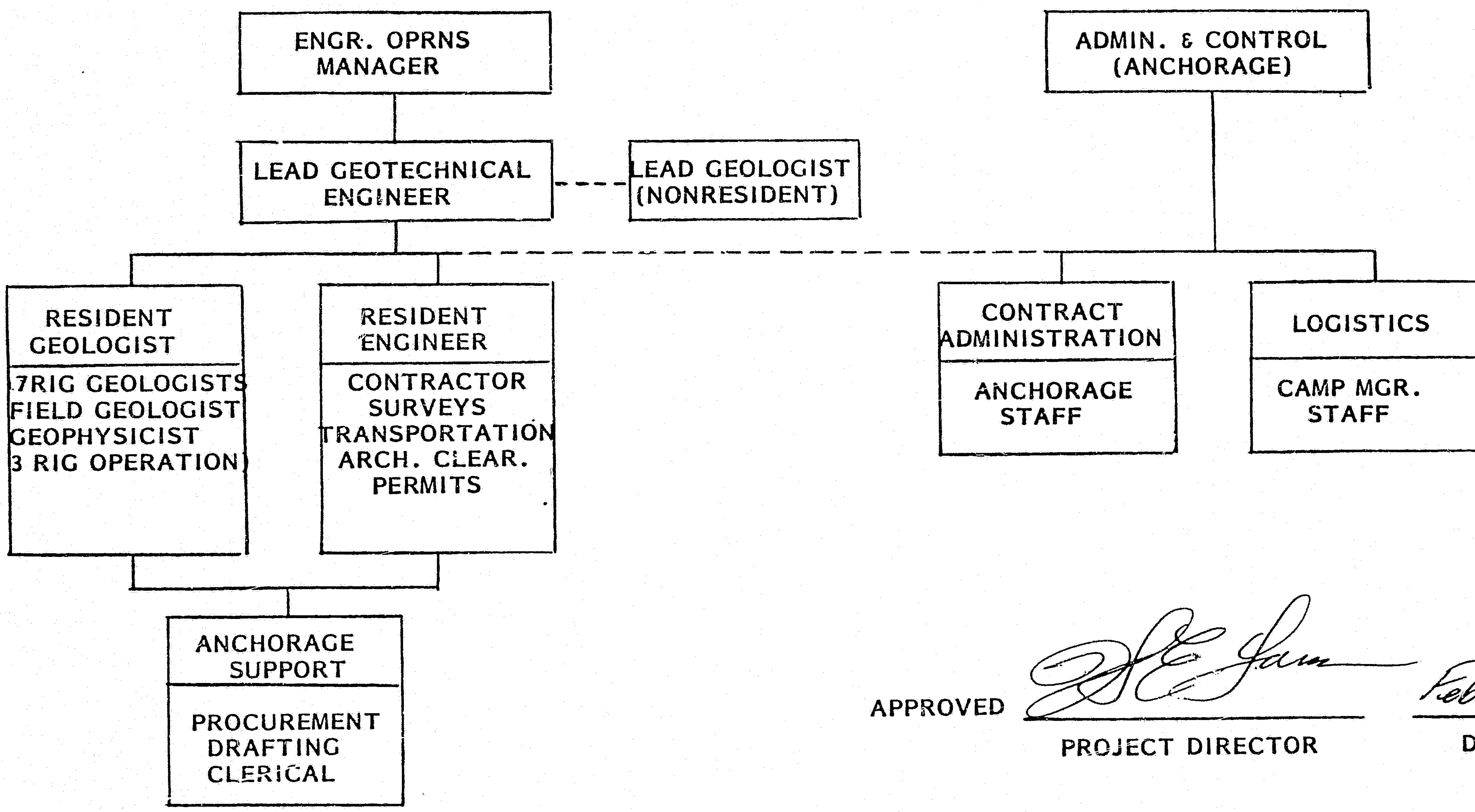


EXHIBIT 4

WATANA

SUMMER 1984 GEOTECHNICAL EXPLORATION PROGRAM

ORGANIZATION CHART II



APPROVED [signature] Feb. 3 1984

PROJECT DIRECTOR — DATE

EXHIBIT 5

HARZA-EBASCO
SUSITNA JOINT VENTURE

SOIL BORING LOG

Sheet ______ of ______
Date ______

Boring No EXAMPLE
Feature ______
Coordinates: N ______
E ______
Logged by ______

Angle (from Horizontal) ______
Bearing ______
Date Started ______
Date Completed ______

Ground Elevation ______
Rock Elevation ______
Total Depth ______
Ground-Water Elevation ______

Depth (Elevation)	Rig Blow Count	Sample Interval	Sample No. and Type	Blow Count per 0.5 ft.	Length Recovered	Graphic Log	Soil Description	Instrumentation	Remarks
						USCS	Coarse Grained: Texture, prefix for >25% coarse or >12% fines: density, moisture, particle shape, particle distribution, organics, geologic characteristics, color		- more than 50% visible or retained on # 200 sieve
	38 49		1			GM	Silty, sandy, GRAVEL, medium dense, moist, subrounded-subangular, fine to coarse, with trace organics, light grey silt and clay clumps, olive grey		Organic 4%
						USCS	Fine Grained: Texture, plasticity, density, consistency, moisture, structure, organics, geologic characteristics, color, texture		-more than 50% not visible or passing # 200 sieve
			3SS	12 26 29	10"	CL	Silty CLAY, low plasticity, v. dense, stiff, moist, frozen, clay layers 6" thick, ice lenses, olive grey, <5% gravel, occasional cobbles		LL31 PL20 PI11 CL M/C 75%

EXHIBIT 6

HARZA-EBASCO
SUSITNA JOINT VENTURE

GEOLOGIC LOG

Sheet ______ of ______
Date ______

Boring No EXAMPLE
Feature
Coordinates: N
E
Core Sizes

Angle (from Horizontal)
Bearing
Date Started
Date Completed
Total Depth

Ground Elevation
Rock Elevation
Overburden Thickness
Ground-Water Elevation
Logged by

Depth (Elevation)	Graphic Log: Lithology	Graphic Log: Structure	Graphic Log: Fracture Log 3 6 9 12	Classification and Physical Description	C.R.-Graphic	C.R./RQD %	Box Number	Instrumentation	Remarks
				Depth, ROCK TYPE, weathering/alteration, hardness, strength, texture, color, mineralogical composition (%), alteration products Degree of fracturing, spacing, filling or stain, separation, roughness, angle of fractures to core axis, size of pieces Depth, DISCONTINUITY TYPE, composition, weathering or alteration, fracture spacing, moisture, density and plasticity if applicable, color					
15									
16				15.2-150.5 DIORITE, slightly weath., hard, strong, medium xyln, grey; plagio, hbl ≃ 25% ~~Closely to mod.~~ fract., aver. 0.5'-1.0 stained, tight to p. open, sl. rough, 10° to 30°, max 2.0'. min. 0.3'		98/41	1/12		
17									
18				18.2-27.8 SHEAR ZONE, gouge & breccia 2.3', sl. alteration, fracturing close to very close, damp, stiff, sl. plasticity, white					
19									

Boring No. ______ Sheet ______ of ______

EXHIBIT 6

BOREHOLE LOG ABBREVIATIONS

Geological Abbreviations

AND	-	Andesite	weath	-	Weathered
DIO	-	Diorite	unweath	-	Unweathered
plag	-	Plagioclase	alter	-	Altered
bio	-	Biotite	unstd	-	Unstained
qtz	-	Quartz	hrd	-	Hard
kspar	-	Potassium Feldspar	mass	-	Massive
hbl	-	Hornblende	strg	-	Strong
Fe	-	Iron	xyln	-	Crystalline
Mg	-	Magnesium	mins	-	Minerals
PY	-	Pyrite	pseudo	-	Pseudomorphs
kaolin	-	Kaolinite	reheal	-	Rehealed
chlor	-	Chlorite	jt	-	Joint
$CaCo^3$	-	Calcium Carbonate	coat	-	coating
fract	-	Fractured	F	-	Fine Grained
			C	-	Coarse Crained

General Abbreviations

frzn	-	Frozen	lt	-	Light
v	-	Very	drk	-	Dark
dcrs	-	Decreases	mod	-	Moderate
prbly	-	Probably	occ	-	Occasional
fr	-	Fresh	pc	-	Piece
mech	-	Mechanical	sl	-	Slight
brkg	-	Breaking	max	-	Maximum
			min	-	Minimum

Drilling Abbreviations

	Type			Sample Type	
COR	-	Rock Coring	SS	-	Split Spoon
HSA	-	Hollow Stem Auges	Ts	-	Shelby Tube
HAM	-	Hammer	Db	-	Dennison Barrel
CSR	-	Center Stem Rotary	Pb	-	Pitcher Barrel
DDH	-	Down Hole Hammer	Cb	-	Churn Drill Barrel
			Cs	-	Single Tube Core Barrel
			Cd	-	Double Tube Core Barrel

Engineering Abbreviations

M/C	-	Moisture Content
LL	-	Liquid Limit
PL	-	Plastic limit
PI	-	Plastic Index
NP	-	Nonplastic
Gs	-	Specific Gravity
K	-	Permeability

DESCRIPTIVE TERMINOLOGY FOR HARDNESS

Very Hard:	Cannot be scratched with knife; metal powder left on sample.
Hard:	Scratched with knife with difficulty; trace of metal powder left on sample; scratch faintly visible.
Moderately Hard:	Readily scratched with knife; scratch leaves heavy trace of dust and is readily visible.
Low Hardness:	Gouged or grooved to 1/16 inch by firm pressure on knife; scratches with penny.
Soft:	Gouged or grooved readily with knife; small thin pieces can be broken by finger pressure.
Very Soft:	Carves with knife; scratched by fingernail.

DESCRIPTIVE TERMINOLOGY FOR DISCONTINUITY SPACING

Discontinuity		Spacing	
Joints	Bedding, Cleavage, Foliation	English	Metric
Very Close	Very Thin	Less than 2 inches	Less than 5 cm
Close	Thin	2 inches to 1 foot	5 cm to 30 cm
Moderately Close	Medium	1 foot to 3 feet	30 cm to 1 m
Wide	Thick	3 feet to 10 feet	1 m to 3 m
Very Wide	Very Thick	Greater than 10 feet	Greater than 3 m

EXHIBIT 8

DESCRIPTIVE TERMINOLOGY FOR WEATHERING/ALTERATION

Fresh:	Rock fresh; crystals or gains bright; a few joints may show slight staining; crystalline rocks ring if struck with a hammer.
Slight:	Rock generally fresh; joints stained and may show clay filling if open; staining may extend into rock fabric adjacent to weathered planes; if present, feldspars may be dull and discolored; crystalline rocks ring if struck with hammer.
Moderate:	Except for quartz, most of the rock mass shows discoloration and weathering; most feldspar is dull and discolored and kaolinitization (alteration to clay minerals) is common; rock gives a dull sound if struck with hammer; rock shows overall loss of strength; portions may be "excavated" with a geologist's pick.
Severe:	All minerals, except quartz, discolored or stained; rock fabric still discernible; intergranular or intercrystalline disassociation virtually complete; internal structure essentially that of soil; fragments of strong rock may remain; may be called saprolite.
Complete:	Rock is decomposed to a soil; fabric not discernible or only rarely discernible; quartz may remain as dikes or stringers.

Occasionally, weathering due to solution (in carbonate rocks) or alteration (in hydrothermally attacked rocks of any composition) may be encountered. The following terms are used to describe any voids which result from such action:

Solid:	Contains no voids.
Pitted:	Small voids generally restricted to joint surfaces, bedding planes, or other surfaces which provide access for attacking fluids.
Vuggy:	Use restricted to solution voids in carbonate rocks and hydrothermally altered rocks; voids may be found throughout the rock mass; voids up to 4 inch average diameter.
Vesicular:	Use restricted to voids in igneous (occasionally metamorphic) rocks, void origin usually due to gas bubbles; voids up to 4 inch average diameter.
Cavernous:	Applicable to any rock; voids and channels greater than 4 inches in average diameter, voids large enough to cause serious leakage and/or structural problems.

EXHIBIT 9

DESCRIPTIVE TERMINOLOGY FOR ROCK STRENGTH

Very Strong	-	resists breakage from hammer blows, will yield dust and small chips
Strong	-	withstand a few hammer blows but will yield large fragments
Moderately Strong	-	withstands a few firm hammer blows
Weak	-	crumbles with light hammer blows

DESCRIPTIVE TERMINOLOGY FOR ROUGHNESS

CLASSIFICATION	DESCRIPTION
Smooth	Appears smooth and is essentially smooth to the touch. May be slichensided.
Slightly Rough	Asperities on the fracture surfaces are visible and can be distinctly felt
Medium Rough	Asperities are clearly visible and fracture surface feels abrasive
Rough	Large angular asperities can be seen. Some ridge and high side angle steps evident.
Very Rough	Near vertical steps and ridges occur on the fracture surface

Unified Soil Classification

(Including Identification and Description)

Major Divisions			Group Symbols	Typical Names	Field Identification Procedures (Excluding particles larger than 3 inches and basing fractions on estimated weights)			Information Required for Describing Soils
1	2		3	4	5			6
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 (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.			For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.			
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	Nonplastic fines or fines with low plasticity. (for identification procedures see ML below)			Give typical name; indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).			
	Sands More than half of coarse fraction is smaller than No. 4 sieve size.	Clean Sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.			
			SP	Poorly-graded sands, gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.			Example: Silty sand, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity. (for identification procedures see ML below)			
	(For visual classification, the ¼-in. size may be used as equivalent to the No. 4 sieve size)		SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).			
The No. 200 sieve size is about the smallest particle visible to the naked eye.					Identification Procedures on Fraction Smaller than No. 40 Sieve Size			
					Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)	
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 or clayey silts with slight plasticity.	None to slight	Quick to slow	None	Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains, color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information; and symbol in parentheses.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium	
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
	Silts and Clays	Liquid limit greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium	
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High	
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium	Example: Clayey silt, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes, firm and dry in place, loess; (ML).
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.			

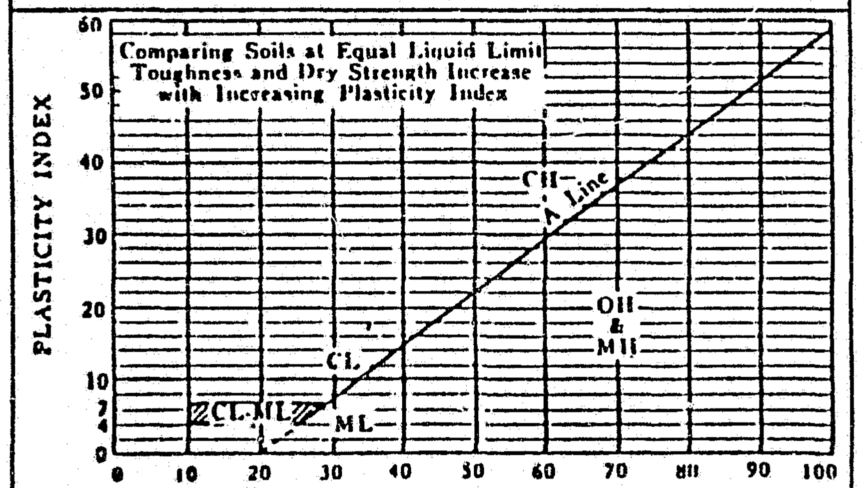
(1) Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well-graded gravel sand mixture with clay binder. (2) All sieve sizes on this chart are U.S. standard.

EXHIBIT 11

Unified Soil Classification

Laboratory Classification Criteria			
7			
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.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3	(See note, far right)	
	Not meeting all gradation requirements for GW		
	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.	
	Atterberg limits above "A" line with PI greater than 7		
	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3	(See note, far right)	
	Not meeting all gradation requirements for SW		
	Atterberg limits below "A" line or PI less than 4	Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols.	
	Atterberg limits above "A" line with PI greater than 7		

Use grain-size curve in identifying the fractions as given under field identification.



LIQUID LIMIT PLASTICITY CHART

For laboratory classification of fine-grained soils

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky. Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its 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.

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 between the fingers. This strength is a measure of the character and quantity 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. 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.

Toughness (Consistency near plastic limit)

After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. 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, the pieces should be lumped together and a slight kneading action continued 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-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

Note
(Laboratory Classification)

C_u = uniformity coefficient
C_c = coefficient of curvature
D_{60} = grain diameter at 60% passing
D_{30} = grain diameter at 30% passing
D_{10} = grain diameter at 10% passing

The grain-size distributions of well-graded materials generally plot as smooth and regular concave curves with no sizes lacking or no excess of material in any size range. The uniformity coefficient (Cu) of well-graded gravels is greater than 4, and of well-graded sands is greater than 6.

The coefficient of curvature (Cc) insures that the grading curve will have a concave curvature within relatively narrow limits for a given D_{60} and D_{10} combination. All gradations not meeting the foregoing criteria are classed as poorly graded.

EXHIBIT 12

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

TYPE OF TEST		Procedures performed on fraction passing No. 40 sieve; for field classification, simply remove by hand coarser particles that interfere.		
		TOUGHNESS (Consistency near plastic limit)	**DILATANCY** (Reaction to shaking)	**DRY STRENGTH** (Crushing characteristics)
TEST PROCEDURE AND EVALUATION OF RESULTS		Mold a lump of soil about 1/2 in. in size to the consistency of putty. If too dry, add water. If too wet, spread out into a thin layer and allow it to lose some water by evaporation. Roll the lump by hand on a smooth surface or between the palms to a thread about 1/8 in. in diameter. Fold the thread and reroll repeatedly until the thread crumbles at a diameter of about 1/8 in. when the plastic limit is reached. After the thread crumbles, the pieces should be lumped together and kneaded until the lump crumbles. The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more active are the clay minerals in the soil. Weakness of the thread at the plastic limit and quick loss of coherance of the lump below the plastic limit indicate plastic silts, kaolin-type clays, or organic soils which occur below the A-line on the plasticity chart.	Mold a lump of soil about 1/2 in. in size with enough water, if necessary, to make the soil soft but not sticky. Smooth the soil pat in the palm of one hand and shake horizontally, striking the back of the hand vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pay which changes to a livery consistency and becomes glossy. When the pat is squeezed between the fingers, the water and gloss disappear, the pat stiffens, and finally it cracks or crumbles. The reaction is **sudden** if instantly produced by a single blow, **fast** if produced by less than 5 blows, **slow** if requiring more than 5 blows, and **none** if no change can be seen after many blows. Nonplastic silts give a sudden or fast reaction whereas clays which occur above the A-line show no reaction.	Mold a lump of soil about 1/2 in. in size to the consistency of putty, adding water if necessary. Allow the lump to dry completely in the sun, air, or oven, and test its strength by crushing between the fingers. The crushing strength is as follows: **none** or **very low** if the lump crumbles with the mere pressure of handling. **low** if the lump crumbles to powder with little finger pressure. **medium** if considerable finger pressure is required to powder the lump, but a smear of powder can be easily rubbed off a smooth surface of the lump. **high** if the lump cannot be crushed to powder by finger pressure, even though it may be broken, and it is not even possible to rub off a smear of powder from a smooth surface of the lump. **very high** if lump cannot be broken between thumb and a hard surface.
IDENTIFICATION OF GROUPS	ML	None to weak	Sudden to slow	None to low
	MH	Very weak to firm	Slow to none	Low to medium
	OL	Very weak to weak	Slow	Low to medium
	OH	Very weak to firm	Slow to none	Medium to high
	CL	firm to tough	Slow to none	Medium to high
	CH	Firm to very tough	None	High to very high

REFERENCE: U.S. Department of the Interior, Earth Manual, Second Edition, 1974.

DESCRIPTIVE TERMINOLOGY FOR RELATIVE DENSITY

Proposed Correlations of Penetration Resistance and Soil Properties

Extreme caution should be exercised in using any table of correlations outside the areas or for other conditions than those for which the correlations have been established; even then large deviations from such correlations have been reported. The penetration resistance depends not only on dimensions of the equipment and the consistency or relative density of the soil, but it may also vary with the method of operation, depth below ground surface, and other factors not yet fully investigated.

AUTHOR		H. A. MOHR		TERZAGHI AND PECK		NEW YORK CITY CODE		NEW ENGLAND DIV., C.E.	
SAMPLER		1-in. Extra Heavy Pipe 1.315-in. OD, 0.957-in. ID		Raymond - Fig. 178 2.0-in. OD, 1.375-in. ID		2.50-in. OD		3.00-in. OD	
HAMMER		140 lb ±, 30-in. ± Fall		140 lb, 30-in. Fall		300 lb, 18-in. Fall		300 lb, 18-in. Fall	
SOIL		Designation	Blows/Ft.	Designation	Blows/Ft.	Designation	Blows/Ft.	Designation	Blows/Ft.
SAND and SILT	Rel. Density	Loose	Less 9	Very loose	Less 4	Loose	0 - 15	Very loose	Less 8
				Loose	4 - 10			Loose	8 - 16
		Firm	9 - 13	Medium	10 - 30	Compact	16 - 50	Medium	16 - 55
		Hard	14 - 49	Dense	30 - 50			Compact	55 - 110
		Hardpan	Over 50	Very dense	Over 50	Very compact	Over 50	Very compact	Over 110
CLAY	Consistency			Very soft	Less 2	Very soft	0 - 2	Very soft	Less 8
		Soft	Less 5	Soft	2 - 4	Soft	3 - 10	Soft	8 - 16
				Medium	4 - 8			Medium stiff	16 - 55
		Medium	5 - 10	Stiff	8 - 15			Stiff to Medium hard	55 - 110
		Hard	11 - 30	Very stiff	15 - 30	Stiff	11 - 30		
				Hard	Over 30	Hard	Over 30	Very hard	Over 110

DESCRIPTIVE TERMINOLOGY FOR CONSISTENCY (Cohesive Soils)

Consistency	Field Identification
Very Soft	Easily pentrated several inches by fist
Soft	Easily pentrated several inches by thumb
Medium	Can be penetrated several inches by thumb with moderate effort
Stiff	Readily indented by thumb but penetrated only with great effort
Very Stiff	Readily indented by thumbnail
Hard	Indented with difficulty by thumbnail

DESCRIPTIVE TERMINOLOGY FOR MOISTURE

Dry	-	addition of moisture is necessary to compact the material
Moist	-	close to the optimum moisture content; varying degrees of wetness
Wet	-	beyond the optimum moisture content; visible signs of water
Saturated	-	voids are filled with water

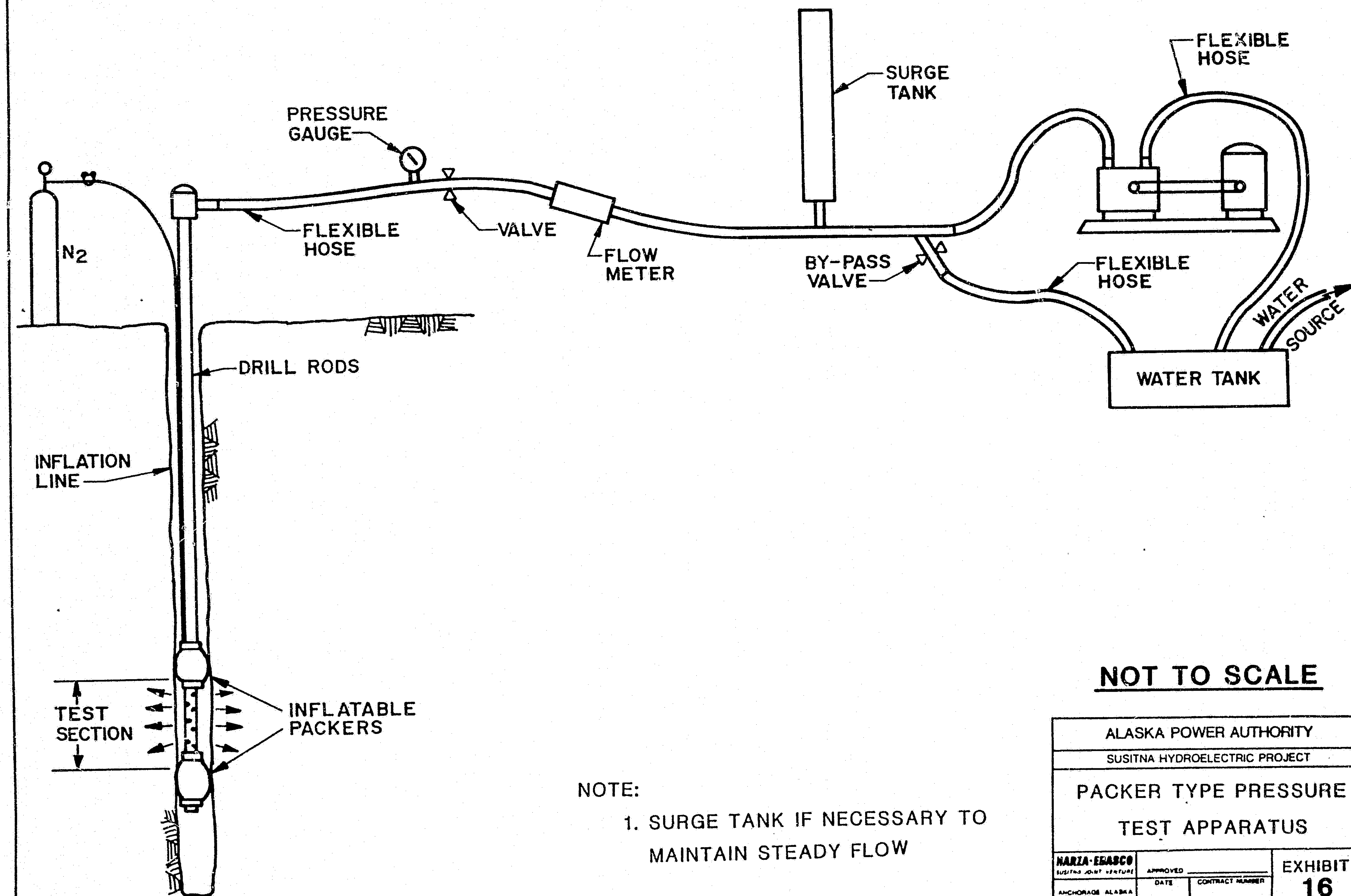
DESCRIPTIVE TERMINOLOGY FOR PLASTICITY

Degree of overall plasticity	PI	Identification (Burmister system)	Smallest diameter of rolled threads, mm
Nonplastic	0	SILT	None
Slight	1-5	Clayey Silt	6
Low	5-10	SILT and CLAY	3
Medium	10-20	CLAY and SILT	1.5
High	20-40	Silty CLAY	0.8
Very high	>40	CLAY	0.4

*After Burmister (1951a).[33] Reprinted with permission from the *Annual Book of ASTM Standards*, Part 19, copyright, American Society for Testing and Materials.

DESCRIPTIVE TERMINOLOGY FOR SOIL MODIFYING TERMS

Trace	1-10%
Little	10-20%
Some	20-35%
And	35-50%



NOTE:

1. SURGE TANK IF NECESSARY TO MAINTAIN STEADY FLOW

NOT TO SCALE

ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
PACKER TYPE PRESSURE TEST APPARATUS

HARZA-EBASCO SUSITNA JOINT VENTURE	APPROVED		EXHIBIT 16
ANCHORAGE ALASKA	DATE	CONTRACT NUMBER	

EXAMPLE:

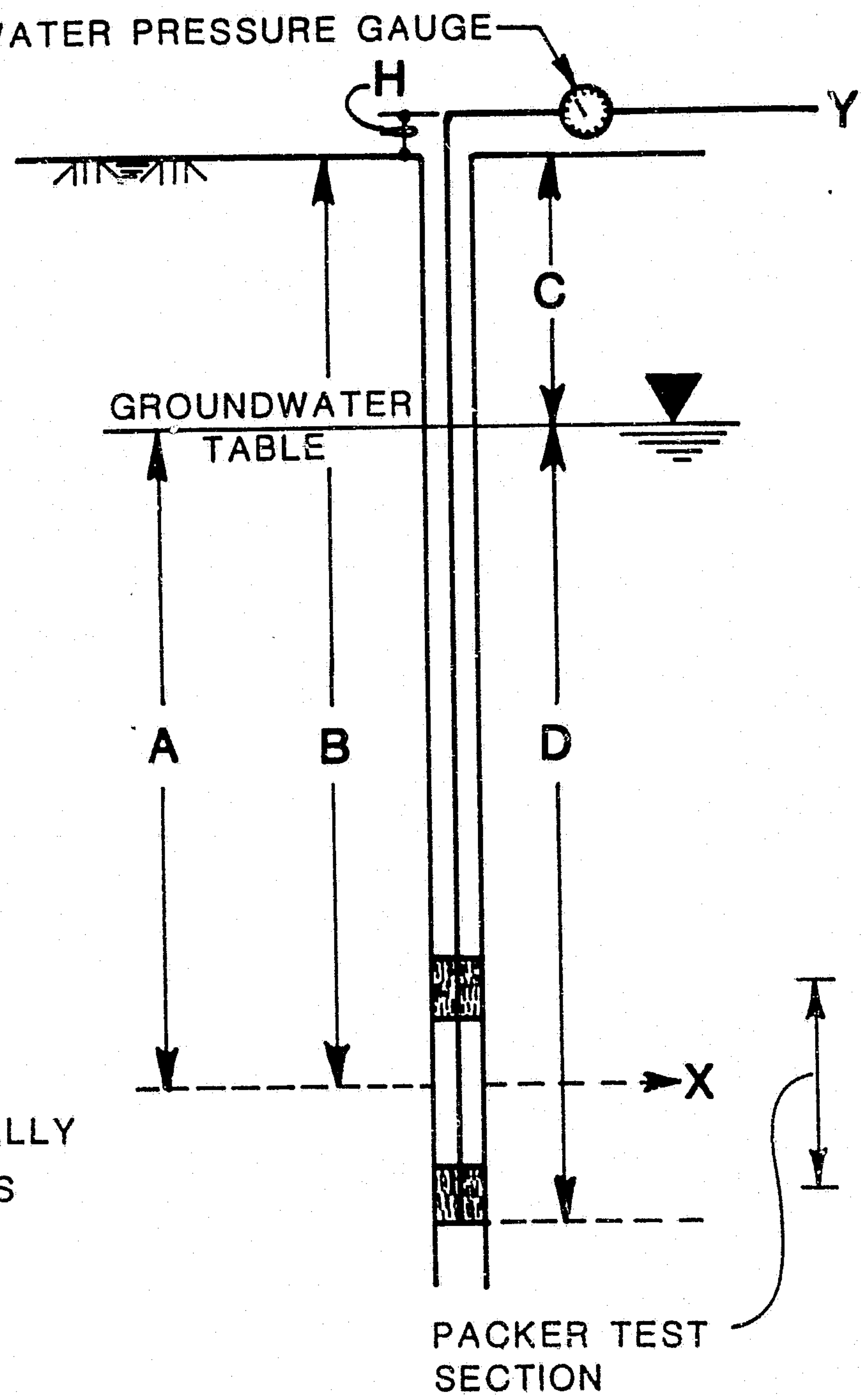
A = 105 FT.

B = 150 FT.

C = 45 FT.

D = 120 FT.

H = 2 FT.



1. INFLATION PRESSURE (I P) FOR PACKERS MUST BE SUFFICIENT TO WITHSTAND:
   a) HYDROSTATIC PRESSURE AT THE DEPTH OF LOWEST PACKER (D)
   b) PRESSURE OF THE WATER COLUMN IN THE TEST PIPE (C+H)
   c) INJECTION PRESSURE OF WATER BEING PUMPED INTO THE TEST INTERVAL (X)
   d) INCREMENT OF PRESSURE, USUALLY ≃ 30 PSI, TO SEAL THE PACKERS

$$I\ P = 0.43D + 0.43(C+H) + X + 30$$
$$= 0.43(120) + 0.43(45+2) + 105 + 30$$
$$I\ P = 210 \text{ PSI}$$

2. MAXIMUM NET APPLIED WATER PRESSURE AT MIDPOINT OF INTERVAL. NOT TO EXCEED 1 PSI PER. FOOT OF DEPTH ABOVE WATER TABLE

$$X = B - 0.43A$$
$$= 150 - 0.43(105)$$
$$= 105 \text{ PSI}$$

3. MAXIMUM GAUGE PRESSURE

$$Y = X - 0.43(C+H)$$
$$= 105 - 0.43(45+2)$$
$$= 85 \text{ PSI}$$

ALASKA POWER AUTHORITY	
SUSITNA HYDROELECTRIC PROJECT	
HYDRAULIC PRESSURE TESTS	
HARZA-EBASCO SUSITNA JOINT VENTURE, ANCHORAGE, ALASKA / APPROVED / DATE / CONTRACT NUMBER	EXHIBIT 17

Harza Engineering Company
Form SG-Is

Sheet ___ of ___
Date __________

REPORT OF WATER PRESSURE TESTING

PROJECT ______________________

Hole No. ______________ Angle (from Vertical) ________ Ground Elevation ____________
Location ______________ Bearing ________________ Rock Elevation ____________
Coordinates: N ________ Date Started ____________ Water depth during test ________
E ________ Date Completed __________ Logged by ______________

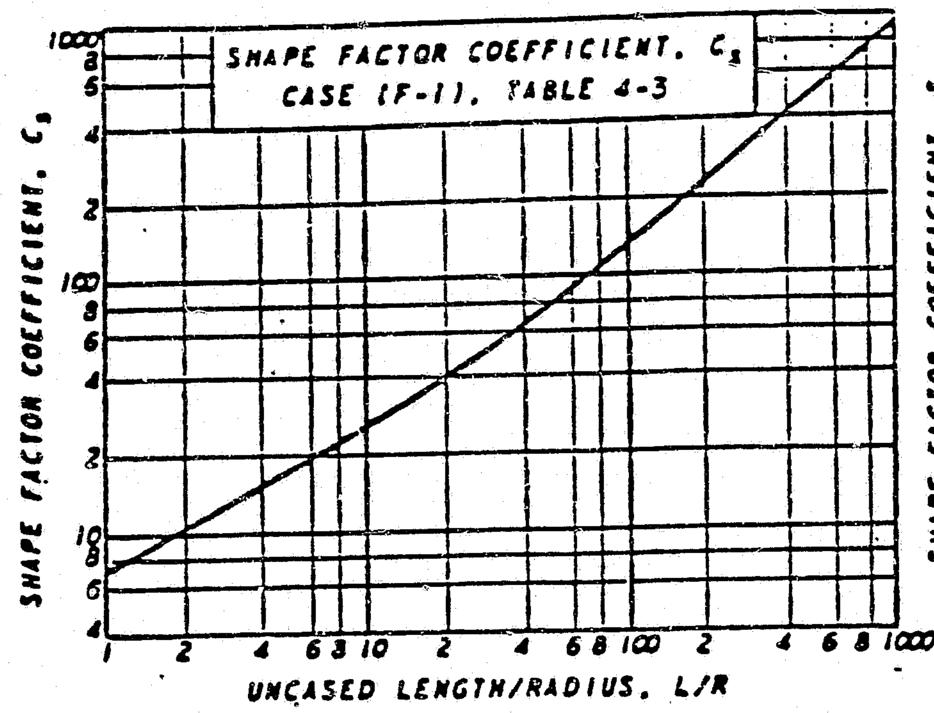
Test No.	Depth		Length of Interval Tested (feet)	Meter		Water Loss __cu.ft. __gal __liters	Elapsed Time (min.)	Rate of Loss (gpm)	Pressure				Permea-bility ____ units
	From __ft. __m.	To		Start	End				Gage __p.s.i. __kg/cm^2	(+) *Column	(-) Friction	Net (psi)	

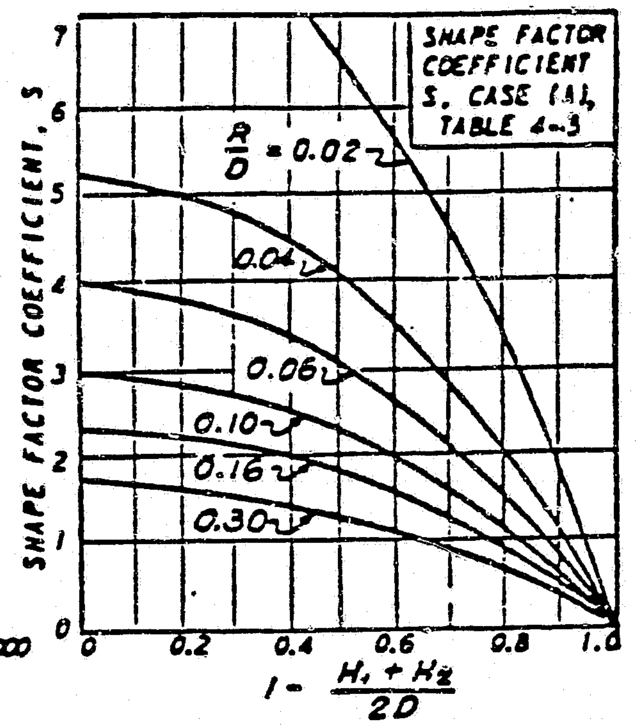
Depth to groundwater ____ feet X 0.433= ____ psi Gage + Column - Friction Loss=Net Pressure

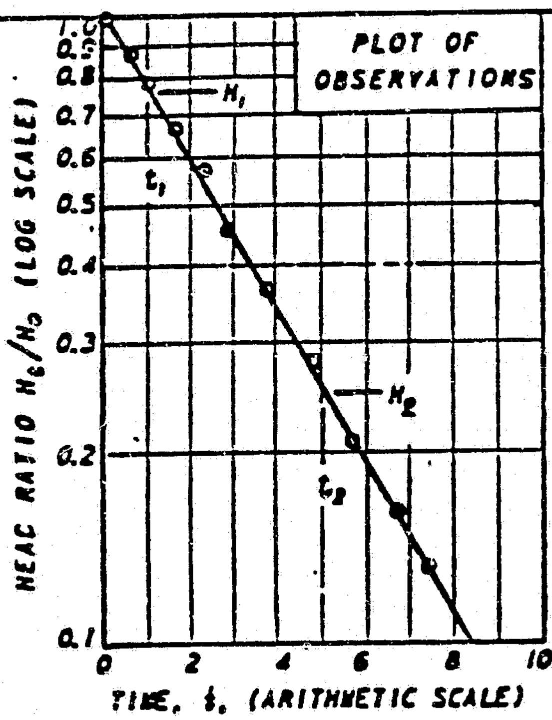
* Column pressure = (depth to middle of tested interval or depth to groundwater, whichever is smaller) X (0.433)

Conversion factors: cu.ft.X 7.48=gallons kg/cm^2 X 14.22=psi
meters X 3.28=feet liters X 0.264=gallons

EXHIBIT 18



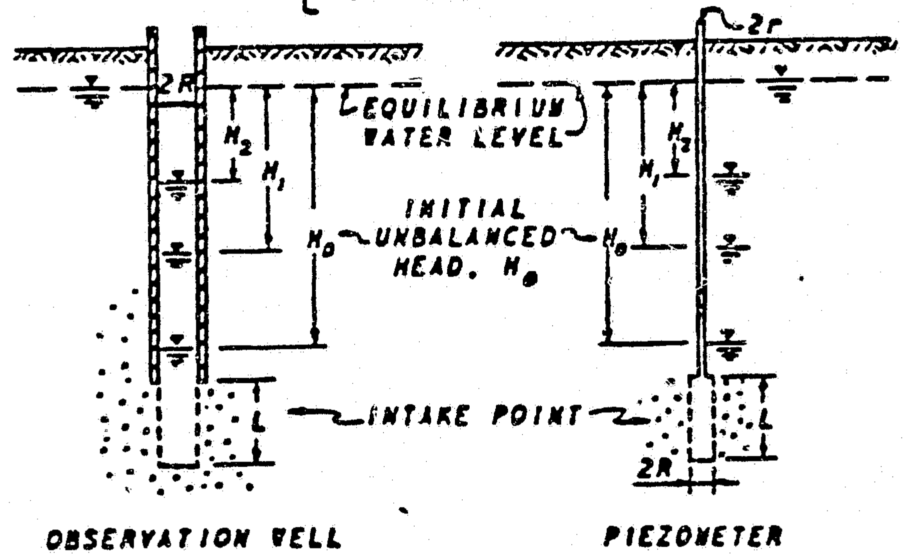




IN GENERAL:

$$K = \frac{A}{F(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$

F = SHAPE FACTOR OF INTAKE POINT
A = STANDPIPE AREA
K = MEAN PERMEABILITY
$\ln H_1/H_2$ AND $(t_2 - t_1)$ ARE OBTAINED FROM PLOT OF OBSERVATIONS.



OBSERVATION WELL IN ISOTROPIC SOIL:

OBTAIN SHAPE FACTOR FROM TABLE 4-3.

FOR CASE (C):

$$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$$

$$K = \frac{R^2}{2L} \ln\left(\frac{L}{R}\right)\left[\frac{\ln H_1/H_2}{(t_2 - t_1)}\right]$$

PIEZOMETER IN ISOTROPIC SOIL:

RADIUS OF INTAKE POINT (R) DIFFERS FROM RADIUS OF STANDPIPE (r).

$$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$$

$$A = \pi r^2$$

$$K = \frac{A}{F(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$

$$K = \frac{r^2}{2L} \ln\left(\frac{L}{R}\right)\left[\frac{\ln H_1/H_2}{t_2 - t_1}\right]$$

TEST IN ANISOTROPIC SOIL:

ESTIMATE RATIO OF HORIZONTAL TO VERTICAL PERMEABILITY AND DIVIDE HORIZONTAL DIMENSIONS OF THE INTAKE POINT BY:

$m = \sqrt{K_H/K_V}$ TO COMPUTE MEAN PERMEABILITY $K = \sqrt{K_H \times K_V}$.

FOR CASE (C), TABLE 4-3:

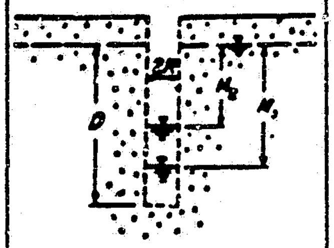
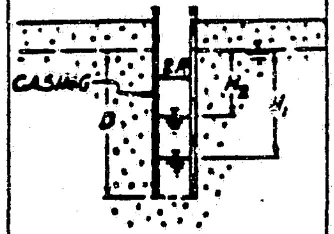
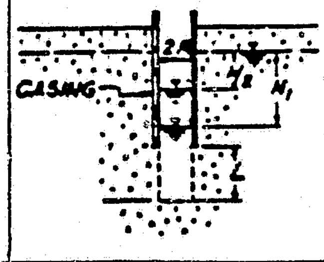
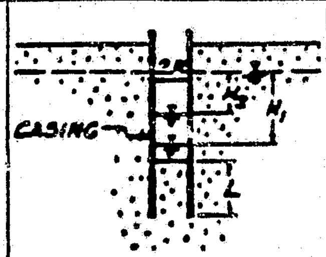
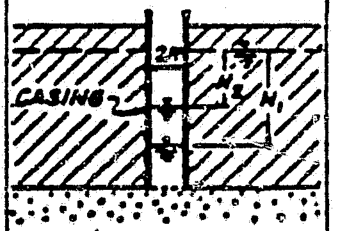
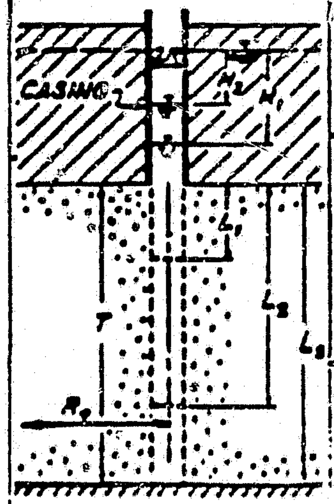
$$F = \frac{2\pi L}{\ln\left(\frac{mL}{R}\right)}$$

$$K = \frac{r^2}{2L} \ln\left(\frac{mL}{R}\right) \frac{\ln H_1/H_2}{(t_2 - t_1)}$$

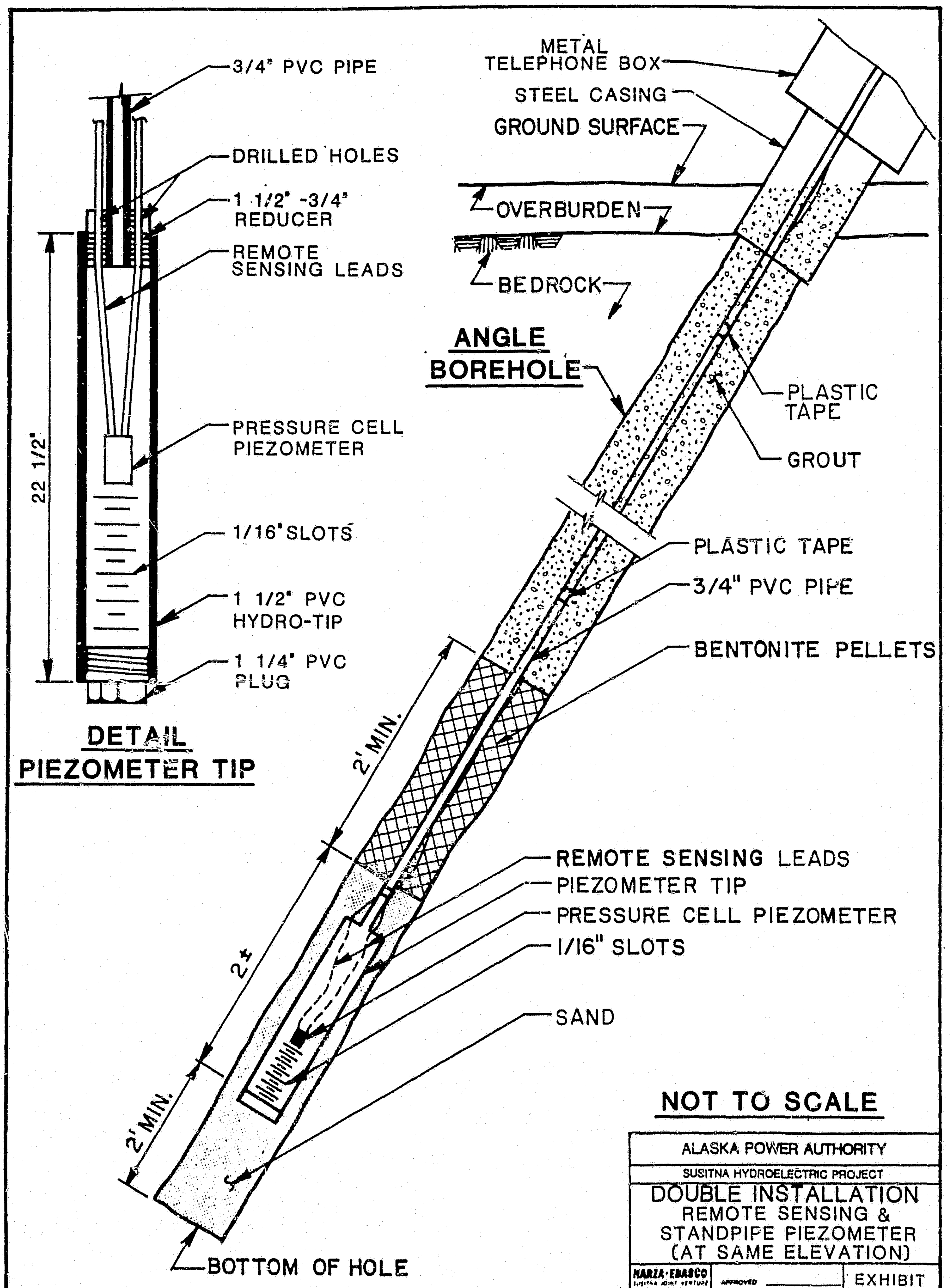
Analysis of Permeability by Variable Head Tests

REFERENCE: Department of the Navy, NAVFAC DM-7, 1971.

Shape Factors for Computation of Permeability From Variable Head Tests

	Condition	Diagram	Shape factor, F	Permeability, K by variable head test	Applicability
OBSERVATION WELL OR PIEZOMETER IN SATURATED ISOTROPIC STRATUM OF INFINITE DEPTH	(A) Uncased hole.....		$F = 16\pi DSR$	(for observation well of constant cross section) $K = \frac{R}{16DS} \times \frac{(H_2-H_1)}{(t_2-t_1)}$ FOR $\frac{D}{R} < 50$	Simplest method for permeability determination. Not applicable in stratified soils. For values of S, see Figure 4-3.
	(B) Cased hole, soil flush with bottom.		$F = \frac{11R}{2}$	$K = \frac{2\pi R}{11(t_2-t_1)} \ln\left(\frac{H_1}{H_2}\right)$ FOR $6" \leq D \leq 60"$	Used for permeability determination at shallow depths below the water table. May yield unreliable results in falling head test with silting of bottom of hole.
	(C) Cased hole, uncased or perforated extension of length "L".		$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$	$K = \frac{R^2}{2L(t_2-t_1)} \ln\left(\frac{L}{R}\right) \ln\left(\frac{H_1}{H_2}\right)$ FOR $\frac{L}{R} > 8$	Used for permeability determinations at greater depths below water table.
	(D) Cased hole, column of soil inside casing to height "L".		$F = \frac{11\pi R^2}{2\pi R + 11L}$	$K = \frac{2\pi R + 11L}{11(t_2-t_1)} \ln \frac{H_1}{H_2}$	Principal use is for permeability in vertical direction in anisotropic soils.
OBSERVATION WELL OR PIEZOMETER IN AQUIFER WITH IMPERVIOUS UPPER LAYER	(E) Cased hole, opening flush with upper boundary of aquifer of infinite depth.		$F = 4R$	$K = \frac{\pi R}{4(t_2-t_1)} \ln\left(\frac{H_1}{H_2}\right)$	Used for permeability determination when surface impervious layer is relatively thin. May yield unreliable results in falling head test with silting of bottom of hole.
	(F) Cased hole, uncased or perforated extension into aquifer of finite thickness: (1) $\Psi \leq 0.20$ (2) $0.2 < \Psi < 0.85$ (3) $\Psi = 1.00$ Note: R_0 equals effective radius to source at constant head.		(1) $F = C_s R$	$K = \frac{\pi R}{C_s(t_2-t_1)} \ln\left(\frac{H_1}{H_2}\right)$	Used for permeability determinations at depths greater than about 5 ft. For values of C_s, see Figure 4-3.
			(2) $F = \frac{2\pi L_2}{\ln(L_2/R)}$	$K = \frac{R^2 \ln\left(\frac{L_2}{R}\right)}{2L_2(t_2-t_1)} \ln\left(\frac{H_1}{H_2}\right)$ FOR $\frac{L_2}{R} > 8$	Used for permeability determinations at greater depths and for fine grained soils using porous intake point of piezometer.
			(3) $F = \frac{2\pi L_3}{\ln\left(\frac{R_0}{R}\right)}$	$K = \frac{R^2 \ln\left(\frac{R_0}{R}\right)}{2L_3(t_2-t_1)} \ln\left(\frac{H_1}{H_2}\right)$	Assume value of $\frac{R_0}{R} = 200$ for estimates unless observations wells are made to determine actual value of R_0.

REFERENCE: Department of the Navy, NAVFAC DM-7, 1971.



ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

DOUBLE INSTALLATION
REMOTE SENSING &
STANDPIPE PIEZOMETER
(AT SAME ELEVATION)

HARZA-EBASCO SUSITNA JOINT VENTURE | APPROVED | DATE | CONTRACT NUMBER

ANCHORAGE, ALASKA

EXHIBIT 21

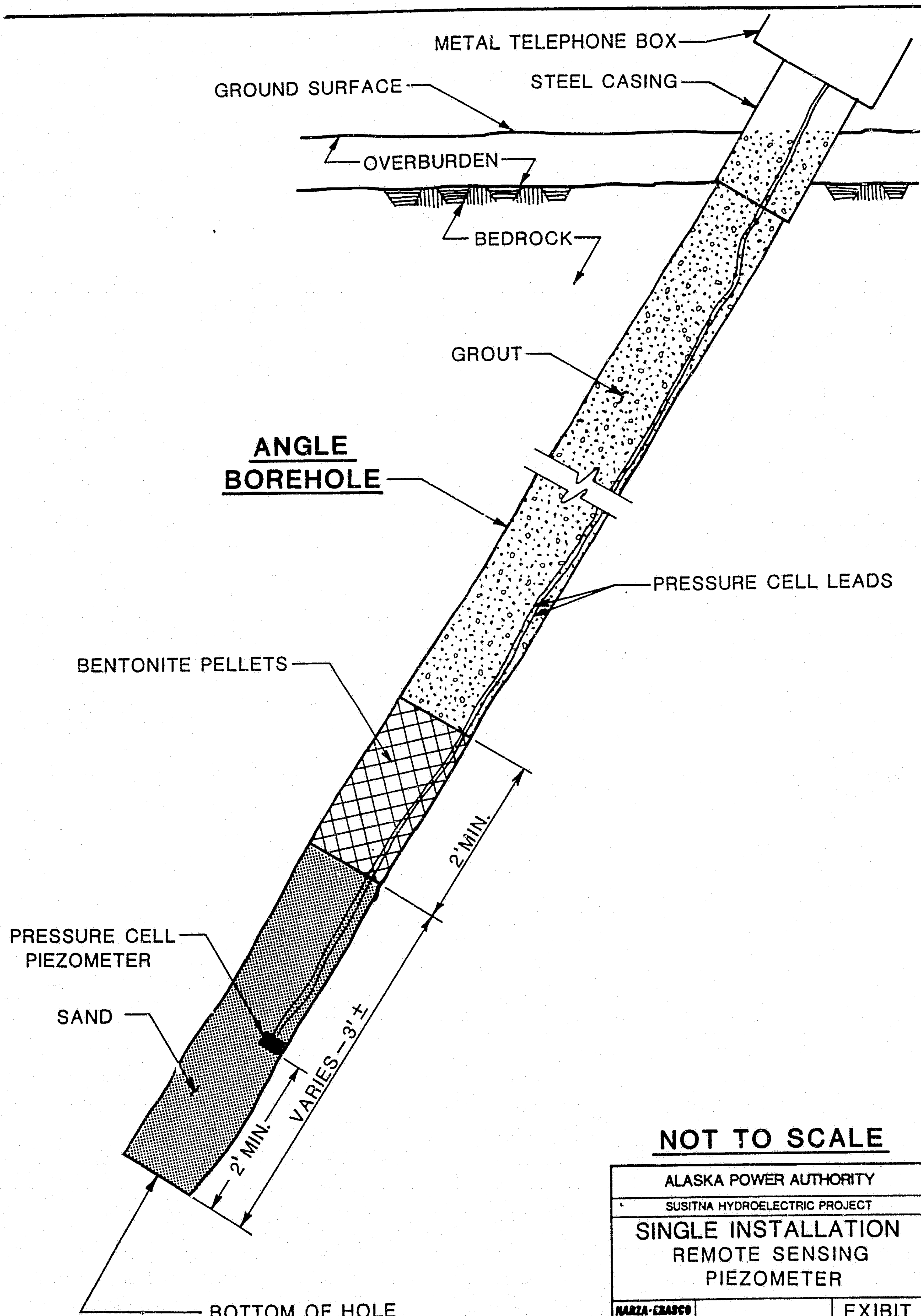
Measurement of Ground Water Levels

JOB NUMBER __________ DATE __________

OWNER __________ BY __________

LOCATION __________ METHOD OF MEASUREMENT __________

Boring Number	Time	Referance Point	Elevation of Reference	Depth to Water	Water Elevation	Notes
		Top of Casing, Ground Surface, etc.				Note Casing Stick-up, clogging, etc.



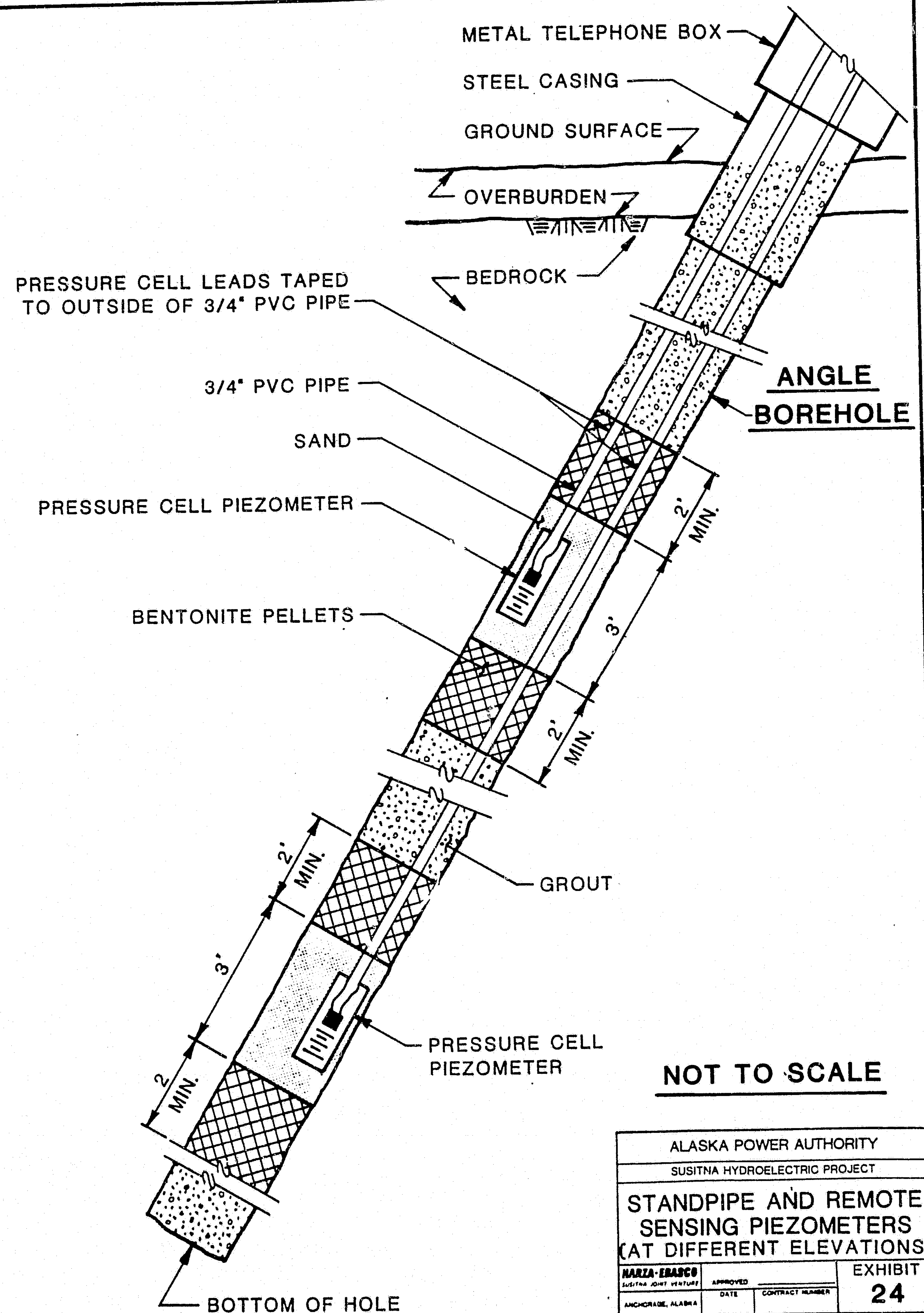
NOT TO SCALE

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

SINGLE INSTALLATION
REMOTE SENSING
PIEZOMETER

HARZA-EBASCO SUSITNA JOINT VENTURE	APPROVED		EXIBIT
ANCHORAGE, ALASKA	DATE	CONTRACT NUMBER	23



ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

STANDPIPE AND REMOTE SENSING PIEZOMETERS (AT DIFFERENT ELEVATIONS)

HARZA-EBASCO SUSITNA JOINT VENTURE	APPROVED		EXHIBIT
ANCHORAGE, ALASKA	DATE	CONTRACT NUMBER	24

SHEET ____ OF ____

DAILY ENGINEERS REPORT
1984 SUSITNA FIELD PROGRAM
WATANA CAMP

DATE ______________________ REPORT NO. ______________________

WEATHER CONDITIONS ____

REPORT

ACTIVITIES

DELAYS

REMARKS

____
ENGINEER

EXHIBIT 25

ARZA-EBASCO
SITNA JOINT VENTURE

WATANA DEVELOPMENT

RIG GEOLOGIST SHIFT REPORT

CONTRACTOR: ______________________________

DATE: ____________ HOLE No. ____________ H/E RIG GEO. ____________

SHIFT: ____________ ANGLE: ____________ DRILLER: ____________

RIG No. ____________ HOLE SIZE: ____________ HELPER: ____________

DEPTH OF HOLE: START OF SHIFT ____________ END OF SHIFT ____________

ITEM No.	WORK DESCRIPTION	STARTING TIME: A.M.____ P.M.____													SHIFT TOTALS		
																HOURS	
		7	8	9	10	11	12	1	2	3	4	5	6	7	FOOTAGE	S.T.	O.T.
1	MOB./DEMOB.																
2	SITE PREP. a) PADS																
	b) PLATFORM																
	c) HELICOPTER																
3	SOIL DRILLING																
4	ROCK CORING a) "H" SIZE																
	b) "N" SIZE																
	c) CASING																
5	PRESSURE TESTING																
6	OPERATING RATE																
	a) INSTRUMENT																
	b) ENGR. DIR.																
7	AVAIL. RATE																
	a) ALIGNMENT																
	b) GEOPHYSICS																
	c) STANDBY																
8	WATER SUPPLY a) LAKE																
	b) RIVER																
	c) CAMP HYD.																
9	MOVE TIME																
10	DOWN TIME																
11	OTHER (EXPLAIN)																

MATERIAL DESCRIPTION	QUANTITY	MATERIAL DESCRIPTION	QUANTITY

EXHIBIT 26

WIND SPEED MILES PER HOUR	COOLING POWER OF WIND EXPRESSED AS "EQUIVALENT CHILL TEMPERATURE"																				
	TEMPERATURE (°F)																				
CALM	40	35	30	25	20	15	10	5	0	-5	-10	-15	-20	-25	-30	-35	-40	-45	-50	-55	-60
	EQUIVALENT CHILL TEMPERATURE																				
5	35	30	25	20	15	10	5	0	-5	-10	-15	-20	-25	-30	-35	-40	-45	-50	-55	-65	-70
10	30	20	15	10	5	0	-10	-15	-20	-25	-35	-40	-45	-50	-60	-65	-70	-75	-80	-90	-95
15	25	15	10	0	-5	-10	-20	-25	-30	-40	-45	-50	-60	-65	-70	-80	-85	-90	-100	-105	-110
20	20	10	5	0	-10	-15	-25	-30	-35	-45	-50	-60	-65	-75	-80	-85	-95	-100	-110	-115	-120
25	15	10	0	-5	-15	-20	-30	-35	-45	-50	-60	-65	-75	-80	-90	-95	-105	-110	-120	-125	-135
30	10	5	0	-10	-20	-25	-30	-40	-50	-55	-65	-70	-80	-85	-95	-100	-110	-115	-125	-130	-140
35	10	5	-5	-10	-20	-30	-35	-40	-50	-60	-65	-75	-80	-90	-100	-105	-115	-120	-130	-135	-145
40	10	0	-5	-15	-20	-30	-35	-45	-55	-60	-70	-75	-85	-95	-100	-110	-115	-125	-130	-140	-150
WINDS ABOVE 40 HAVE LITTLE ADDITIONAL EFFECT.	LITTLE DANGER						INCREASING DANGER (Flesh may freeze within 1 min.)					GREAT DANGER (Flesh may freeze within 30 seconds)									
	DANGER OF FREEZING EXPOSED FLESH																				

EXHIBIT 27

APPENDICES

A - Drilling Methods

B - Rock Core Storage and Photos

C - Piezometer Devices

D - Earth manual - USBR

E - Permafrost Field Description

F - Field Description of Soils

G - Lugeons Measure Hydraulic Pressure Testing

H - Site Geology

I - Helicopter Safety Manual

APPENDIX A
DRILLING METHODS

TABLE OF CONTENTS

1.0 ROTARY DRILLING

The design of engineering structures demands accurate knowledge of subsurface conditions and physical properties of the foundation materials. The least disturbed core or samples are required to determine these properties, demanding extreme care in application of core and sampling methods. No single method will preclude satisfactory results in all types of bedrock. Different devices and techniques have been developed for drilling and sampling a wide variety of foundations. Proper coring and sampling is a combination of science and art; many procedures have been standardized, but alteration and adaptation of techniques are often dictated by specific investigation requirements. The highest quality samples or core are obtained at the least cost by using a variety of equipment and techniques applied with experience and sound judgment as dictated by the subsurface conditions.

1.1 Straight Rotary

The Watana boreholes will be advanced using the rotary drilling method, by rotating a drill string consisting of a series of hollow drill rods to the bottom of which is attached either a cutting bit or a core barrel with a coring bit. Cutting bits shear off chips of the material penetrated and thus are used primarily for penetrating overburden. Coring bits cut an annular hole around an intact core which enters the barrel and is retrieved. As the rods with the bit or barrel are rotated, a downward pressure is applied to the drill string (rods) to obtain penetration, and drilling fluid under pressure is introduced into the bottom of the hole through the hollow drill rods and passages in the bit and barrel. The drilling fluid serves the dual function of cooling the bit and removing the cuttings from the bottom of the hole as it returns to the surface in the annular space between the drill rods and the walls of the hole. In an uncased hole, it also serves to support the walls of the hole.

Rotary drilling for putting down holes in soil may be used for borings 2.5 inches in diameter and larger. The rotary drilling method has several advantages over methods such as cable tool drilling as it is more rapid, in general, than the other methods and usually results in less disturbance to the material to be sampled. The primary disadvantage of this method is that it is not well adapted for use in materials containing a large percentage of particles of gravel size or larger since these particles will rotate beneath the drill bits and cannot be easily broken. Thus, a nest of gravel will continually remain at the base of the hole.

1.2 Reverse Rotary

The procedure described above is referred to as straight rotary drilling. A second method of rotary drilling commonly used is referred to as reverse rotary or reverse cirulation drilling. The difference in the two methods is primarily in the circulation of the drilling fluid to remove the cuttings and in the equipment used; the reverse rotary also is limited to use with noncoring bits. In the reverse rotary method, as the rods are rotated, the drilling fluid is introduced under gravity into the annular space between the drill rods and the walls of the hole. The fluid, laden with cuttings from the bottom of the hole, returns to the surface via the hollow drill rods. The return flow is accomplished by (a) application of a head at the top of the annulus relative to the discharge end of the drill rods; (b) application of suction on the drill rods; (c) introduction into the drill rods of a supply of air which mixes with the drill fluid and causes it to be removed by air lift. When compared to straight rotary drilling, this method has the dual advantage of (1) minimization of disturbance to the walls of the hole owing to the higher head in the hole and more outward seepage pressure on the hole walls; and (2) more rapid and efficient removal of cuttings from the hole since the area of the drill rods is less than the annulus thereby giving higher upward velocity. However, it is best suited to holes 12 inches and larger in diameter.

1.3 Basic Elements

The basic elements of rotary drilling consist of a rotary drive mechanism and a means of applying downward pressure to the drill rods as they are rotated. In the rotary drive mechanism, spiral bevel gears are used to transmit power from the motor to a drive quill which commonly has a hexagonal or octagonal bore. The quill in turn drives a strong hollow steel spindle of similar cross section referred to as the drive head, which has a chuck attached to its lower end. Drill rods and flush steel casing are fed into the hole directly through the hollow spindle and are gripped by the chuck so that the rotary motion of the spindle is transferred to the rods or casing. These hydraulic cylinders are used to apply a downward pressure, through the yoke, to the drill rods as they are rotated. Some rotary rigs are equipped with a screw-feed mechanism in lieu of the hydraulic feed unit.

The characteristics of some of the more common rotary rigs, including two portable rigs, are shown in table form on Exhibit A1.

1.4 Overburden Drill Bits - No Sampling

Several types of cutting bits are available for use in rotary drilling. Some of these are shown in Exhibit A2. In general, these bits may be divided into two broad categories, drag bits and rock bits. The type of bit used will be determined primarily by the characteristics of the material to be penetrated. Drag bits rely on a shearing and scraping action to remove material and therefore are suitable primarily for use in overburden; some may also be used in soft rock. Included in this category are fishtail, bladed, replaceable blade, and carbide insert bits (Exhibit A2). The fishtail (a) is a bit resembling a straight chopping bit with a split cutting edge, each half of which has been turned slightly in the direction of rotation. The term bladed bit (b) is applied to bits having two, three, or four blades or wings which have been forged to a thin cutting edge and turned slightly in the direction of rotation. The tips of the cutting edges of both types of bit are made of tungsten carbide alloy for wear resistance. The carbide insert drag bits (c) are similar except that they do not have turned edges and the insert forms the cutting edge. These are commonly available with three or four wings. The Hawthorne replaceable blade bit and similar bits commonly have three insert blade bits which are themselves individually replaceable. All of the drag bits have passages through which the drilling fluid may be pumped. These jets are directed at the blades for cleaning purposes. The fishtail bit and the two-bladed bit are used in sands, clay, and other soils. The three-and four-bladed bits of the fixed blade, carbide insert, and replaceable blade types are suitable for use in firmer soils and in somewhat harder materials than the fishtail.

1.5 Rock Bits

The rock bits used in rotary drilling are classified as noncoring or coring bits depending upon whether the rock is broken into small fragments and washed out of the hole or is recovered in the form of an intact core. Noncoring bits are used to advance a hole when there is no need for intact samples to be taken. They are of three types, the cone bit, the roller bit, and the diamond plug bit. The cone and roller bits have teeth milled on the surfaces of cones and rollers which are so mounted that the teeth rotate as it is turned. The cone type bits are available with two or three cones, the latter of which (Exhibit A2) is commonly referred to as a tricone bit (d). The roller bit has two rollers on inclined axes and two rollers on a horizontal axis which are mounted perpendicular to the included rollers. The spacing and height of the teeth on both the cone and roller bits depend on the material to be penetrated. Long and widely spaced teeth are used for soft materials. Diamond plug bits are of three types, namely, concave, pilot, and taper. The concave bit (Exhibit A2) is used in relatively soft rock. The pilot bit which has a lead section of smaller diameter than the remainder of the bit is used in hard rock and in vertical holes in rock strata of differing hardness. The taper-type is particularly well adapted for use in very hard rock. However, it is common practice to advance holes through rock or very dense overburden by taking continuous cores with core barrels. In such cases a coring bit is required which attaches to the bottom of the core barrel and cuts an annular hole rather than a hole covering the entire cross section of the boring. Four basic categories of coring bits are available, namely, diamond, carbide insert, sawtooth and calyx or shot bits.

2.0 CORE SAMPLES

2.1 General

Rock and overburden coring is the process in which a tube (core barrel) with a cutting bit at its lower end, cuts an annular hole in the foundation mass thereby creating a cylinder or core which is recovered in the core barrel or within a second or inner tube within the core barrel.

The primary purpose of core drilling is to obtain an intact sample truly representative of the in situ material. The behavior of a foundation mass is affected not so much by the type and hardness of the material composing the rock itself but more significantly by the nature of fractures in the rock. The size and spacing of fractures, degree of weathering of fractures, and the presence of soil within the fractures are critical items. Generally the resistance of a rock mass to sliding depends on the strength of soil within fractures or shear zones. In some instances in better rock and with the use of proper equipment and good drilling technique, close to 100 percent core recovery is achieved.

2.2 Drilling Technique

Insofar as proper technique is concerned, the primary objective is to obtain the maximum percent core recovery and the maximum amount of information rather than to achieve the maximum rate of production. In all cases the drilling procedure to be followed is the one which brings about the highest percent recovery. The exact procedure must be determined in the field. Variations in the speed of rotation, the downward pressure on the core barrel, the pressure at which the drilling fluid is introduced into the hole and the length of hole drilled (run length) prior to removal of the core are major items which must be controlled by the driller. In general, coring should be initiated with short runs both because the upper portions of the rock mass are commonly highly fractured and also because the elevations of any core losses can be more accurately determined. The length of the runs may be increased to 5 or even 10 feet when conditions indicate that this is permissible. However, under no circumstances should coring be continued when it is obvious that the core barrel is blocked. This can only result in a grinding down and loss of core. In zones which are highly fractured or where the barrel contiually becomes blocked it is essential that short runs be used even though this means removal of the entire string of drilling tools every foot or even every few inches unless a wireline core barrel is being utilized.

In general, core barrels are operated at speeds from 50 to 1750 rpm. Essentially, the harder the rock the faster the permissible speed. However, the ultimate factor determining the speed is the amount of vibration encountered as the speed is increased. Operating speeds in good rock are indicated by Acker (1956) to be 800 to 1200 rpm for EX and AX bits; up to 800 rpm for BX bits; and up to 600 rpm for NX bits.

Downward pressure on the bit is determined on the basis of experience. Rod vibration or "chatter" generally indicates the need for a reduced bit pressure. Coring is soft rock also requires low bit pressure.

The pressure under which the drilling fluid should be introduced into the hole is the minimum consistent with adequate removal of cuttings from the hole and proper cooling of the bit.

2.3 Drilling Equipment and Methods

The wide variation in the characteristics of rock materials and the conditions under which they must be sampled has led to the development of a wide variety of core barrel assemblies and special drilling methods in an effort to achieve high quality sampling and a high percent recovery. Efforts have been directed toward equipment which is capable of coring, within a single sampling operation, materials ranging from extremely soft to extremely hard and equipment in which erosion of the core by the drilling fluid is minimized. Devices to improve core retention also have received attention.

The core barrels currently in use are of three basic types, namely, the single tube, the rigid-type double tube, and the swivel-type double tube. A special type of single tube barrel, which is referred to as a shot core or calyx barrel, is also commonly used to obtain large-size cores. Core barrels are available in a variety of sizes ranging from those which produce cores having a nominal diameter of 13/16 of an inch to those producing 48-inch and larger cores. The Diamond Core Drill Manufactures Association (DCDMA) has established standards for the more commonly used sizes of diamond core drill equipment. These standards cover not only the core barrels and their component parts but also drill rods and casing. (Selected standards - Exhibit A3)

Basically, core barrel assemblies comprise, from top to bottom, the following components which are threaded to one another: a head section; tubular sections variously referred to as outer tubes or core barrels and inner tubes or inner barrels; liners; reaming shells; and core bits. Core retainers or lifters are devices located at the lower end of the barrel and designed to hold the core in the barrel. The nature of the reaming shells, core retainers, and coring bits is similar for most barrels. It is the arrangement of the core retainer and both the nature and arrangement of the remaining items which distinguish one type of barrel from another. Therefore, in the discussion which follows, coring bits, reaming shells, and core retainers are described separately; the remaining elements are described as they are related to one another in each of the specific types of barrel designated at the start of this section. Also described are wireline core barrels which were developed to increase the speed of drilling; a sampling procedure and the Integral Coring Method.

2.4 Coring Bits

The coring bit is the bottom most component of the core barrel assembly. It is the grinding action of this component which cuts the core from the rock mass. Three basic categories of bits are in use: diamond, carbide insert and sawtooth, described below.

Diamond coring bits may be of the surface set or diamond-impregnated type. In the former (Exhibit A4) (a) and (B)), the diamonds are set in the metal matrix at the interior and exterior faces of the bit near its bottom and also on the bottom or cutting face of the bit. The diamond-impregnated bit, on the other hand, has small pieces of diamond embedded throughout the metal matrix of the bit. The diamonds used for both types of bit are commonly West African, processed, or Congo bortz. Surface set bits of standard design, which will provide good performance under average conditions, are readily available from drill equipment manufactures. However, the wide variation in the hardness, abrasiveness, and degree of facturing encountered in rock has led to the design of bits to meet specific conditions known to exist or encountered at given sites. Thus, wide variations in the quality, size, and spacing of diamonds; the composition of the metal matrix; the face contour; and the type and number of waterways are found in bits of this type. Similarly, the weight of diamonds and the composition of the metal matrix of impregnated bits also are varied to meet differing rock conditions.

Diamond coring bits are the most versatile of all coring bits since they can produce high quality cores in rock materials ranging from soft to extremely hard. No other type of bit will produce a satisfactory core in extremely hard rock or in deposits comprising alternating layers of hard and soft rock. Compared to other types, diamond bits in general permit more rapid coring and, exert lower torsional stresses on the core. The latter permits the retrieval of longer cores and cores of small diameter. Compared to one another, the diamond-impregnated type of bit is particularly well adapted for use in drilling extremely abrasive materials which can cause the dislodging of the diamonds in the surface set bit. In such materials the impregnated bit has the advantage that as the bit is worn, new diamonds are exposed to continue the cutting action. On the other hand, the surface set bit can be reset as the bit becomes worn, whereas the impregnated bit is used until it is worn out.

The selection of the correct diamond size or type of crown is the key to fast and economical drilling. The type of drilling equipment and the technique used have some influence on the selection of diamond size, but primarily the correct diamond selection depends on the characteristics of the formations being drilled, not only the predominant rock type, but also the alterations, inclusions and structure of the rock.

The size of diamonds selected for use in a specific geological formation follows the general rule that a soft formation requires large stones and a hard formation smaller stones. Diamond sizes are always related to the number of stones in one carate weight of diamonds (1 carat = 0.2 gram). The hardness of the matrix in which the diamonds are set is also dependent on the hardness and abrasiveness of the formation, the machine RPM's and the bit pressure. To hard a matrix will cause the crown to become polished producing low penetration rates. To soft a matrix will produce loosened stones that will be either destroyed or washed out.

Responsibility for the final choice and use of diamond crown type usually lies with the drilling contractor. The choice will greatly influence his penetration rates and drilling costs.

Carbide insert bits are of several types. Two types, the standard and the pyramid, are shown in Exhibit A4 (c) and (d) respectively. These bits use tungsten carbide in lieu of diamonds to penetrate the material being cored. Bits of this type are used to core soft to medium-hard rock. They are less expensive than diamond bits. However, the rate of drilling is slower than with diamond bits.

In sawtooth bits, shown in Exhibit A4 (e), the cutting medium comprises a series of teeth which are commonly cut into the bottom of the bit. The teeth are faced and tipped with a hard metal alloy such as tungsten carbide to provide wear resistance and thereby to increase the life of the bit. These bits have the advantage of being less expensive than diamond bits. However, they do not provide as high a rate of coring and do not have a salvage value. The sawtooth bit is used primarily to core overburden and very soft rock.

An important feature of all bits which should be noted is the type of waterways provided in the bits for the passage of the drilling fluid. Bits are available with so-called "conventional" waterways, which are passages cut on the interior face of the bit (Exhibit A4), or with bottom discharge waterways, which are internal and discharge at the bottom face of the bit behind a metal skirt separating the core from the discharge fluid (Exhibit A4). Bottom discharge bits should be used when coring soft rock or rock having soil-filled joints.

2.5 Reaming Shells

The reaming shell is a metal sleeve, threaded at both ends, which serves as a coupling between the core barrel and the bit. It is slightly larger in diameter than the core barrel and its surface is set with diamonds, has insert strips with diamonds or has carbide insert strips. These are set to a diameter slightly larger than that of the coring bit. The shell thus reams the hole and thereby serves to maintain the gauge of the hole and to reduce wear on the bit as the barrel is moved in and out of the hole.

2.6 Core Lifters and Retainers

There are two devices commonly used to retain the core as the core barrel is removed from the borehole. These are the split-ring core lifter and the basket retainer. The split-ring lifter is a tapered split ring of tempered steel which is fluted on either its interior or exterior surface. It is held in place by the tapered inner face of the coring bit or the core lifter case. As the core passes through the bit the core lifter spreads to permit the core to enter the recovery tube. When the barrel is withdrawn from the hole and the core tends to slip down, the tapered shape of the core lifter causes the lifter to jam between the barrel and core and so grip the core. This type of lifter is used primarily to retain cores of sound rock.

The basket retainer, comprises a base ring to the periphery of which are welded curved strips of fingers of spring steel. These fingers initially rise vertically and then curve toward the center of the ring. The steel used may be stiff or extremely flexible. Retainers with stiff fingers are used when soft rock and dense or hard soil are being cored; flexible fingers are used when extremely soft or fine-grained soils are cored. The basket retainer may be held in place by the tapered inner face of a bit or may be set in a recess at the upper end of the bit which holds it in place against the bottom of the core recovery tube. In operation, the fingers of the basket retainer spread to permit the core to enter the recovery tube as the core barrel is advanced. If the core tends to slip out as the sampler is removed from the hole, the fingers dig into the core and retain it.

2.7 Single Tube Core Barrels

This is the most rudimentary, the least expensive and the most durable of the core barrels. The barrel from top to bottom consists of a core barrel head, a core barrel, a reaming shell, a core lifter, and a core bit.
Each component, except the core lifter, is threaded to the piece immediately above. The core lifter is held in place by the tapered inner surface of the coring bit and by the bottom of the reaming shell against which it bears when the core is passing through it.

In operation, the single tube barrel (Exhibit A5 - Fig. 137) is rotated as a downward force is applied and drilling fluid is introduced to the hole under pressure. The fluid flows through the drilling rods into the core barrel tube where it passes between the core and the walls of the core barrel to the bottom of the bit. From there it flows upward in the drill hole, between the barrel and the walls of the hole, carrying the cuttings to the surface. In some cases a piece of tube, called a sludge barrel or calyx is threaded to the top of the core barrel head. This serves to collect the larger particles of the cuttings which tend to drop from suspension as the drilling fluid flowing from the small annular space around the barrel enters the larger annular space around the drill rods above and has its velocity reduced.

Single tube core barrels of this type are available in sizes ranging from EWX size (Exhibit A3) to 6 1/2 inches o.d. and in lengths of 2, 5, and 10 feet. The smallest of the barrels is referred to as a starting barrel. This barrel is required when a hole is started directly in rock since the clearance between the chuck of the drilling rig and the ground surface normally is not adequate to accommodate a larger barrel.

The main disadvantage of the single tube core barrel is that the entire core is subject to the erosive action of the drilling fluid as it passes through the core barrel. Because of this, the use of the single tube barrel should be restricted to hard rock which is unaffected by the flow of the fluid.

2.8 Double Tube Core Barrels

The double tube core barrel was developed to minimize the erosive action of the drilling fluid on the core and thereby to improve core recovery. Double tube core barrels (Exhibit A5 - Figs. 138 & 139) fall into two basic categories, the rigid type and the swivel type. A variation of the swivel type known as the Series "M" barrel is extensively used.

The rigid type double tube barrel comprises a core barrel head, an outer barrel, an inner core recovery tube, a reaming shell, a core lifter, and a coring bit. In this barrel water passages in the head are so arranged that the drilling fluid reaches the coring bit via an annular space between the inner and outer tubes, both of which are rigidly attached to the head. In operation, both the inner and outer tubes turn when the rotary force is applied. As the coring bit at the bottom of the outer tube cuts the core, the core passes through the core lifter and into the inner tube which protects it from erosion by the drilling fluid. Holes at the bottom of the inner tube provide for a small flow of drilling fluid between the core and the wall of the inner tube in order to minimize the possibility of accumulating cuttings which would cause friction between the core and the tube and thereby cause large torsional forces to be applied to the core.

The rigid type double tube barrel is available in sizes from EWX to NWX with barrel lengths of 2, 5, 10, 15, and 20 feet (Exhibit A3). The major advantages of this type of barrel are that, in general it provides for protection of the core against the erosive action of the drilling fluid and will provide a higher percent recovery than the single tube barrel. It is used primarily in medium to hard rock which is not highly fractured.

The swivel type double tube barrel differs from the rigid type in that the inner tube remains stationary while the outer barrel is rotated. This minimizes the possibility of core disturbance through torsional forces and thereby improves recovery. Two forms of the swivel type barrel are available, the conventional and the Series "M" barrels. The upper portions of both barrels are similar and as illustrated (Exhibit A5) comprise an outer rotating tube, an inner stationary tube, and outer and inner tube heads. As in the rigid type, water passages direct the flow of the drilling fluid into the annular space between the two tubes and vents provide for the exit of water from the barrel. The inner tube assembly is suspended from the outer tube head in such manner that a downward force can be applied to both tubes while only the outer tube is rotated. In the sampler illustrated this is accomplished by the use of ball bearings; roller bearings also may be used.

The conventional and the Series "M" barrels differ from each other in the arrangement of the lower portion of the barrel. The inner barrel terminates above the core lifter which has a 5° taper and operates within the level section of the bit. The configuration for the Series "M" barrel is as shown in Exhibit A5. Here, the core lifter, which has a 2.5° taper and is thinner than the conventional lifter, operates within a lifter case attached to the bottom of the inner tube. This case has a thin wall bottom which extends almost to the cutting face of the coring bit. The arrangement used in the Series "M" barrel makes this barrel superior to the conventional barrel in two respects. In the conventional barrel the lifter may tilt and block the entrance to the inner tube or the lifter may rotate with the bit and cause grinding of the core. In addition, the portion of the core from the bottom of the bit to the bottom of the inner tube is exposed to the erosive action of the drilling fluid. In the Series "M" barrel the core is in the inner tube and somewhat aligned before it encounters the lifter and the lifter remains oriented since the inner barrel moves little, if at all. Therefore, the possibility for blocking or grinding is minimized. The effective extension of the inner barrel to the cutting face of the bit by the lifter case also minimizes the area of core exposed to the drilling fluid.

Conventional and the Series "M" barrels are available in sizes ranging from EWX and EWM t NWM, respectively (Exhibit A3). They may be obtained in lengths of 2, 5, 10, 15, and 20 feet.

The conventional barrels are used in coring fractured or broken formations which are of average hardness and which are not excessively susceptible to erosion. The Series "M" barrels are particularly adapted to achieving a high recovery in badly fractured or broken strata or in soft an friable formations which are easily eroded.

2.9 Large Diameter Core Barrels

In addition to the double tube core barrels discussed above, there are available large-diameter barrels which are similar in construction and operation to the Series "M" barrels. The large barrels are available in the following nominal sizes: 2 3/4" i.d. X 3 7/8" o.d., 4" o.d. x 5 1/2" o.d., and 6" o.d. 7 3/4" o.d. Actual dimensions are given in Exhibit A3. The two largest of these are equipped with a sludge barrel attached to the outer barrel head in order to collect large particles too heavy to be carried to the surface by the return flow of the drilling fluid. These particles tend to settle out of suspension and often cause barrels to be wedged in boreholes.

In general, the larger the size of a core the better the recovery. Consequently, the large-diameter barrels frequently are used when highly erodible material such as soft or friable rock is to be cored. Conversion units are available to permit the use of these barrels to obtain undisturbed soil samples in liners. Therefore, they may be used when it is necessary to obtain cores of overburden containing large particles and when large-diameter samples are required for testing.

2.10 Wire Line Core Barrels

The use of the core barrels described above requires the removal of the entire string of drill rods from a borehole whenever it is necessary to remove core from the barrel. In the drilling of deep holes, this is an extremely time-consuming operation which is partially eliminated by the use of the wire line core barrel (Exhibit A6). The wire line core barrel assembly consists of an inner barrel assembly which can be retrieved independently of the outer barrel assembly through the required special drill rod. The outer barrel assembly consists of a tube at whose lower end is a reaming shell that couples the tube to a diamond coring bit. The upper end of the tube is attached to the special large-diameter wire line drill rods. The retrievable inner barrel assembly consists of the core recovery tube with a core lifter at its lower end and a swivel type ball bearing head at its upper end. Attached to the top of the swivel head is a locking head with a spearhead at its upper end to permit retrieval of the barrel by a device called an overshot. The overshot has at its upper end a socket attaching it to a wire line and at its lower end a lifting dog which grips the spearhead during the retrieval operation. An optional feature of the wire line barrel is a water shut-off valve which is made a part of the inner barrel assembly. This valve is a rubber washer which, when a core block occurs, is squeezed out to fill the annular space between the outer and inner barrels. This causes the pump pressure to rise, thereby alerting the driller to the core block and permitting unnecessary grinding of the core to be averted.

In operation, the outer barrel, to which the inner barrel assembly is locked, is rotated to cut the core. When the run has been completed or a block occurs the hole is flushed and the drill string is broken at the first joint above ground. The overshot is lowered on the wireline and the lifting dog grasps the spearhead at the top of the inner tube. When the wire line is pulled, the latches which couple the inner and outer barrel assemblies are disengaged and the inner barrel assembly maybe lifted to the ground surface. The inner tube assembly may be returned to the bottom of the hole in several ways. In a dry hole it must be lowered with the overshot. In holes filled with water the assembly may be dropped to the bottom. However, in deep holes and relatively flat inclined holes water circulation is used to pump the assembly into place. The drill string and outer barrel assembly are removed only when it is necessary to replace a bit.

Wire line core barrels are available in sizes which produce cores 1 1/16, 1 7/16, 1 7/8, 2 1/2, and 3 11/32 inches in diameter. Core barrel assemblies are available in 5, 10, and 15 foot lengths.

In addition to increasing the rate of progress, the wire line barrel has the following advantages: (1) informations which are prone to caving the danger is reduced since the drill string and outer barrel are not removed after each run and the opportunity for loosening of material in the walls of the hole is thereby decreased; (2) bit life is increased by reducing the number of times the bit must core through caved material on reentry into the hole; and (3) if the barrel has the optional water shut-off valve, grinding of the core is decreased and recovery thereby increased.

2.11 Integral Coring Method

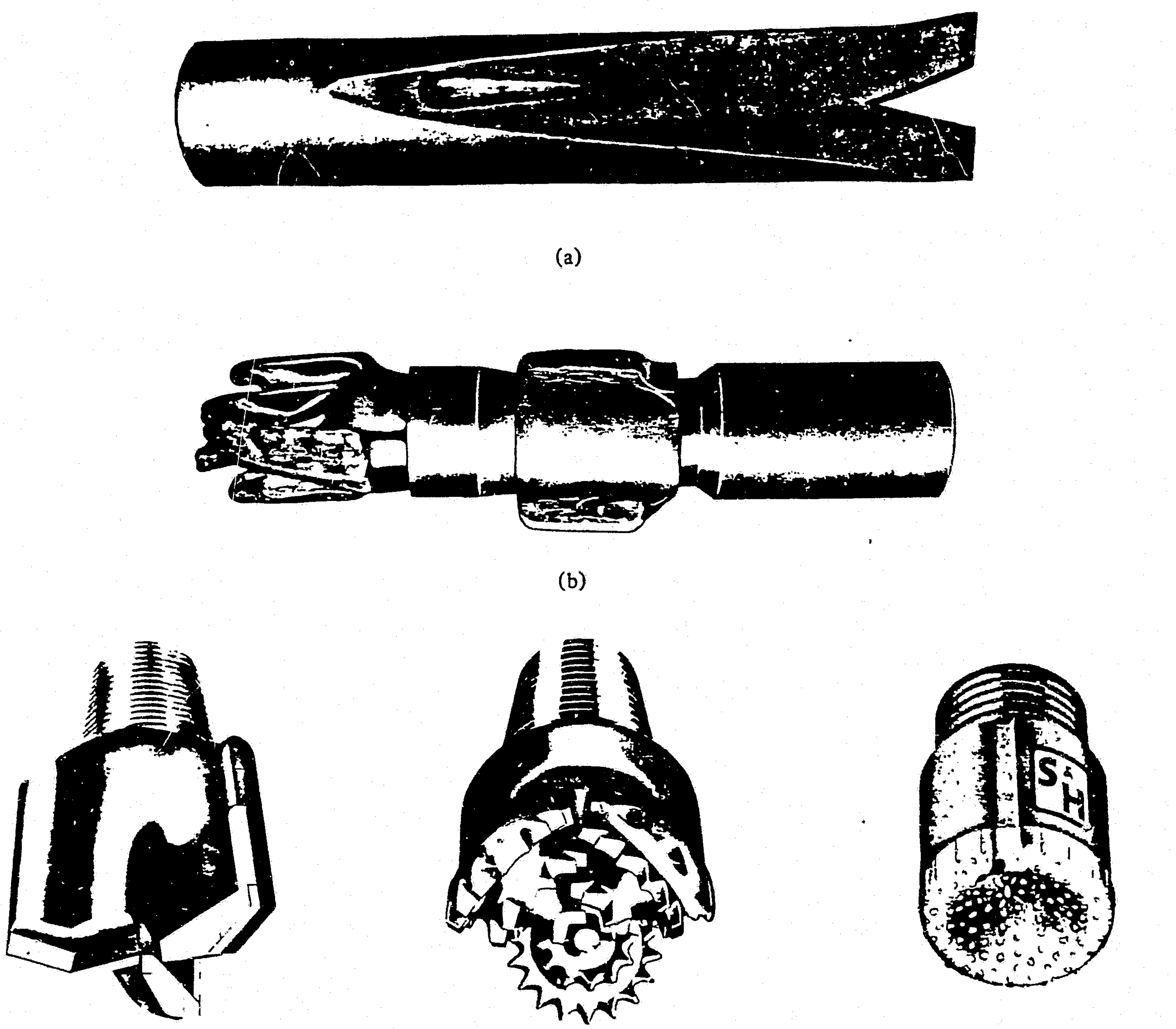
This sampling technique was developed in response to the need for rock cores truly representative of in situ rock masses including their discontinuities such as open, tight, or clay-filled joints, shear zones, and cavities. The method consists of taking a core at whose center is a steel rod previously inserted and bonded to the rock mass to hold the mass together during the coring operation. This technique, when properly used, produces 100 percent recovery and provides oriented cores. The method may be used to obtain cores throughout the entire length of a boring or a selected locations. The procedure used is as follows. A hole with a minimum diameter of 7.5 cm is drilled to the depth at which integral coring is required. A second, small-diameter hole is then drilled coaxially with the first hole and extending from the bottom of the first hole to a depth equal to the length of the required core. The diameter of the second hole must be large enough to accommodate a pipe of adequate stiffness to minimize deformation of the core during the coring but should be as small as feasible to increase the thickness of the annular sample recovered. The majority of integral coring done to date has been accomplished using boreholes 7.5 cm in diameter with reinforcing pipe holes 2.6 cm in diameter. The length of the coring run must be as long as possible to minimize the cost but it must not be so long as to result in difficulties due to hole deviation.

CHARACTERISTICS OF TYPICAL ROTARY DRILLING RIGS

MAKE AND MODEL OF RIG	CAPACITY								ROTARY FEED				WEIGHT[6]
	RATED DEPTH CAPACITY[1]					HOIST				CHUCK[5]			
	ROD SIZE (DCDMA STDS.)					CABLE CAPACITY	CABLE SIZE	ENGINE[2] POWER	LENGTH[4]	NO. OF SPEEDS	RANGE	SIZE	
	EW	AW	BW	NW	HW	FT.	IN.	H.P.	IN.		RPM	IN.	LBS.
Acker Packsack	100(RW)	—	—	—	—	—	—	10	—	1	3600	—	115
Smit Winkie	400	350	150	—	—	—	—	10		2	1200-2800		225
Central Mine Equipment 45C	—	500	—	—	—	—	—	37[3]	68	4	75-475	1-7/8	3600
Sprague & Henwood 37	700	600	500	400	—	65	3/8	17	24	3	505-1400	1-15/16	1600
Longyear 24	890	675	—	—	—	65	5/16	12	24	9	224-2173	1-7/8	1000
Acker Hillbilly	1250	1000	750	650	—	165	3/8	22.5	24	4	155-1000	2-15/16	2100
Chicago Pneumatic 8 HD	1250	1000	750	650	—	125	3/8	27.5	24	4	225-1050	2-3/4	2800
Sprague & Henwood 40C	1650	1475	1000	900	—	175	3/8	33	24	4	235-1500	3	2900
Mobile B-50	—	1400	—	1000	—	—	—	97[3]	78	15	33-900	2-3/4	4500
Acker Teredo	1800	1300	1250	1000	—	150	1/2	36.8	24	4	141-900	2-15/16	2900
Failing 1500S	—	1500	—	—	—	525	1/2	55	30	3	73-220	2-3/8	22000[7]
Central Mine Equipment 55	—	—	—	1000	—	—	—	124[3]	72	4	75-650	2-3/4	6000
Longyear 34	—	1575	1275	1000	625	190	1/2	36	24	8	20-1000	3	3100
Chicago Pneumatic 15	2250	2000	1350	1150	—	125	1/2	34	24	4	225-1050	2-3/4	3300
Mobile B-61	—	2000	1500	—	—	400	7/16	97[3]	68	4	65-850	4-3/4	8200
Joy Ram Rod II	—	—	1850	1500	—	125	1/2	47.6	30	8	35-2000	2-3/4	3100
Sprague & Henwood 142C	2250	3500	3000	2400	—	200	1/2	49.7	24	4	215-1250	3	4300
Joy 22	—	4000	3100	2500	—	180	5/8	47.6	24	9	95-1200	3-5/8	4400
Longyear 44	—	4000	3200	2500	1600	200	1/2	59	24	12	205-2200	3	4500
Acker Presidente	—	4500	3800	3000	—	150	5/8	51	24	8	93-1120	3	6100
Chicago Pneumatic 50	—	—	6500	5000	—	120	1	130	31	∞	0-1250	3-9/16	15800

NOTES:

(1) Rated capacity is greater if wire line equipment is used.
(2) Horsepower ratings vary with RPM. Most models are offered with a choice of horsepower. A choice of gasoline, diesel, air, electric, LPG or power takeoff is also available for many models.
(3) Supplies drive for auger also.
(4) Other lengths are available on some models.
(5) Other chuck sizes and RPM ranges are available on some models.
(6) Weight varies with the options requested.
(7) Includes weight of truck.



(a) (b) (c) (d) (e)

Rotary bits: (a) fishtail bit; (b) Hawthorne replaceable blade drag bit; (c) carbide insert drag bit; (d) tricone bit; (e) diamond plug bit. (*Courtesy of Sprague & Henwood, Inc.*)

EXHIBIT A2

STANDARD SIZES OF DRILL TOOLS

Drill Rods – Flush Coupled

SIZE	O.D. In.	I.D. In.	WEIGHT Lbs./Ft.	COUPLING I.D. In.	O.D. mm	I.D. mm	WEIGHT Kg./Ft.	COUPLING I.D. mm
E[1]	1-5/16	7/8	2.7	7/16	33.3	22.2	12.0	11.1
A[1]	1-5/8	1-1/8	3.[illegible]	9/16	41.3	28.6	17.2	14.3
B[1]	1-7/8	1-1/4	5.0	5/8	47.6	31.7	21.1	15.9
N[1]	2-3/8	2	5.2	1	60.3	50.8	22.5	25.4
EW[2]	1-3/8	15/16	3.1	7/16	34.9	23.8	14.1	11.1
AW[2]	1-3/4	1-1/4	4.2	5/8	44.4	31.8	19.5	15.9
BW[2]	2-1/8	1-3/4	4.3	3/4	54.0	44.5	20.0	19.0
NW[2]	2-5/8	2-1/4	5.5	1-3/8	66.7	57.1	25.2	34.9

[1] Original diamond core drill tool designations
[2] Current standards of the Diamond Core Drill Manufacturers Association (DCDMA)

Casing – Flush Jointed – DCDMA Standards

SIZE	O.D. In.	I.D. In.	WEIGHT Lbs./Ft.	O.D. mm	I.D. mm	WEIGHT Kg./Ft.
EW	1-13/16	1-1/2	2.76	46.0	38.1	1.25
AW	2-1/4	1-29/32	3.80	57.2	48.4	1.73
BW	2-7/8	2-3/8	7.00	73.0	60.3	3.18
NW	3-1/2	3	8.69	88.9	76.2	3.95

Casing – Flush Coupled – DCDMA Standards

SIZE	O.D. In.	I.D. In.	WEIGHT Lbs/Ft.	COUPLING I.D. In.	O.D. mm	I.D. mm	WEIGHT Kg./Ft.	COUPLING I.D. mm
EX	1-13/16	1-5/8	1.80	1-1/2	46.0	41.3	0.82	38.1
AX	2-1/4	2	2.90	1-29/32	57.2	50.8	1.32	48.4
BX	2-7/8	2-9/16	5.90	2-3/8	73.0	65.1	2.68	60.3
NX	3-1/2	3-3/16	7.80	3	88.9	81.0	3.54	76.2

Casing – Standard Drive Pipe

NOMINAL SIZE In.	O.D. In.	I.D. In.	WEIGHT Lbs./Ft.	COUPLING O.D. In.	NOMINAL SIZE mm	O.D. mm	I.D. mm	WEIGHT Kg./Ft.	COUPLING O.D. mm
2	2-3/8	2-1/6	5.5	2-7/8	50.8	60.3	52.4	2.49	73.0
2-1/2	2-7/8	2-15/32	9.0	3-3/8	63.5	73.0	62.7	4.08	85.7
3	3-1/2	3-1/16	11.5	4	76.2	88.9	77.8	5.22	101.6
3-1/2	4	3-9/16	15.5	4-5/8	88.9	101.6	90.5	7.03	117.3
4	4-1/2	4-1/32	18.0	5-3/16	101.6	114.3	102.4	8.17	131.8

Casing – Extra Heavy Drive Pipe

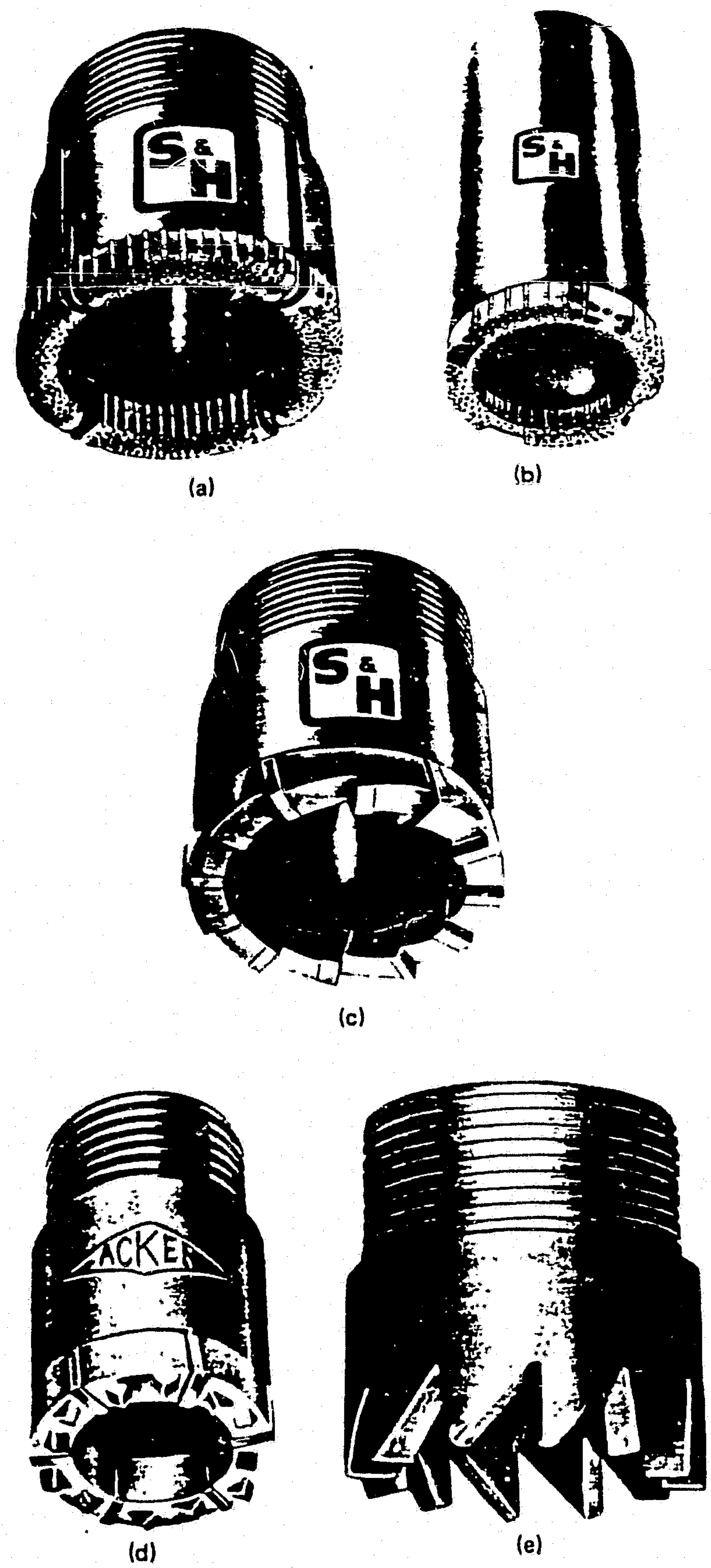
NOMINAL SIZE In.	O.D. In.	I.D. In.	WEIGHT Lbs./Ft.	COUPLING I.D. In.	NOMINAL SIZE mm	O.D. mm	I.D. mm	WEIGHT Kg./Ft.	COUPLING I.D. mm
2	2-3/8	1-15/16	5.0	2-7/32	50.8	60.3	49.2	2.27	56.4
2-1/2	2-7/8	2-21/64	7.7	2-5/8	63.5	73.0	59.1	3.50	66.7
3	3-1/2	2-29/32	10.2	3-1/4	76.2	88.9	73.8	4.63	82.5
3-1/2	4	3-23/64	12.5	3-3/4	88.9	101.6	85.3	5.68	95.3
4	4-1/2	3-53/64	15.0	4-1/4	101.6	114.3	97.2	6.81	107.9

Diamond Core Bits

DCDMA STANDARDS				
SIZE	CORE DIAM.[1] In.	HOLE DIAM.[2] In.	CORE DIAM.[1] mm	HOLE DIAM.[2] mm
EWX & EWM	0.845	1.485	21.5	37.7
AWX & AWM	1.185	1.890	30.0	48.0
BWX & BWM	1.655	2.360	42.0	59.9
NWX & NWM	2.155	2.980	54.7	75.7
2-3/4" x 3-7/8"	2.690	3.875	68.3	98.4
4" x 5-1/2"	3.970	5.498	100.8	139.6
6" x 7-3/4"	5.[illegible]70	7.7[illegible]	151.6	196.8

WIRE LINE				
SIZE	CORE DIAM.[1] In.	HOLE DIAM.[2] In.	CORE DIAM.[1] mm	HOLE DIAM.[2] mm
AQ Wire Line	1-1/16	1-57/64	27.0	48.0
BQ Wire Line	1-7/16	2-23/64	36.5	60.0
NQ Wire Line	1-7/8	2-63/64	47.6	75.8
HQ Wire Line	2-1/2	3-25/32	63.5	96.0
PQ Wire Line	3-11/32	4-53/64	85.0	122.6

[1] I.D. of Core Bit Set.
[2] O.D. of Reaming Shell Set.

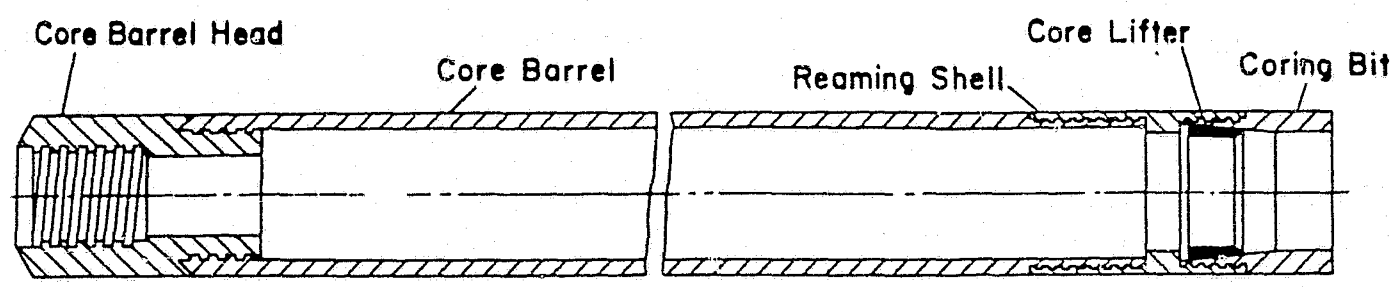


Coring bits: (a) diamond with conventional waterways; (b) diamond with bottom discharge waterways; (c) carbide insert, blade type; (d) carbide insert, pyramid type; (e) sawtooth. (*Courtesy of Sprague & Henwood, Inc. and Acker Drill Co., Inc.*)

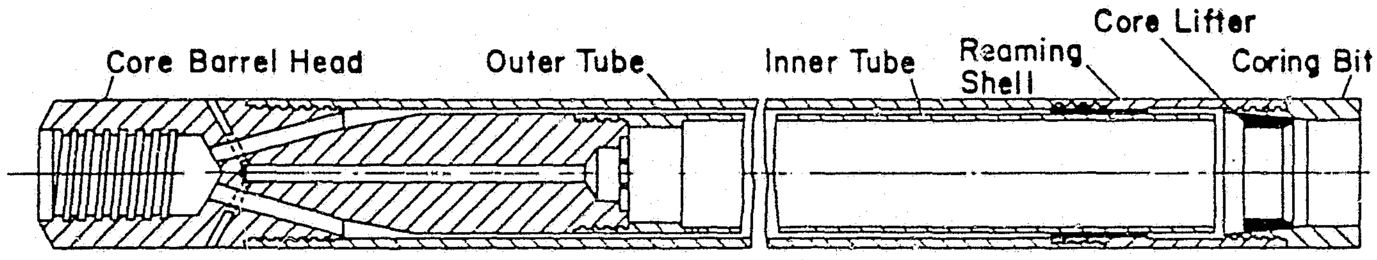
ROCK HARDNESS	ROCK TYPE	DIAMOND SIZE
Soft Rock	Chalk Calcrete Tuff Shale	2 - 8 stones per carat or Sawtooth bits
Hard Rock	Limestone Siltstone Sandstone Slate Dolomite	8 - 16 stones per carat or Sawtooth bits 16 - 30 stones per carat
Very Hard Rock	Schist Gneiss Diorite	30 - 60 stones per carat
Extremely Hard Rock	Gabbro Basalt Andesite Syenite Granite Felsite Quartzite Banded Ironstone	60 - 150 stones per carat or Impregnated bits

GENERAL RECOMMENDATIONS FOR BIT SELECTIONS

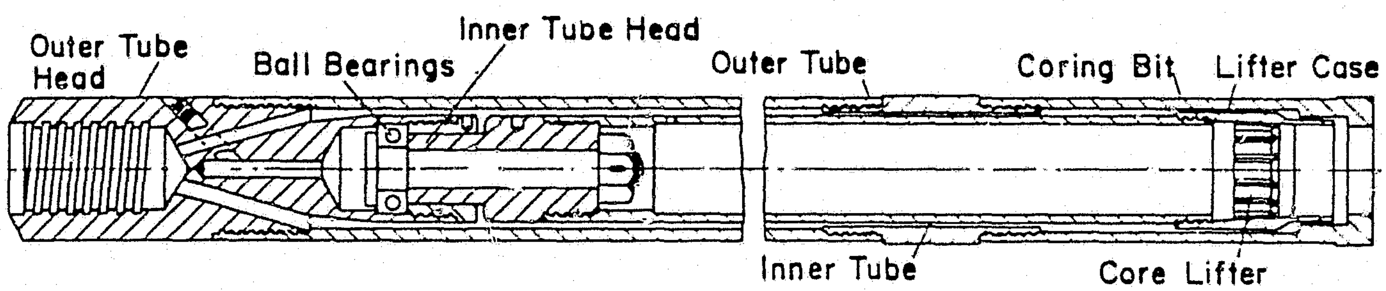
EXHIBIT A4



Single tube core barrel. (*Courtesy of Sprague & Henwood, Inc.*)

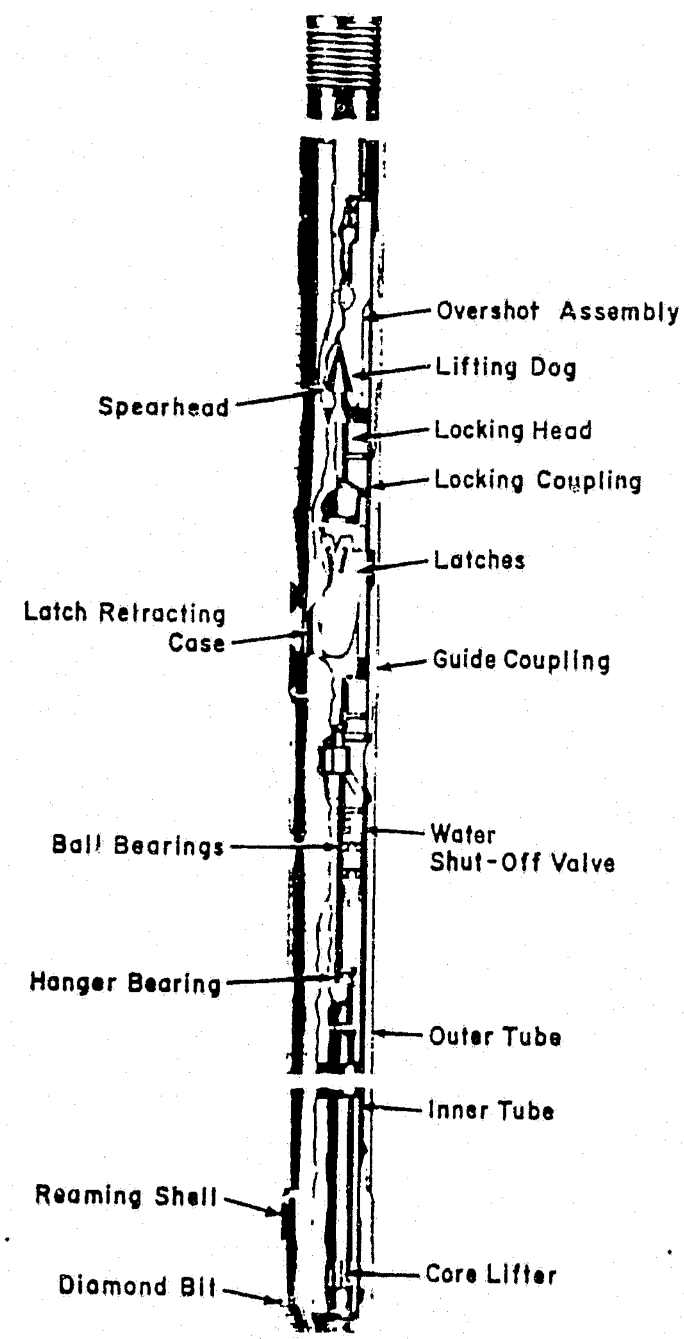


Rigid type double tube core barrel. (*Courtesy of Sprague & Henwood, Inc.*)



Swivel type double tube core barrel, series "M" with ball bearings. (*Courtesy of Sprague & Henwood, Inc.*)

EXHIBIT A5



Wire line core barrel and overshot assembly. *(Courtesy of E. J. Longyear.)*

EXHIBIT A6

APPENDIX B
ROCK CORE STORAGE AND PHOTOS

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1.0 CORE STORAGE

Rock cores are stored in partitioned boxes (Exhibit B-1). The cores are boxed in the same sequence in which they were taken from the drill hole. Their arrangement in the boxes is as follows. With the core box opened so that the hinged cover is away from the viewer and the partitioned section is adjacent to him, the core is arranged in order of decreasing elevation starting at the left end of the partition nearest the hinges, proceeding to the right and continuing from left to right in succeeding partitioned areas. Core boxes are numbered in sequence with Box No. 1 containing the core of highest elevation. The cores from each drilling run are separated from the core from adjacent runs by wooden blocks on which the depths of the beginning and end of the run are clearly and permanently marked. Blocks, also are used to indicate core loss. If the loss can be pinpointed, the block is placed at the depth of the loss; otherwise it is placed at the end of the run in which the loss occurred. Labelling of the box is discussed in a subsequent paragraph.

It also is common practice to preserve some cores by sealing them in waxed paper or aluminum foil. This is done to preserve representative samples of the materials encountered, cores of material susceptible to slaking, and cores containing soil-filled fractures or fault zone gouge. The method used is merely to wrap the alluminum foil around the core and seal the ends or the entire core by dipping in microcrystalline wax.

Soil cores which are not required to be preserved in an undisturbed condition may be wrapped and sealed as described above and stored in the same type of box as rock cores.

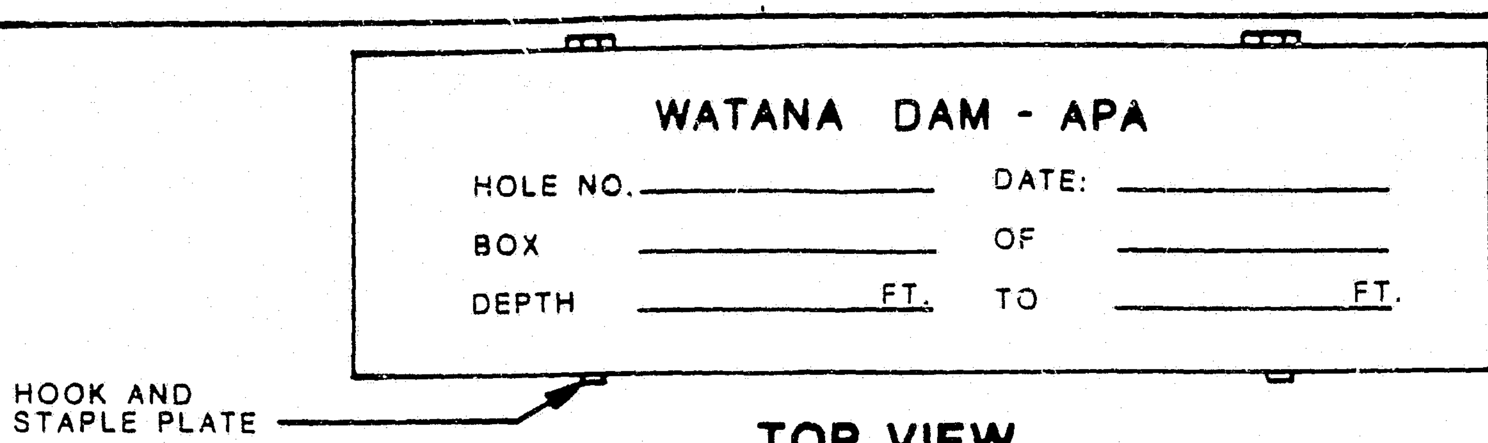
2.0 LABELLING

All samples must be clearly marked so as to leave no doubt as to their exact source. The information shown should include the project name; exploration identifying number such as the boring; sample number; top elevation of the hole; depth of the sample below ground surface; description of the material; etc.

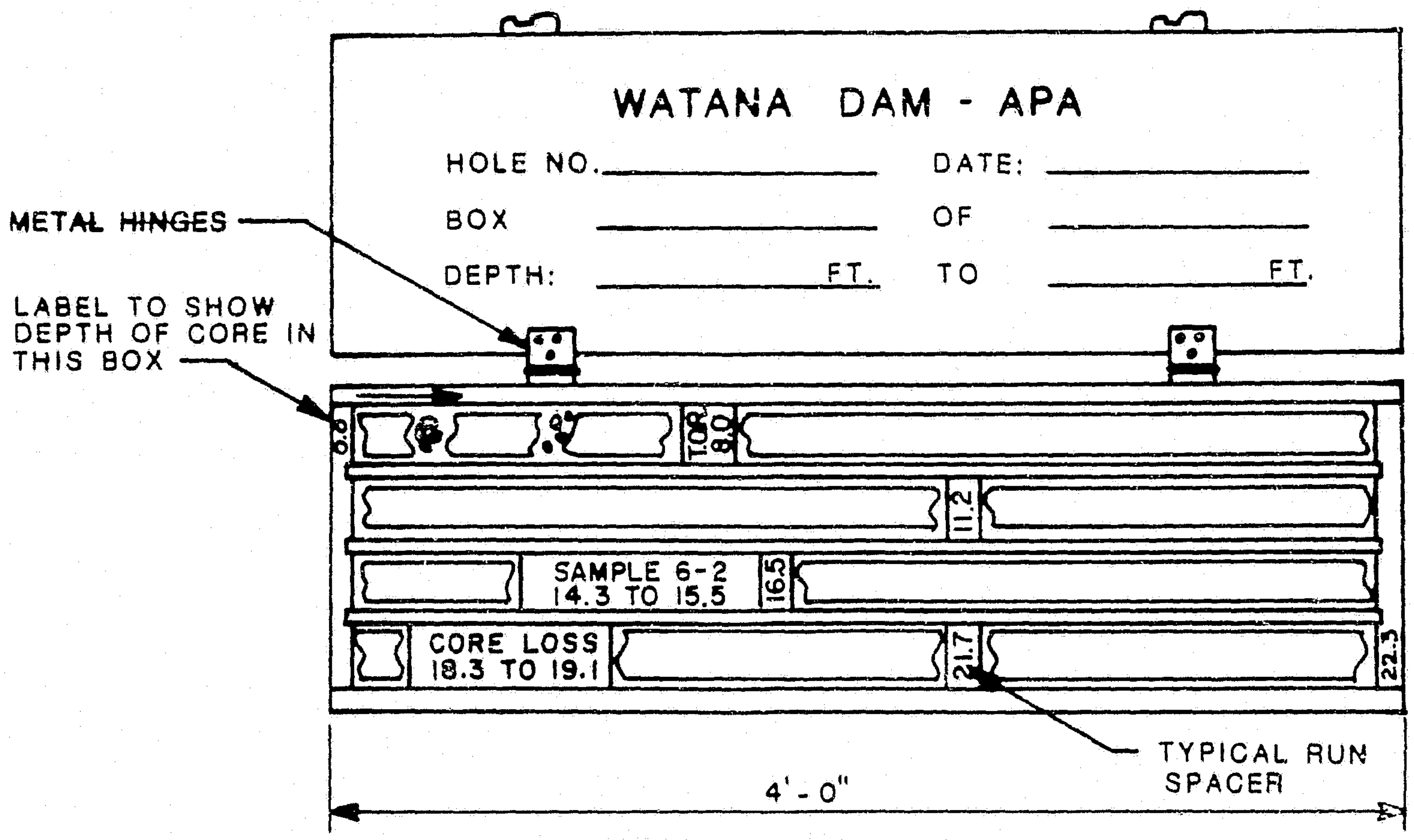
The project name and the box number should be stenciled on both ends of the core box as well as the cover. This makes identification more efficient when the boxes are stacked.

3.0 CORE PHOTOGRAPHS

Color photographs of the cores in each box will be obtained as soon as practical. This provides an excellent record of the "as retrieved" condition of the cores, which is particularly important in the case of air slaking materials, and permits the design engineer to review the nature of the rock, as required, at subsequent times. Also, the photographs provide a record of the correct sequence of the core pieces in case the core box is spilled accidentally or cores are not returned to their proper place by persons examining them. The photographs should be taken from directly above the box and should include the inside of the cover of the core box which contains the project name, boring number, box number and depth covered by the box. A maximum of two core boxes and preferably one box should be included in each photograph. If the cores have dried out prior to taking the photographs, the cores should be wet with a light water spray or a damp cloth to acentuate the color of the cores.



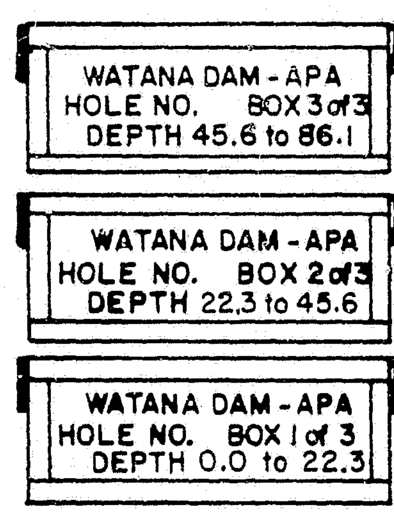
TOP VIEW



VIEW WITH BOX OPEN

NOTES:

1. THE CORE BOX SHALL BE CONSTRUCTED OF WOOD.
2. START PLACING CORE IN UPPER LEFT-HAND CORNER OF BOX AND FINISH RUNS IN LOWER RIGHT-HAND CORNER OF BOX AS SHOWN BY ARROWS.
3. BOXES TO BE MARKED AS SHOWN. A STENCIL SHALL BE USED ON THE OUTSIDE AND INSIDE OF COVER.
4. INSERT A 2"X2" (INCH) ORANGE CORE LOSS BLOCK WHEREVER LOSS OCCURS. LENGTH TO CORRESPOND TO LENGTH OF LOST CORE.



END VIEW

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

CORE BOX DETAILS

HARZA-EBASCO SUSITNA JOINT VENTURE ANCHORAGE, ALASKA

APPROVED

DATE FEB 84

CONTRACT NUMBER

EXHIBIT B-1

APPENDIX C
PIEZOMETER DEVICES

TABLE OF CONTENTS

1.0 BASIC CONCEPTS

1.1 Groundwater Levels and Pore Pressures

A Regular Conditions. The free groundwater level or table is defined as the elevation that the free water surface assumes in a hole extending a short distance below the capillary zone, as illustrated in Exhibit C1, where the water surface is at equilibrium with atmospheric pressure. The capillary zone is defined as the interval between the free water surface and the limiting height above which water cannot be drawn by capillarity. Groundwater conditions are described as "regular" when the pore water pressure increases hydrostatically with depth below the groundwater level. A condition of hydrostatic pressure is defined as that pore pressure equal to the product of the unit weight of water and the vertical distance from the observation point of the groundwater surface adjacent to the piezometer tube. The condition is illustrated in Exhibit C2 (a).

B. Irregular Conditions. When the pore water pressure does not increase hydrostatically with depth below the groundwater level, groundwater conditions are described as "irregular." These conditions may result from perched water tables caused by relatively impermeable strata above the main groundwater level. The presence of more pervious and better drained strata below the groundwater level may cause irregular groundwater conditions. Subsurface features that cause irregular groundwater condtions are illustrated in Exhibit C1 (b) and (c).

C. Variation in Groundwater Levels and Pressures. Groundwater levels and pressures are rarely constant over an extended period of time. Natural forces such as precipitation, evaporation, freezing, and seepage may cause wide variations in the groundwater level. Minor changes in groundwater level may also be caused by variations in atmospheric pressure. The pore water pressure is considered to be under positive excess pressure when the pore water pressure at a point is more than hydrostatic and to be under subhydrostatic pressure when the pore water pressure is less than hydrostatic. The two conditions are illustrated in Exhibit C2 (b) and (c) respectively.

(1) Artesian Pressure. Artesian pressures are found in strata that are confined between impervious strata and are connected to a water source at a higher elevation. A well drilled to a stratum having a pore water pressure above the ground surface will flow without pumping and is called a free-flowing artesian well.

(2) Induced Pore Water Pressure. Pore water pressure may be temporarily changed from a hydrostatic condition as a result of stress changes. Stress changes may be caused by such activities

as construction loading or unloading, by induced vibrations, or by natural forces such as erosion, deposition, and earth tremors. Stress changes are accompanied by strains as the soil mass adjusts to the new stress. The stress-strain adjustment in the soil mass is accompanied by changes in the soil skeleton whereby pore spaces are either reduced or increased. When a reduction of void spaces occurs, the increased stress is transferred directly to the pore water, or to the gas-water combination in a partially saturated soil mass, resulting in an excess pore water pressure. This excess pressure is rapidly dissipated in pervious soils with free drainage outlet and is generally of little concern. In impervious soils or in pervious soils surrounded by impervious soils, the excess pressure may dissipate slowly and since the stress is being borne largely by the pore water, serious loss of soil shearing resistance may result. A tendancy toward increase in void spaces may reduce pore pressures to subhydrostatic levels.

2.0 TYPES OF PIEZOMETERS

2.1 Basic Types of Piezometers. Piezometers can be classified into three basic types, depending on the principles used to activate the device and transmit the data to the point of observation. A brief description of the basic types of piezometers follows.

A. Open Standpipe (Open-System) Piezometers. This type consists of a vertical pipe or tube having a porous tip at the lower end. The water level is measured by lowering a probe on a tape or scaled line until it contacts the water surface. The probe may be electric or acoustic. There are several variations of the electric probe such as that shown in Exhibit C3 available commercially, but all are similar in priciple. The device consists of an ohmmeter connected by insulated wires to a weighted sounding probe not more than 1/4 in. in diameter. Additional weights may be added at intervals along the wire to facilitate lowering the probe. Approximately 1/4 in. of each conductor is stripped on the lower end of the probe. The probe tip should be inspected to ensure that the exposed conductors are not touching and are free of dirt or corrosion. Contact with the water surface in the piezometer closes the circuit and is registered on the ohmmeter. A simple sounding probe such as a short piece of pipe may be suitable for many shallow installations. An open-system piezometer can be converted to a closed-system type, as described in subparagraph B.

B. Hydraulic (Closed-System) Piezometers. This type consists of one or two pipes or tubes filled with fluid and a porous tip; the piezometer is connected to a Bourdon gage or a manometer at the observation point. In the two-tube type, the second tube or pipe serves as a means of flushing the device to remove accumulated gas or sediment.

C. Remote Sensing Piezometers. The remote sensing piezometer is a cylindrical cell with an impermeable diaphragm protected from soil contact by a porous tip that allows access of pore water and/or pore air pressure to the diaphragm. Remote sensing piezometers can be either pneumatic or electric, depending on the means provided for determining the pressure on the diaphragm. In pneumatic piezometers, two tubes leading from an observation station connect to two openings in the cell body. Pore pressure exerted on the cell causes the diaphragm to close or open, depending on the design. To measure the pore pressure, Nitrogen or CO_2 is introduced into one tube until the pressures on both sides of the diaphragm are balanced, thus allowing the air to return to the observation station through the second tube. Gases or fluids may be used instead of air. Diaphragm deflection caused by pore pressure may be measured electrically by means of electric strain gages or a vibrating wire attached to the diaphragm and monitored by appropriate equipment at the observation station. Remote sensing piezometers have also been developed that use a combination of pneumatic and electric means for determining the pore pressure on the diaphragm. A gas is introduced into the cell through one tube until the pressures on both sides of the diaphragm are balanced as indicated by a broken or completed electrical circuit, depending on the design.

3.0 MEASUREMENT OF PORE GAS AND PORE WATER PRESSURES

a. Effect of Pore Gas Pressures. It has been recognized in recent years that failure to consider pore gas pressure as separate from pore water pressure in partially saturated soils can lead to significant errors in the determination of effective stresses, especially in the case of impermeable soils with a low degree of saturation. Pore gas pressures are higher than adjacent pore water pressures because of tension in the menisci surrounding a gas bubble.

b. High and Low Air Entry Filters. The air entry value of a filter is defined as the difference in pounds per square inch (psi) between gas (usually air) pressure on one side of a saturated filter and water pressure on the other side when blow-through of air occurs as the air pressure is slowly increased. A low air entry filter has an air entry value in the order of 1 to 1-1/2 psi and a high air entry filter a value of 30 psi or more. If the air entry value of a filter in a closed-system hydraulic piezometer is low, gas from the soil enters the filter and water is drawn from it into the soil (especially during attempts to drain the system). The pressure measured by the piezometer in this case tends to be pore air pressure. This appears to apply also to electrical type piezometers. In the high air entry filter, only pore pressure minus the pore water pressure in the soil exceeds the air entry value of the filter by about 30 psi or unless the pore water pressure in the system has a negative value large enough to cause cavitation; in this case, water in the filter is drawn into the surrounding soil and gas is then free to pass through the filter. Gas will diffuse through the high air entry filter over a long period of time, but use of this type of filter together with tubing that is impermeable to air has drastically reduced the amount of de-airing formerly necessary to maintain an air-free system. Depending on the intended use of the piezometric data, both pore gas and pore water pressures may be determined separately by installing two piezometers in proximity to each other, one with a high air entry filter and one with a low air entry filter. Early installations of hydraulic piezometers in embankments were equipped with low air entry filters that allowed passage of air into the filter. As the air pressure is greater than the water pressure due to the meniscus effect at the air-water interface, the piezometers tended to record abnormally high piezometric levels reflecting pore air pressures instead of pore water pressures. Frequent de-airing of the devices, with attendant adverse effects, was necessary. In many more recent installations, high air entry filters have been used that successfully excluded air from the measuring chambers of the devices for a long period of time.

4.0 DESCRIPTION OF DEVICES

4.1 General

Brief descriptions, including schematic drawings, of various types of piezometers are presented below. A list of some commonly used piezometers together with the manufacturer or U.S. supplier is given in Exhibit C4. Advantages and limitations of the various piezometers are listed in Exhibit C5. Foreign-made devices, if selected for use, should be procured in complete units, as threads and sizes of connecting components may not be compatible with standard U.S. items.

4.2 Open-System Piezometers

(1) Casagrande (Porous Tube). The Casagrande piezometer consists of a standpipe of an appropriate length of 3/8-in.-inside diameter (ID) saran tubing connected to a fine-grade Norton porous tube. A schematic of the tip is shown in Exhibit C6a. For ease of lowering the sounding probe into the tubing, 1/2-in.-ID saran tubing is sometimes used. However, this increases the time lag. The piezometer is usually installed vertically in a borehole and a sand filter is placed around the porous tip. The time lag of the Cassagrande piezometer is relatively short compared with other open standpipe piezometers installed in soils having coefficients of permeability k as low as 10^{-5} centimeters per second (cm/sec).

(2) Twin-tube Casagrande. In order to overcome the restrictions of piezometer location and the vulnerability of a vertical riser tube to damage during embankment construction, the Casagrande piezometer can be modified as shown in Exhibit C6b to permit offsetting the upper portion of the riser. Observations are made by slowly applying air pressure to the inner tube until the pressure becomes constant, indicating that air is escaping from the lower end into the outer tube; at this time, the applied air pressure is assumed to equal the pore water pressure. An

objection to this system, in addition to those listed in Exhibit C5, is that water in the inner tube must be forced either into the outer tube or into the soil surrounding the tip and an excessively high pore pressure will be measured unless sufficient time is allowed for pressure equalization.

(3) Norwegian Geotechnical Institute (Geonor). The Geonor piezometer, shown in Exhibit C6c, is a drive-point type device with E-rod threads. The device is usually operated as an open-system piezometer but can be converted to a single- or twin-tube, closed-system piezometer by attaching a Bourdon gage to the tubing. Its disadvantages, in addition to those listed in Exhibit C5 for open-system piezometers, are (a) the soil is disturbed in the vicinity of the point by driving, which increases the stress adjustment time lag (installation time lag); (b) an effective seal between the porous point and the upper soil strata often is not obtained; and (c) a sand filter cannot be provided around the porous point so that the time lag for the piezometer, except in very pervious soils, may be much longer than for other open-type piezometers installed in boreholes.

(4) Wellpoint. The wellpoint piezometer consists of a perforated tip connected to the lower end of a standpipe. The perforated tip may be a standard commercial well screen as shown in Exhibit C6d; a perforated section of the standpipe such as one fabricated by the Portland District as shown in Exhibit C6b; or a plastic polyvinyl chloride (PVC) pipe tip with sawed slots, drilled holes, or similar perforations that provide a screen tip. The standpipe may also be saran tubing or PVC (Schedule 80) pipe. PVC pipe is less susceptible to pinching off, verticality is easier to maintain, and the water level probe is more readily lowered into the pipe than into the flexible tubing. However, pipe connections must be made with extreme care as leaking standpipe joints usually go undetected. PVC pipe is preferable to metal pipe in areas having corrosive groundwater. When observations are not being made, the upper end of the standpipe is capped to prevent entry of foreign matter into the piezometer. The cap has a hole drilled in it to provide ventilation. The device may be installed in a borehole or driven into the ground to the desired elevation. Driven piezometers should be used only as an expedient means of obtaining groundwater information and may not be used in lieu of well-designed piezometer systems. In addition to the advantages and limitations listed in Exhibit C7 for open standpipe piezometers, the wellpoint piezometer is probably the least expensive, but it should not be used for measuring varying pore pressures in soils having permeabilities less than 10^{-4} cm/sec.

4.3 Hydraulic (Closed-System) Piezometers

(1) U.S. Bureau of Reclamation (USBR). Two types of closed-system, hydraulic piezometers have been developed by USBR: one for use in foundations and one for use in embankments

(Exhibit C7). The foundation type has a single ceramic filter disc, either high or low air entry, mounted in the bottom of the plastic body and twin tubes entering through the top. The embankment type has two ceramic filter discs, either high or low air entry, mounted on the top and bottom of the plastic body with twin tubes entering opposite sides.

(2) Bishop (Imperial College). The Bishop piezometer, shown in Exhibit C8, is a twin-tube hydraulic type piezometer having a tapered ceramic filter. The piezometer may be used in either foundations or embankments. When used in embankments, the piezometer tip is placed in a shallow hole formed by a shaped steel mandrel to fit the tapered filter so that a better contact is obtained with the soil.

4.4 Remote Sensing Piezometers

(1) Warlam. The Warlam cell, shown in Exhibit C9a, is a pneumatically operated diaphragm piezometer. Pore pressure measurements are made by introducing air under pressure through the air entry line at the observation station unitl the pore pressure acting on the opposite side of the diaphragm is slightly exceeded as indicated by escape of air through the vent line. The air pressure is then reduced until the escape of air ceases, at which time the air pressure in the entry line is assumed to be equal to the pore water pressure.

(2) Hall. The Hall hydrostatic pressure cell, shown in Exhibit C9(b), consists of a stainless steel body housing a ceramic filter, a stainless steel membrane, and a stainless steel piston resting on the membrane. The steel piston has two gas passages from top to bottom. Twin nylon tubes, sealed in the upper end of the gas passages and encased in a protective tube, connect the cell to the observation station. The O-rings mounted on the side of the piston prevent leakage of water into the upper cavity of the cell. Dry nitrogen gas under pressure is introduced into one tube until the diaphragm deflects sufficiently to allow the passage of gas from the inlet tube past the stainless steel diaphragm into the other tube. The minimum pressure required to maintain a constant rate as measured by a small flowmeter is assumed to be equal to the pore water pressure.

(3) Dames and Moore. The Dames and Moore piezometer, shown in Exhibit C9(c), is pneumatically operated. One of the twin tubes is connected to a pressure gage, and the other is connected to a compressed air source. To measure the pore water pressure, air is introduced through one tube into the pressure chamber until the diaphragm is forced away from the piezometer body sufficiently to allow air to escape through the excess pressure bypass into the surrounding polyethylene vent and protective tubing. As the air escapes, the pressure indicated on the pressure gage becomes constant and is assumed to be equal to the pore water pressure. A biasing spring incorporated in the piezometer cap permits measurement of negative pore pressures.

(4) Terra Tec. The Terra Tec Company has developed a pneumatic diaphragm piezometer, the Thorpiezo, which is shown in Exhibit C9(d). The body of the Thorpiezo is constructed of polyethylene. The belofram is of Dacron and Buna N synthetic rubber. Springs are of silicone bronze with a baked Teflon coating. A neoprene O-ring is used as a checking seal. No exposed metal of any type is used underground, either in the instrument or in any of the external fittings. The connection between the instrument and the control station is of heavy-wall-nylon tubing jacketed in polyvinyl chloride. The Thorpiezo measures the air pressure required to close a hydra-pneumatic balance system within the unit. Air pressure from the control unit is applied through line 2A. Lines 2 and 2A comprise the balance system during the pressure buildup wherein each line has the same pressure. When the supply pressure equals the pore pressure, the O-ring check closes. Pore pressure is then measured on line 2 at the control unit. Since the O-ring check is closed, the pressure on line 2A can continue to increase without affecting the pore pressure being monitored on line 2. In order for the O-ring check to close, a very small displacement of gas or liquid must occur. The maximum required displacement of 0.05 cubic centimeter (cc) is compensated for by a 1/16-in. line which is normally vented to atmosphere. However, even if the line were closed off, the displacement is so slight that it would not cause any appreciable time lag in developing the actual pore pressure reading.

(5) Gloetzl. The Gloetzl cell, shown in Exhibit C10a, operates on the principle of a hydraulic relief valve. The pressure required to cause a liitle oil (about 1 cc/min) to pass the valve diaphragm and return to the observation station through the back flow line is considered to equal the pore water pressure acting on the soil side of the diaphragm. Monitoring equipment consists of a small oil reservoir, a constant volume pump, and a pressure gage. A mixture of 90 percent kerosene and 10 percent No. 10 weight nondetergent motor oil is recommended for the hydraulic fluid.

(6) Terrametrics. The Terrametrics pore water pressure cell, shown in Exhibit C10(b), operates on the principle of a hydraulic relief valve with the added feature of an initial prepressure (or zero pressure) to offset the installed hydraulic head of the readout hydraulic fluid. Readout tubes with a vertical differential of up to 500 feet may be compensated for with a proper adjustment of the spring at the time of assembly for the initial prepressure. The pressure required in the inflow tube to cause a little oil (about 1 cc/min) to pass the valve diaphragm and return to the observation station through the back flow line is considered to equal the pore water pressure acting on the soil side of the diaphragm. Monitoring equipment consists of a small oil reservoir, a constant volume pump, and a pressure gage. A mixture of 90 percent kerosene and 10 percent No. 10 weight nondetergent motor oil is recommended for use as the hydraulic

fluid. A nondetergent oil must be used because a detergent oil when mixed with kerosene will cause a sludge to form in the lines in a short time. The cell is stainless steel, except the Kapton diaphragm, and the nylon or steel tubing.

(7) Carlson. The Carlson piezometer is shown in Exhibit C11(a). Diaphragm deflection causes an increase or decrease in the tension of an elastic wire strain meter, one end of which is attached to the diaphragm and the other end to the cell body. The intensity of stress in the elastic wire at the time of observation is measured as a change in resistance ratio of two coils from that determined by calibration of the cell prior to installation. Temperature changes in the cell are compensated for by measuring the series resistance of the two coils and using a cell constant determined during calibration of the cell. The space between the porous stone and diaphragm is filled with water prior to installation to decrease the time lag.

(8) WES Transducer. The WES gage, shown in Exhibit C11b, measures pore water by means of a Consolidated Electrodynamics Corporation (CEC) pressure transducer mounted in a brass housing. The diaphragm, which is part of the transducer unit, is protected by a porous stone filter. The CEC pressure transducer, shown in Exhibit C14, employs an unbonded wire strain gage for measuring deflection of the diaphragm. The piezometer is calibrated prior to installation to determine cell constants. The piezometer is very sensitive to changes in pore water pressure.

(9) University of Alberta GSC Piezometer. A transducer piezometer, shown in Exhibit C11(c), was designed by Brooker and works in a manner similar to the WES cell. The piezometer is very sensitive and has been found to be particularly suitable for measurement of pore pressures in clay shales.

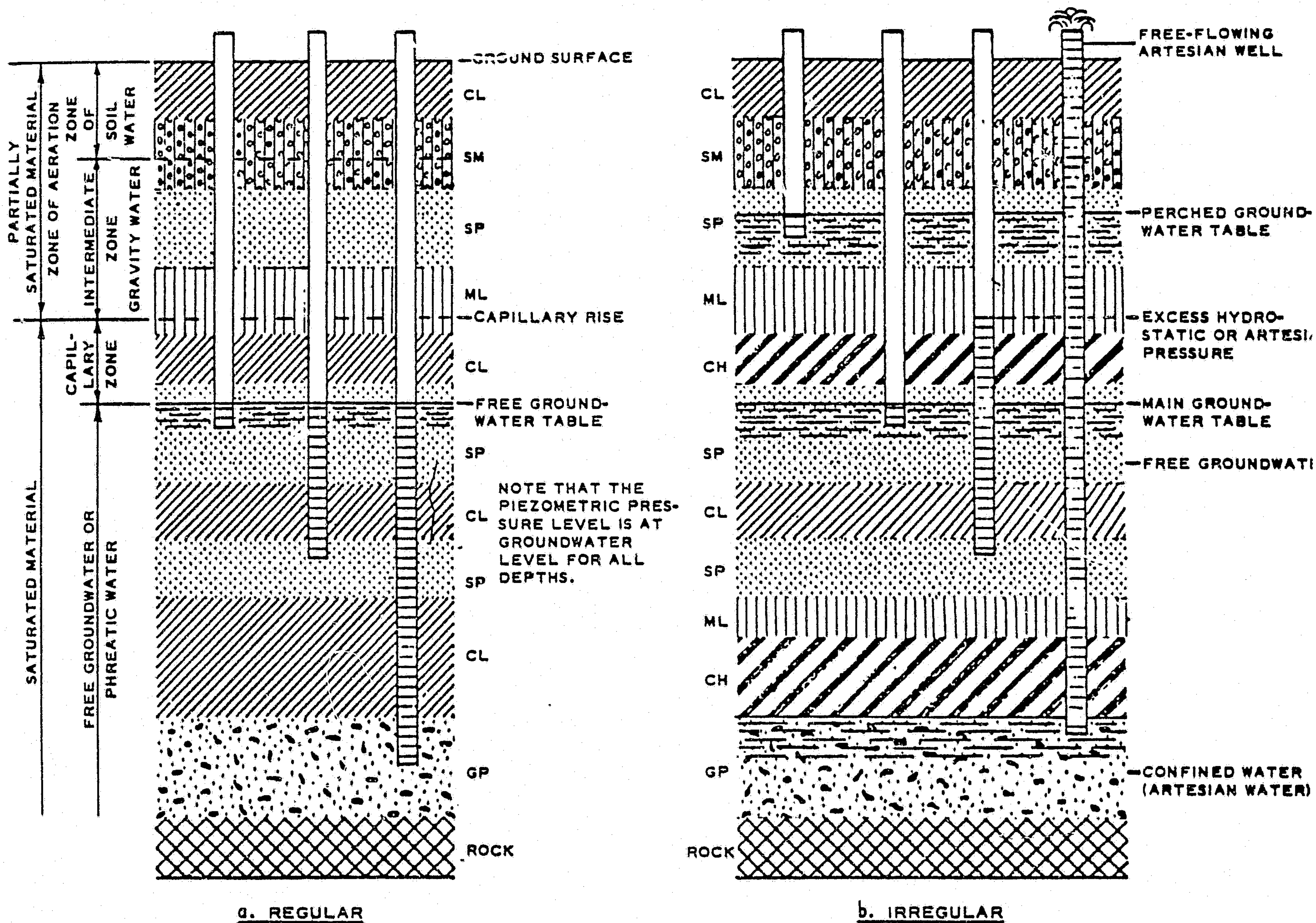
(10) Slope Indicator. The pore pressure transducer developed by Slope Indicator Company is shown in Exhibit C12. In this device the pore water pressure acts upon a rolling, flexible diaphragm having negligible spring force. Small movement of the diaphragm due to water pressure causes a sensitive ball check valve to open. Air pressure is applied to the input fitting causing flow through the valve, the diaphragm chamber, and into the output lines which are connected to a suitable pressure gage. Flow through the valve increases pressure in the two lines until it equals the pore water pressure. When the forces on the diaphragm become equal, the diaphragm will move slightly in the other direction allowing the check valve to close. At this point, the pressure in the output line equals the pore water pressure. Pressure can be increased in the input line, but there will be no flow and therefore no change in the output pressure.

(11) Maihak Cell. The Maihak piezometer, shown in Exhibit C13(a), is manufactured by Maihak AG in Germany. The exposed side of the diaphragm is protected by a fine-grained porous stone filter that allows access of pore water pressure to the diaphragm.

As the diaphragm is stressed by pore pressure, tension in the pretensioned wire is reduced. To measure pore pressure, the wire is caused to vibrate by an electrical impulse to the electromagnet. The frequency of the vibrating wire, which is dependent on the amount of tension, is measured at the observation station by means of a receiver equipped with a cathode ray tube; the frequency is converted to pore pressure by use of a calibration curve for the device.

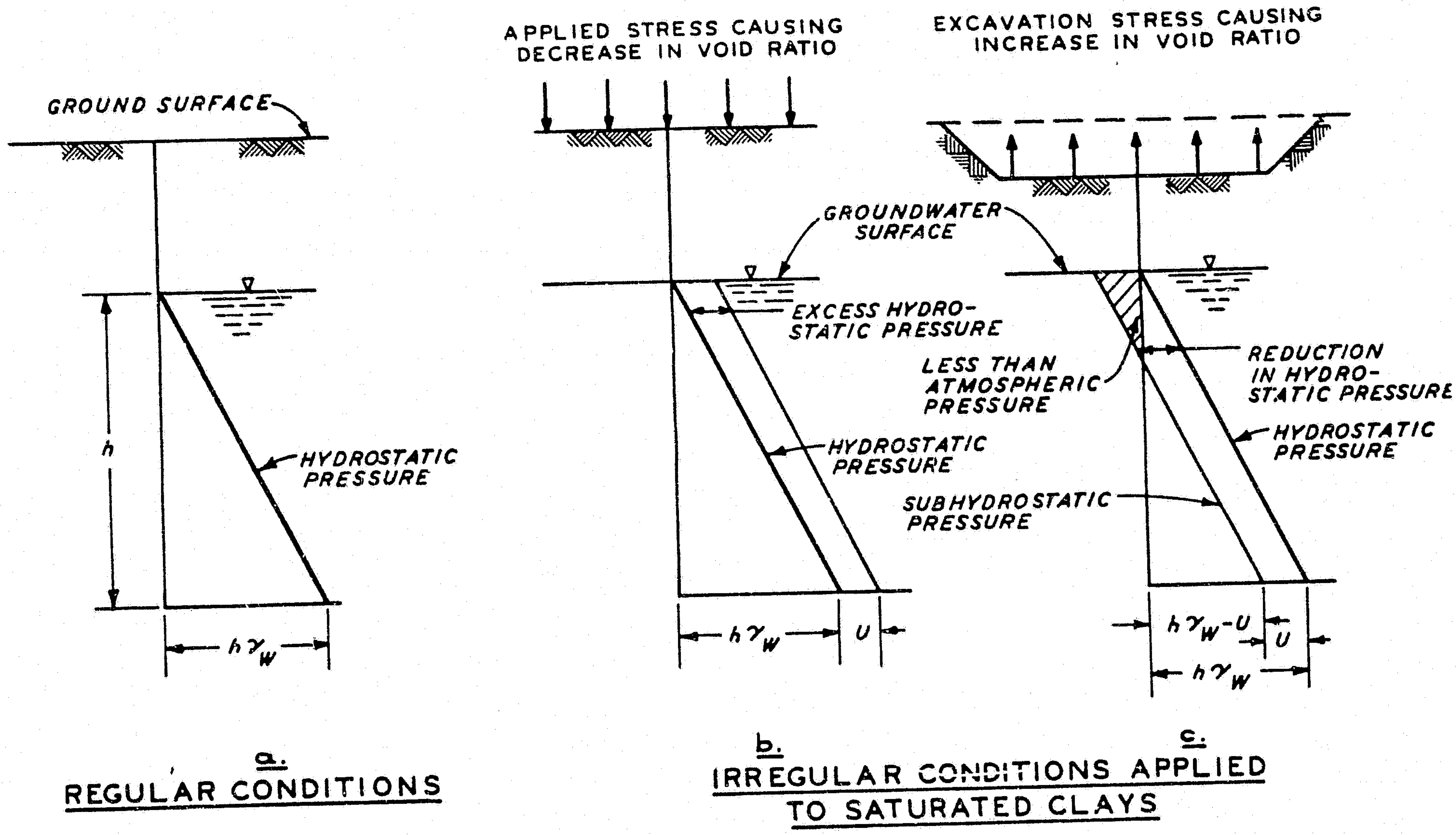
(12) Telemac. The Telemac cell, shown in Exhbit C13(b), developed by Telemac in France, measures the frequency of a vibrating wire under tension. The cell differs from the Maihak cell in two principle respects: the pretensioned wire is anchored inside a compressible steel sensing tube in contact with the diaphragm, and the measurement is made audible at the receiver where the pitch is matched with a sound from a calibrated source. Some difficulty may be experienced in matching the pitch because of background noise.

(13) Geonor. The Geonor vibrating wire cell, shown in Exhibit C13(c), was developed by the Norwegian Geotechnical Institute. The frequency of the vibrating wire is measured by a counter. Any standard frequency counter may be used in conjunction with a special attenuator available from the manufacturer.

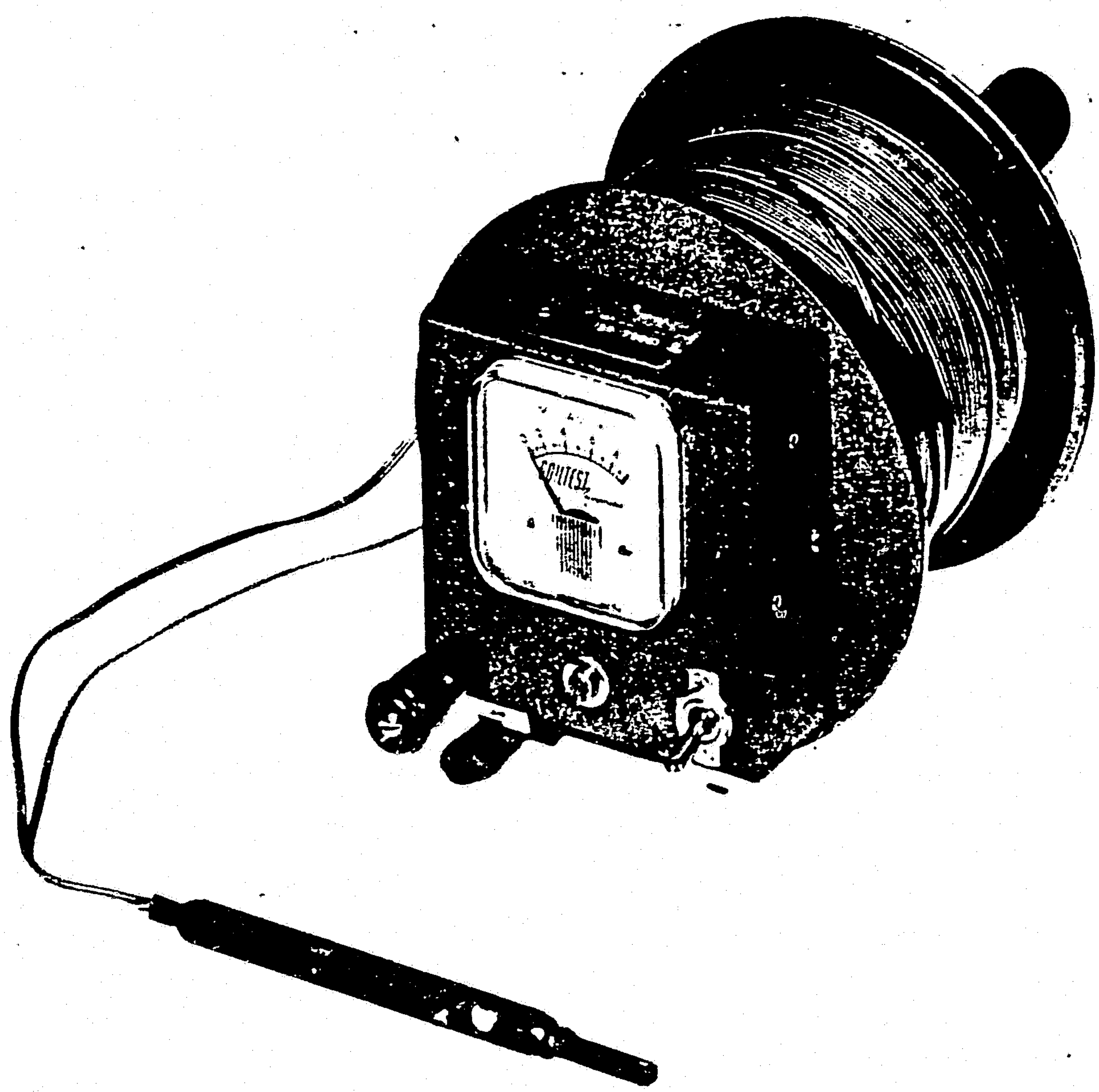


Groundwater conditions, after Hvorslev

EXHIBIT C1



Pore water pressure conditions



Courtesy of Soiltest, Inc.

Electric probe

EXHIBIT C3

Commonly Used Piezometers

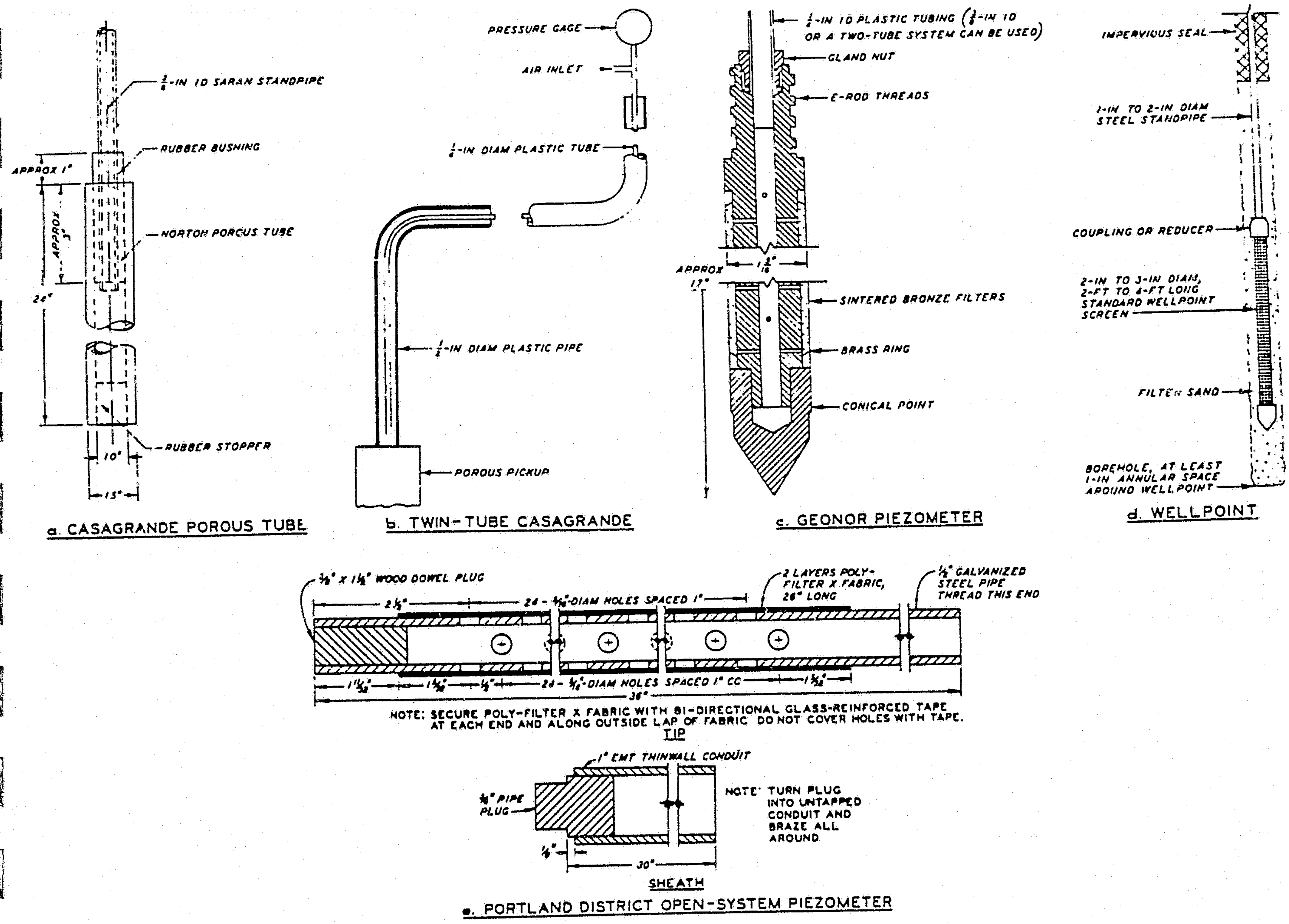
Type	Name	Manufacturer or U. S. Supplier[1]
Open-system	Casagrande	Locally fabricated or several suppliers
	Geonor	Soil and Rock Instrumentation, Inc. 377 Elliot St. Newton Upper Falls, Mass. 02164
	Wellpoint	Local suppliers
	Portland District	Locally fabricated
Closed-system	USBR	Plasticrafts, Inc. 2800 North Speer Blvd. Denver, Colo. 80211
	Bishop	Soil Instruments, L Townsend Lane London NW9, Englai
Diaphragm		
Pneumatic	Warlam	A. A. Warlam Box 122 Saddle River, N. J.
	Hall	Geo-Testing, Inc. P. O. Box 959 San Rafael, Calif. 94
	Dames & Moore	Dames & Moore 2333 West 3rd St. Los Angeles, Calif.
	Terra Tec Thorpezio	Terra Tec, Inc. 250 N. E. 49th St. Seattle, Washington 9
Hydraulic	Terrametrics Hydrostatic Pore Pressure Cell	Terrametrics 16027 West 5th Ave. Golden, Colo. 80401
	Gloetzl	Terrametrics
Electric strain gage	Carlson	Terrametrics
	Wes Transducer	U. S. Army Engineer Wa..., Experiment Station P. O. Box 631 Vicksburg, Miss 39180
	University of Alberta, GSC	Locally fabricated
Electropneumatic	Pore Pressure Transducer	Slope Indicator 3668 Albion Place North Seattle, Wash. 98103
Electrical acoustical	Maihak	Soil and Rock Instrumentation, Inc.
	Telemac	Soil and Rock Instrumentation, Inc.
	Geonor	Soil and Rock Instrumentation, Inc.

(1) Piezometers may be available from manufacturers or suppliers other than those listed.

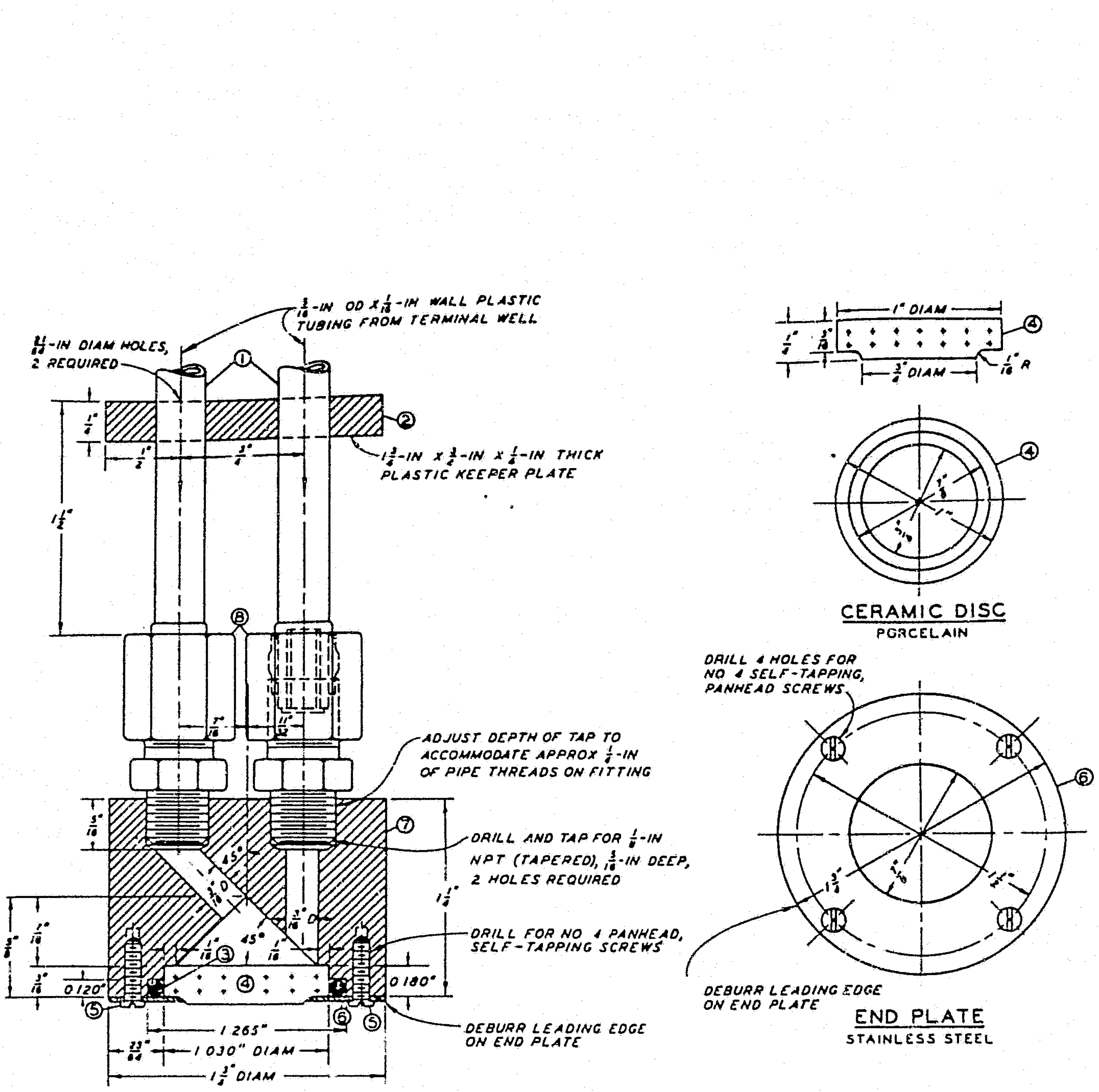
Note: Many of these piezometers are proprietary items and must be procured from the designer or his authorized supplier.

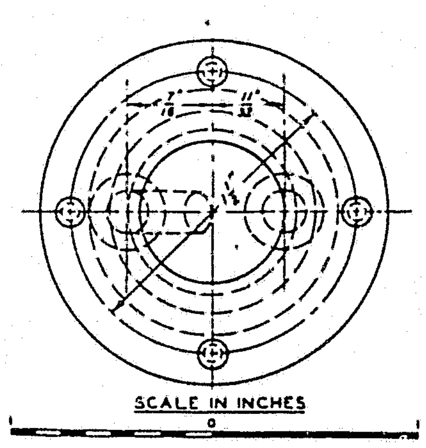
Comparison of Piezometers Types

Basic Type	Advantages	Disadvantages
Open-system	Simple; comparatively inexpensive; generally not subject to freezing; relatively long life; fairly easy to install; long history of effective operation.	Long time lag in impervious soils; cannot measure negative pore pressure; cannot be used in areas subject to inundation unless offset standpipe is used; must be guarded during construction; no central observation station is possible; requires sounding probe.
Closed-system	Small time lag in any soil; can measure negative pore pressures; can be used in areas subject to inundation; comparatively little interference with construction; can be read at central observation stations.	Observation station must be protected against freezing; fairly difficult to install; fairly expensive compared to open systems; sometimes difficult to maintain an air-free system; most types are fragile; some types have limited service behavior records.
Diaphragm	Simple to operate; elevation of observation station is independent of elevation of piezometer tip; no protection against freezing required; no de-airing required; very small time lag.	Limited performance data, some unsatisfactory experience; some makes are expensive and require expensive readout devices; fragile and requires careful handling during installation.
	Pneumatic. Electrical source not required; tip and readout devices are less expensive than for electrical diaphragm types.	Often difficult to detect when escape of gas starts; negative pressures cannot be measured; condensation of moisture occurs in cell unless dry gas is used; requires careful application of gas pressure during observation to avoid damage to cell.
	Electrical. Negative pressures can be measured.	Devices subject to full and partial short-circuits and repairs to conductors introduce errors; some makes require temperature compensation and have problems with zero drift of strain gages; resistance and stray currents in long conductors are a problem in some makes.



Schematic diagrams of open-system piezometers





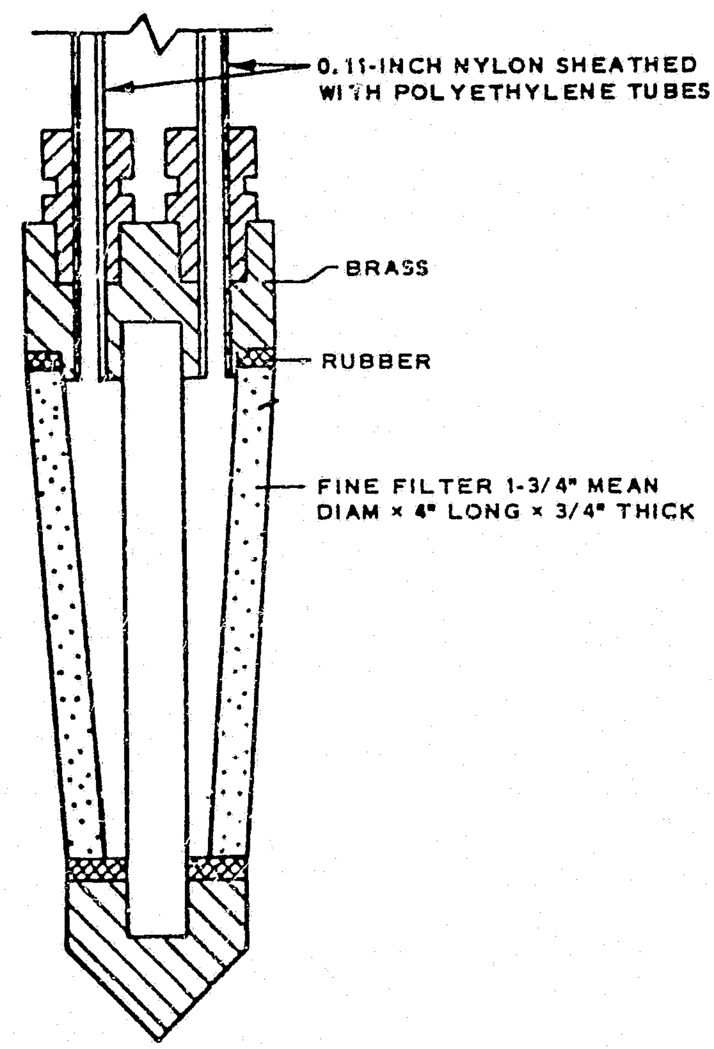
FROM USBR DRAWING NO 40-D-5819

LIST OF PARTS

PART NO	DESCRIPTION	LENGTH REQD	MATERIAL	NO REQD
(b) 1	TUBING, 3/8-IN OD BY 1/16-IN WALL	—	PLASTIC (PVNC)	—
(c) 2	KEEPER PLATE (SEE NOTE ON DWG)	—	PLASTIC (PP)	1
(a) 3	"O" RING, 1 1/4-IN OD BY 1-IN I D	—	RUBBER	1
(f) 4	FILTER DISC, 1-IN DIAM X 1/4-IN THICK	—	PORCELAIN	1
(g) 5	SCREWS, PANHEAD, NO 4 SELF-TAPPING	3/8-IN	STAINLESS STEEL	4
6	END PLATE, NO 18 GAGE (SEE DWG)(0 050-IN)	—	STAINLESS STEEL	1
(c) 7	TIP BODY, 1 3/4-IN DIAM	—	PLASTIC (PP)	1
(d) 8	COMPR CONN, M, 3/8-IN OD TUBE TO 1/8-IN PIPE	—	BRASS	2
(e) 9	INSERT, BRASS, FOR TUBING, PART 1	—	BRASS	2

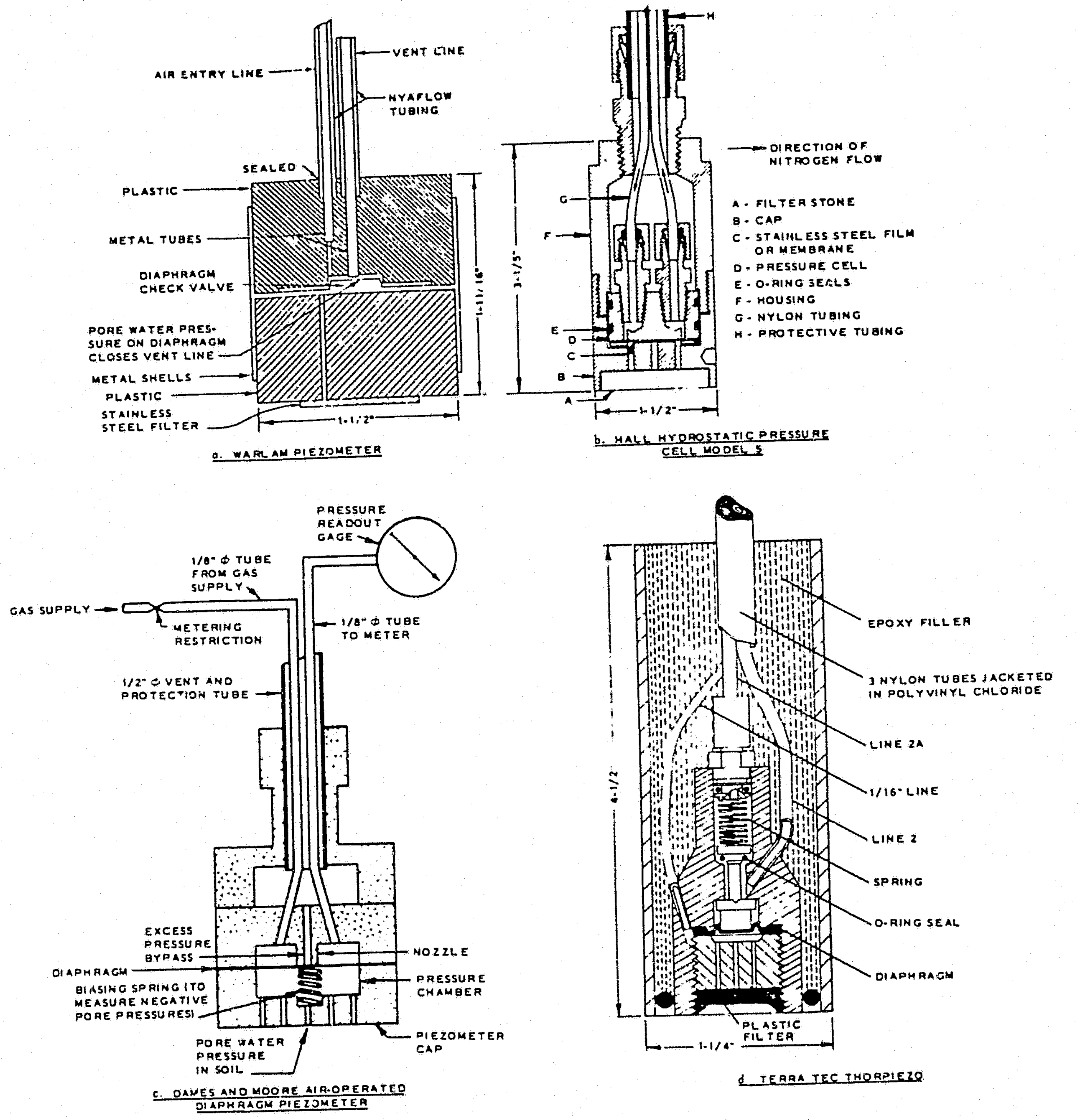
a GARLOCK SYNTHETIC RUBBER "O" RING 24849-19
b PVNC, POLYVINYLIDINE CHLORIDE (SARAN) OR EQUAL
c PP, POLYPROPYLENE
d COMPR CONN NO 5-FBUB-8, PARKER
e PARKER INSERT NO 5-T23UI-8
f. FILTER DISC, COORS PORCELAIN, P-3, 1 3 TO 2 0 MICRON PORE DIAM
g STOVE-HEAD SCREWS MAY BE SUBSTITUTED FOR PANHEAD (BINDING) SCREWS.

USBR hydraulic piezometer, foundation type



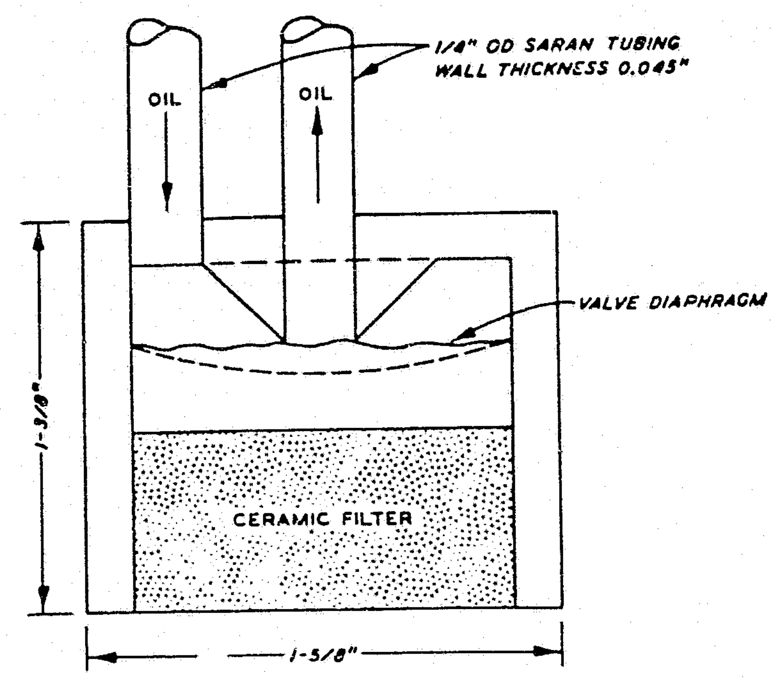
Schematic diagram of
Bishop hydraulic piezometer

EXHIBIT C8

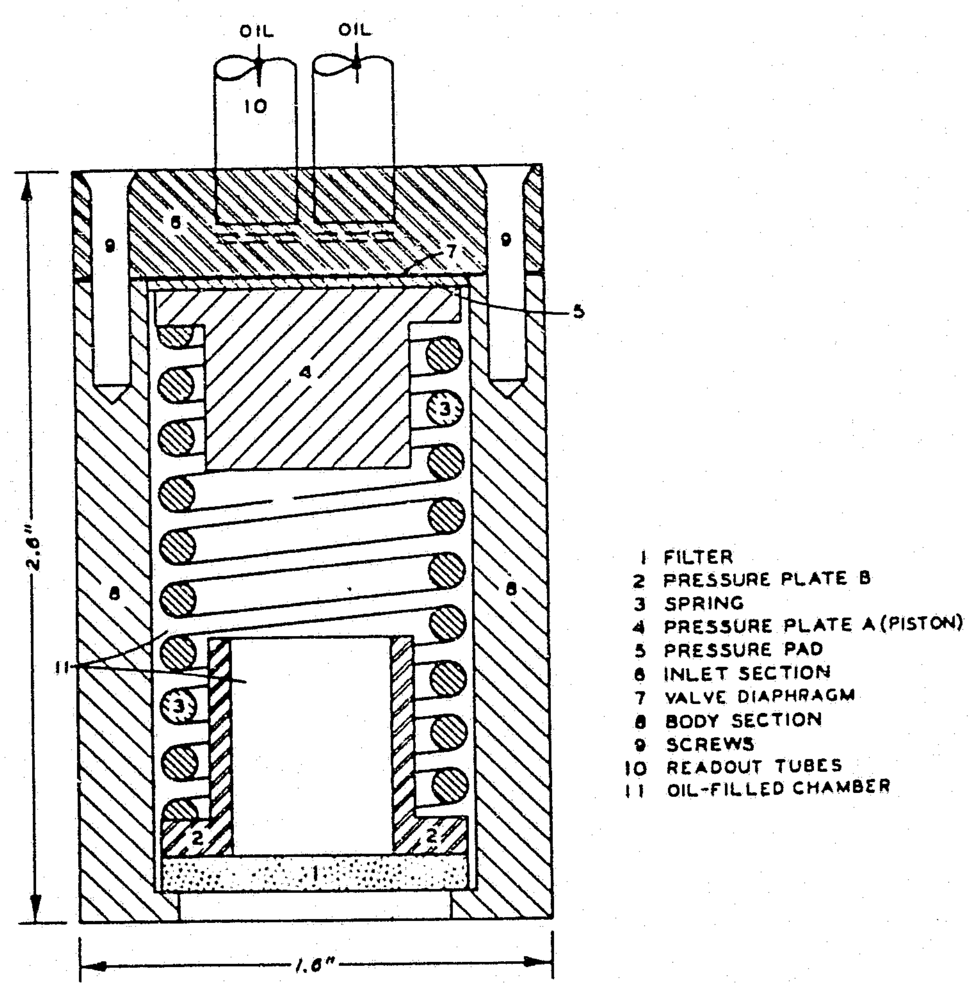


Schematic diagrams of pneumatic diaphragm piezometers

EXHIBIT C9



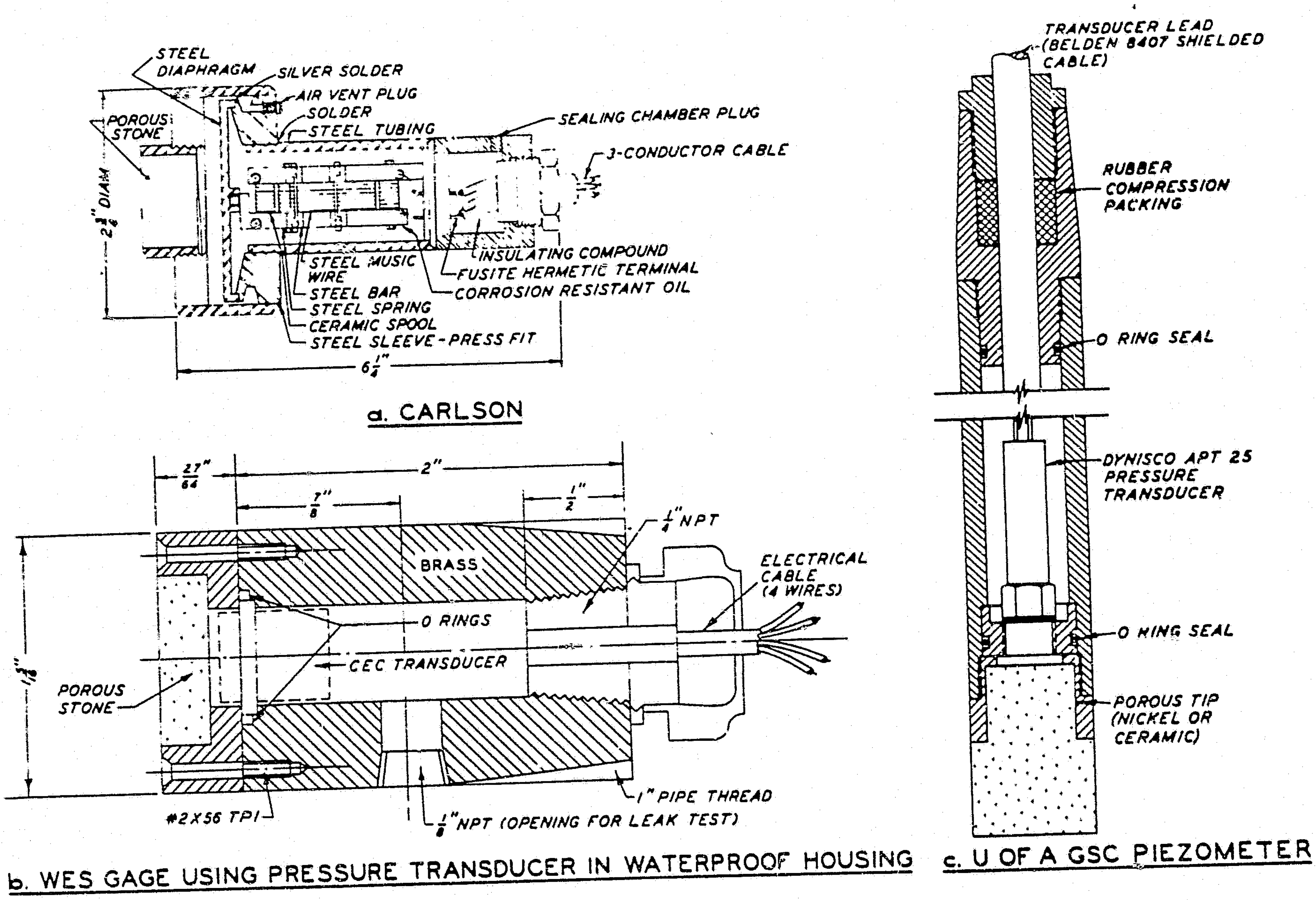
a. GLOETZL PIEZOMETER



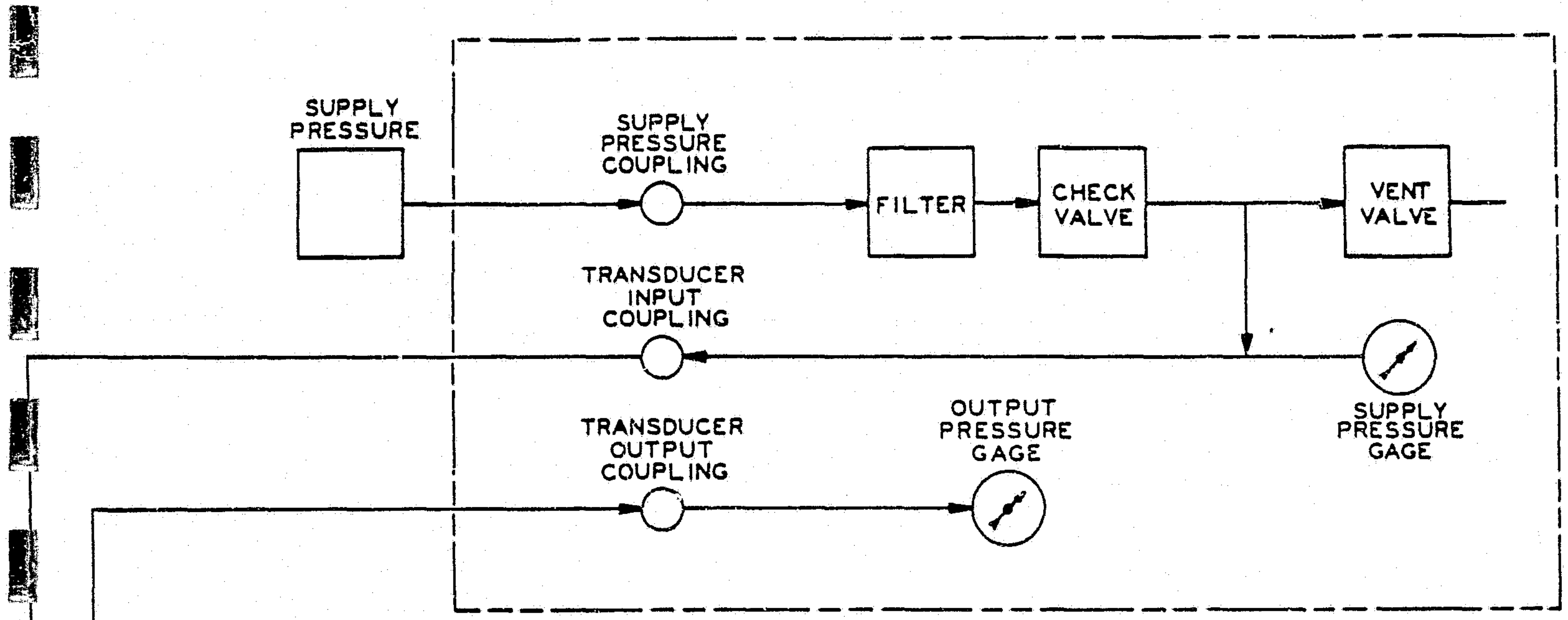
b. TERRAMETRICS HYDROSTATIC PORE PRESSURE CELL

Schematic diagrams of hydraulic diaphragm piezometers

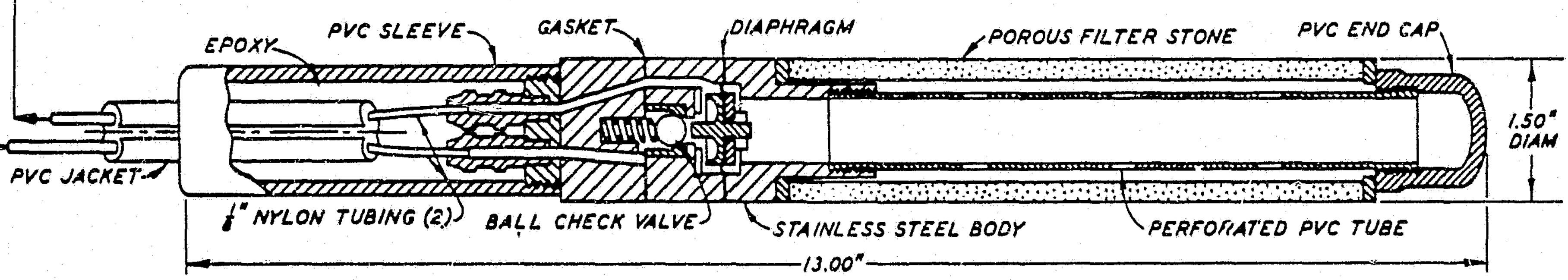
EXHIBIT C10



Schematic diagrams of electric strain gage diaphragm piezometers



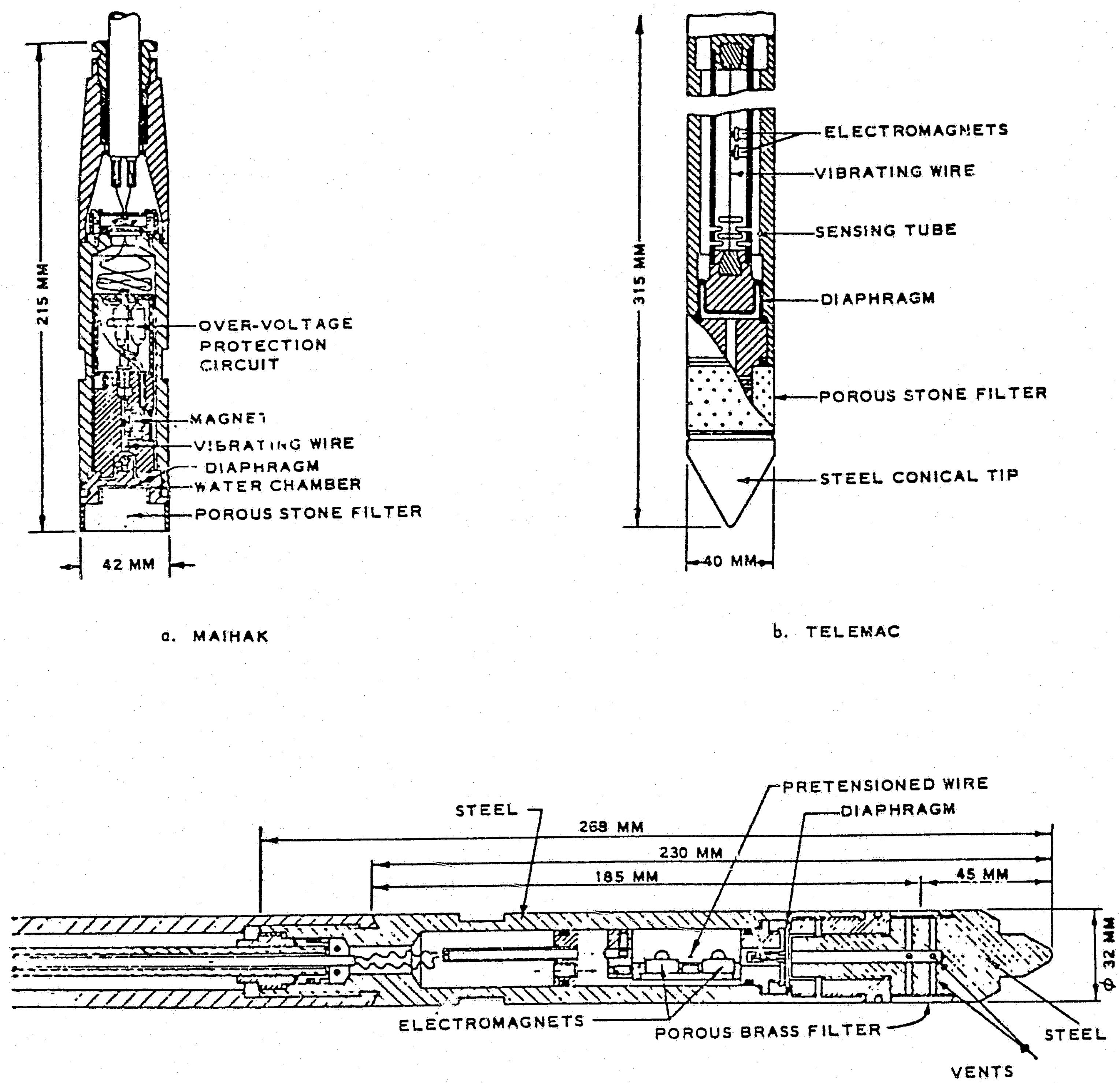
PORTABLE PORE PRESSURE INDICATOR



PORE PRESSURE TRANSDUCER

Slope Indicator pore pressure transducer

EXHIBIT C1 2



c. GEONOR

Schematic diagrams of electroacoustical diaphragm piezometers

APPENDIX D

EARTH MANUAL U.S. BUREAU OF RECLAMATION

VISUAL AND LABORATORY METHODS FOR IDENTIFICATION AND CLASSIFICATION OF SOILS

TABLE OF CONTENTS

PART A - VISUAL METHOD

LIST OF EXHIBITS

EARTH MANUAL

A WATER
RESOURCES
TECHNICAL
PUBLICATION

SECOND EDITION

U.S. DEPARTMENT OF THE INTERIOR
WATER AND POWER RESOURCES SERVICE

(2) Listing and identification of the samples giving type of sample, field sample number, hole number, elevation or depth.

(3) Purposes for which the samples were obtained.

(4) Log of exploration.

VISUAL AND LABORATORY METHODS FOR IDENTIFICATION AND CLASSIFICATION OF SOILS

Designation E-3

1. Scope.—This designation describes the methods and procedures for identifying, classifying, and describing soils in accordance with the Unified Soil Classification System.[1] The system is not limited to a particular use or geographic location. It does not conflict with other systems; in fact, the use of geologic, pedologic, textural, or local terms is encouraged as a supplement to, but not as a substitute for, the definitions, terms, and phrases established by the system and which are easy to associate with actual soils.

In this system 15 basic soil groups have been selected to define certain distinctive and peculiar engineering properties. Depending upon its basic properties, a soil is catalogued according to these groups and assigned a name and symbol; thus a soil is classified. These groups are broad; therefore, supplemental detailed word descriptions are required to point out peculiarities of a particular soil and differentiate it from others in the same group.

This system does not provide quantitative data for design purposes. It does provide qualitative information. Logs of exploration holes containing adequate soil classifications and descriptions may be used (1) in making preliminary estimates, (2) in determining the extent of additional field investigations needed for detailed design, (3) in planning an economical field testing or sampling program for laboratory testing, and (4) in extending the results of tests to additional explorations. In connection with the above, use charts have been developed to indicate the general engineering properties and potential value of the various soils

[1] This system based on the AC system by A. Casagrande was adopted jointly in 1952 by the Corps of Engineers and the Bureau of Reclamation. The procedure given here is adapted from a supplement to the Earth Manual published by the Bureau of Reclamation, Denver, Colo., 1953, and is similar to Technical Memorandum No. 3-357, and appendices A and B, prepared for the Office, Chief of Engineers, by Waterways Experiment Station, Vicksburg, Miss., in 1953.

for engineering uses.[2] For final detailed designs of important structures, the classification must be supplemented by laboratory tests or other quantitative data to determine the performance characteristics of the soil, such as permeability, shear strength, and compressibility under expected field conditions.

2. General Procedure.—Three steps are required to classify a soil.

(a) *First Step.*—The basic properties and characteristics of the soil components which influence the behavior of the soil as a foundation or construction material are identified. These include the sizes of particles, the amounts of the various sizes, and the influence of moisture on the characteristics of the very fine grains. Two methods are provided:

(1) The visual or field method, so called because manual (hand) tests and visual observations are employed in lieu of precise laboratory tests to define the basic soil properties. A knowledge of soil behavior and particularly an understanding of and experience in performing the gradation and soil consistency tests, upon which the hand tests and observations are based, are desirable prerequisites for competent visual classification. The visual method is used primarily in the field to classify and describe soils for logging exploration holes. This method is described in detail in Part A, Visual Method.

(2) The laboratory method, as the name implies, requires laboratory tests, specifically gradation and moisture limits, to define the basic soil properties. This method is used only when precise delineation is required, when unusual soils or conditions are encountered, or if the tests are required to supplement other laboratory tests required for design of major structures. It is also useful as an aid in teaching the visual classification method. This method is described in detail in Part B, Laboratory Method.

(b) *Second Step.*—The soil is placed into a classification group denoted by a group symbol, assigned in accordance with the criteria established by the system for the visual or laboratory method of classification.

(c) *Third Step.*—A written description of the soil is made. Regardless of the method used to identify the basic properties and characteristics, descriptive information is necessary to differentiate between soils in the same group (see pars. 8 and 13 for coarse- and fine-grained soils, respectively). The descriptive information required also depends on the purpose for which a soil is being investigated. For construction materials, as borrow for embankment, base course, backfill, or other uses, paragraph 8 or 13 applies; and for foundations for structures, the require-

[2] The basic principles of the system and use of the classification information are discussed in chapter I.

ments are given in paragraph 14. Examples of field classification and description are given on the classification chart, figure 3-1, and on the data forms, figure 3-2.

Part A. Visual Method

3. Apparatus.—Special apparatus or equipment is not required. However, the following items will facilitate the work:

(1) A rubber syringe or a small oil can having a capacity of approximately ½ pint.
(2) A supply of clean water.
(3) Small bottle of dilute hydrochloric acid.
(4) Classification chart, figure 3-1.

4. Procedure.—The classification of a soil by this method is based on visual observations and estimates of its behavior in a remolded state. The procedure is, in effect, a process of elimination, beginning on the left side of the classification chart, figure 3-1 (see column headed Field Identification Procedures), and working to the right until the proper symbol is obtained. The group symbol must be supplemented by detailed word descriptions, including a description of the inplace conditions for soils to be used in situ as foundations.

By recording, briefly, the observations made in the step by step procedure given below, the information for classifying and describing the soil is obtained. The forms shown on figure 3-2 are recommended for use in the laboratory for training purposes as an aid to attaining proficiency in classification and logging procedures. However, final field classification of soils should be recorded on the form shown on figure 3-3. The classification chart and a check list of descriptive items in paragraphs 8, 13, and 14 are helpful in classifying soils in the field.

(*Note:* Many natural soils will have properties not clearly associated with any one soil group, but which are common to two or more groups. Or they may be near the borderline between two groups, either in percentages of the various sizes or in plasticity characteristics. For this substantial number of soils, borderline classifications are used; that is, an appropriate dual symbol is assigned. A dual symbol consists of the two group symbols most nearly indicating the proper soil description, connected by a hyphen as, for example GW-GC, SC-CL, ML-CL, and others.)

5. Selection and Preparation of Sample.—Select a representative sample of the soil and spread it on a flat surface or in the palm of the hand.

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately $\frac{1}{64}$ in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

DILATANCY (Reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its 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.

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 between the fingers. This strength is a measure of the character and quantity 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. 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.

TOUGHNESS (Consistency near plastic limit)

After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. 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, the pieces should be lumped together and a slight kneading action continued 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-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

(a) Estimate and record the maximum particle size in the sample.

(b) Remove all particles larger than 3 inches from the sample. Estimate the percentage and distribution, by weight (volume is satisfactory), of cobbles (particles 3 to 12 inches in diameter) and boulders (particles over 12 inches in diameter) removed, and record as descriptive information (fig. 3-2), or in the proper columns on figure 3-3. *Only that fraction of the sample smaller than 3 inches is classified.*

6. Division Between Coarse- and Fine-Grained Soils.—Classify the sample as coarse-grained or fine-grained by estimating the percent, by weight, of particles which can be individually seen by the unaided eye. Soils containing more than 50 percent individually visible particles are coarse-grained soils. Soils containing less than 50 percent individually visible particles are fine-grained soils (see fig. 3-1).

(*Note:* For classification purposes, the No. 200 sieve size (0.074 mm.) is the particle-size division between fine- and coarse-grained particles. Particles of this size are about the smallest that can be seen individually by the unaided eye.)

7. Visual Procedure for Coarse-Grained Soils.—If it has been determined that the soil is coarse-grained, the soil is further identified by estimating and recording the percentage of: (1) gravel-sized particles, size range 3 inches to the No. 4 sieve (about ¼ inch); (2) sand-sized particles, size range No. 4 sieve to No. 200 sieve; and (3) silt- and clay-sized particles, size range smaller than No. 200 sieve.

(*Note:* The fraction of a soil smaller than the No. 200 sieve size, the clay and silt fraction, is referred to as "fines.")

(a) *Gravelly Soils.*—If the percentage of gravel is greater than the sand, the soil is a GRAVEL designated by the capital letter G.

Gravel-sized particles are further divided as follows:

Coarse gravel—3 inches to ¾ inch
Fine gravel—¾ inch to No. 4 (about ¼ inch)

These divisions are used to describe the average size of the gravel if poorly graded.

Gravels are further identified as being CLEAN (when containing less than 5 percent fines) or DIRTY (when containing more than 12 percent fines). However, the term "dirty" is usually not used in a description; instead, the properties of the fines that make the gravel "dirty" have to be described. Gravel containing 5 to 12 percent fines are given borderline classifications, that is, dual symbols. If the soil is obviously clean, the classification will be either:

(1) GW, well-graded, if there is good representation of all particle sizes, or

7-1686 (8-71)
Bureau of Reclamation

VISUAL CLASSIFICATION OF SOIL SAMPLES

PROJECT Example FEATURE ______ TABLE NO. ____ SHEET ____ OF ____

IDENTIFICATION				GRADATION (ESTIMATED)				COLOR (WET STATE)	DESCRIPTION AND SOIL CLASSIFICATION	GROUP SYMBOL
SAMPLE NUMBER 16Y-	HOLE NUMBER	LOCATION OR STATION	DEPTH - feet (Meters)	MAXIMUM SIZE	GRAVEL (% PLUS #4)	SAND (% #4 TO #200)	FINES (% MINUS #200)		1. DESCRIPTIVE CLASSIFICATION 2. PARTICLE SIZE, SHAPE, AND GRADATION (UNIFORMLY, WELL, POORLY GRADED, ETC.) 3. CONSISTENCY, ELASTICITY, ETC. 4. REACTION TO SHAKING TEST, DRY STRENGTH, ETC.	
1	3	Borrow area A	0 - 3.0	3"	70	30	0	Gray	Well-graded GRAVEL; clean, hard, subangular gravel sizes, considerable coarse subrounded sand sizes.	GW
2	3		3.0 - 6.0	1"	60	10	30	Tan	Clayey GRAVEL; predominantly fine, hard, subrounded gravel sizes, small amount of fine sand, clay portion slightly plastic, moderate reaction to HCL.	GC
3	3		6.0 - 12.0	6"	60	30	10	Brown	Well-graded GRAVEL; fairly clean, hard, angular gravel sizes, considerable sand, clay portion moderately plastic, (approximately 15 percent oversize 3" to 6", estimates made in field), moderate reaction to HCL.	GW-GC
4	5		1.5 - 3.0	8"	--	95	5	Tan	Poorly-graded SAND; hard, subangular, no medium sand sizes very few fines (approximately 10 percent oversize 3" to 8", estimates made in field), moderate reaction to HCL.	SP
5	5		3.0 - 10.0	¼"	5	70	25	Brown	Silty SAND; predominantly coarse, subangular sand sizes, contains a few angular gravel particles and considerable nonplastic fines.	SM
6	5		10.0 - 15.0	#30	--	50	50	Tan	Silty SAND; fine to medium, poorly-graded, hard, micaceous slightly plastic fines.	SM-ML
7	9		0 - 2.0	#50	--	15	85	Brown	Inorganic SILT; slight plasticity, contains some fine sand no dry strength.	ML
8	10	Borrow area B	0 - 8.0	#100	--	5	95	Gray	Inorganic CLAY; high plasticity, high dry strength, contains a trace of fine sand.	CH

TABLE ____ SHEET ____ OF ____

NOTE: Numbers in parenthesis are metric equivalents of numbers directly above.

GPO 835-571

Figure 3-2.—Data form for visual classification of borrow area samples. (Sheet 1 of 2.) 101-D-526.

7-1660 (8-71)
Bureau of Reclamation

VISUAL CLASSIFICATION OF FOUNDATION SAMPLES

PROJECT Example FEATURE ____ TABLE NO. ____ SHEET ____ OF ____

IDENTIFICATION				GRADATION (ESTIMATED)					DESCRIPTION - UNDISTURBED STATE	SOIL CLASSIFICATION	
SAMPLE NUMBER	HOLE NUMBER	LOCATION OR STATION	DEPTH - feet (Meters)	MAXIMUM SIZE	GRAVEL (% PLUS #4)	SAND (% #4 TO #200)	FINES (% MINUS #200)	COLOR (WET STATE)	1. CONSISTENCY: VERY SOFT, SOFT, FIRM, HARD, VERY HARD, STICKY, BRITTLE, FRIABLE, SPONGY 2. STRUCTURE - STRATIFIED, VARVED, SINGLE GRAIN, ETC. 3. CEMENTATION AND MOISTURE 4. GENERAL GEOLOGIC DESCRIPTION	1. DESCRIPTIVE CLASSIFICATION 2. PARTICLE SIZE, SHAPE, GRADATION (UNIFORMLY, WELL, POORLY GRADED, ETC.) 3. CONSISTENCY, ELASTICITY, ETC.	GROUP SYMBOL
7N-									Denison Sample		
1	DH-1	Sta. 18+00	12.0-13.1	#200	0	5	95	Dark gray	SHALE; moist, hard, laminated has a gray 1" bentonite seam (soapy feel), no reaction to HCl, impervious. (Pierre formation).	Fat CLAY, high plasticity.	CH
									12-inch Cube Samples		
2	TP-3	Borrow area D	13.2-14.2	#100	0	20	80	Brown	LOESS; soft, moist, contains numerous roots and root holes, moderate reaction to HCl.	Sandy SILT; slight plasticity.	ML
3	TP-4	Sta. 3+25 30' rt. of centerline	16.3-17.3	#100	0	10	90	Tan	CLAY; homogeneous, sample appears to be disturbed, firm, fairly moist, reacts violently to HCl, impervious. (Niobrara formation).	Lean CLAY; chalky	CL
4	TP-8	Sta. 16+00	44.3-45.3	#50	0	30	70	Blue and tan	SAND; top 6" soft, very moist, bluish grey with minor fat clay stringers, porous structure, pervious, bottom 6" firm, moist, tan.	Poorly-graded SAND; predominantly fine, slightly clayey.	SP
									3-inch Core Samples		
5	DH-5	Sta. 8+00 centerline	46.1-47.0	2"	30	60	10	Brown	CEMENTED SAND; hard, dense, stratified, calcareous, appears pervious. (Ogallala formation).	Well-graded SAND; rounded particles, gravelly.	SW
*6	DH-9	Sta. 4+25 Spillway centerline	5.6-[illegible].0	--	--	--	--	Brown	VERY HARD SILTSTONE; contains calcareous lenses, impervious. (Montezuma formation).	*Bedrock is not classified as a soil or given a soil group symbol.	

NOTE: Numbers in parenthesis are metric equivalents of numbers directly above.

TABLE ____ SHEET ____ OF ____

Figure 3-2.—Data form for visual classification of foundation samples. (Sheet 2 of 2.) 101-D-527.

LOG OF TEST PIT OR AUGER HOLE
FOR BORROW AND FOUNDATION INVESTIGATIONS

Feature: Example | Project: | Area Designation: Check Structure #2
Hole No.: TP-109 | Coordinates: Station 1212+25 | Ground Elevation: 790.5 | Approx. Dimensions: 4 x 5 feet
Depth to Water Level: *Not reached | Method of Excavation: Hand-dug pit | Date: 6-1 to 6-8, 19__ | Logged by:

Classification symbol: Letter	Classification symbol: Graphic	Depth (feet)	Size and type of sample taken	Classification and description of material (See chart "Unified Soil Classification"; give geologic and in-place description for foundation investigations)	Percentage of cobbles and boulders **: Volume of test sample (cubic feet)	Weight of 3 to 5 inch sampled	Percentage by volume of 3 to 5 inch	Weight of plus 5-inch sampled	Percentage by volume of plus 5-inch
GW	Not required for materials explorations		150-lb. sack, -3 inch	0'-7' Well-graded GRAVEL WITH COBBLES AND BOULDERS. Clean, approx. 70% hard, subrounded gravel, coarse to fine; approx. 30% coarse and medium sand; gray. Approx. 8% cobbles and 14% boulders (by volume) to 30-inch maximum size. Inplace condition - Loose, dry, nonstratified, slightly cemented, alluvial fan material.	6.2	82	8.4	137	14.1
CH			150-lb. sack sample	7'-14' Fat CLAY. Approx. 90% high plasticity fines, high dry strength, high toughness; approx. 10% medium sand; brown; no reaction with HCL; maximum size, medium sand. Inplace condition - soft, wet, homogeneous.		0		0	
SC-CL			150-lb. sack sample	14'-22' Clayey SAND. Approx. 50% hard, angular, coarse to fine sand, slightly micaceous; trace of gravel, maximum size, 1/2-inch; approx. 50% medium plasticity fines; yellow, moderate reaction with HCL. Inplace condition - Firm, moist, homogeneous, nonstratified.		0		0	
Sand-stone				22' SANDSTONE. Hard, highly cemented.					

REMARKS: Average specific gravity of cobbles and boulders - 2.51 by displacement.

Samples obtained from sampling trench.

NOTES: Record water test and density test data, if applicable, under remarks.
* Record after water has reached its natural level; give date of reading adjacent to graphic symbol or in remarks.
** Applicable only to borrow pits and to foundations which are potential sources of construction materials.
*** (lbs of rock sampled) (Bulk spec fic gravity of rock) x (Cubic feet of hole sampled). Record bulk specific gravity in Remarks, stating how obtained (measured or estimated).

Figure 3-3.—Data form for visual classification to be used in the field. 101-D-528.

(2) GP, poorly-graded, if there is either predominant excess or absence of particle sizes within the gravel range. The letters W and P can be used in classification symbols for the coarse-grained soils only when the percentage of fines is less than 12 percent.

If the soil obviously is dirty, the classification will be either:

(3) GM if the fines have little or no plasticity (silty), or

(4) GC if the fines are of low to medium or high plasticity (clayey). (See paragraphs 9 and 19 for procedure for classifying the "fines".)

(b) *Sandy Soils.*—If the percentage of sand is greater than gravel, the soil is a SAND designated by the capital letter S.

Sand-sized particles are further divided as follows:

Coarse sand—No. 4 (about ¼ inch) to No. 10 (about 3/32 inch)

Medium sand—No. 10 (about 3/32 inch) to No. 40 (about 1/64 inch)

Fine sand—No. 40 (about 1/64 inch) to No. 200 sieve (about 3/1,000 inch)

These divisions are used to describe the average size of the sand if poorly-graded.

The same procedure is applied as for gravels, except that the word SAND replaces GRAVEL and the symbol S replaces G. Thus, the clean sands will be classified as either:

(1) SW or

(2) SP

and the dirty sands will be classified as:

(3) SM if the fines have little or no plasticity (silty), or

(4) SC if the fines are of low to medium or high plasticity (clayey).

(c) *Borderline Classifications for Coarse-Grained Soils.*—Borderline classifications can occur within the coarse-grained soil division, between soils within either the gravel grouping or the sand grouping, and between gravelly and sandy soils.

The procedure is to assume the coarser soil, when there is a choice, and complete the classification and assign the appropriate group symbol; then, beginning where the choice was made, assume the finer soil and complete the classification, assigning the second group symbol.

Borderline classifications within the separate gravel or sand groups can occur; symbols such as GW–GP, GM–GC, GW–GM, SW–SP, SM–SC, and SW–SM are common.

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Borderline classifications can occur between the gravel and sand groups; symbols such as GW-SW, GP-SP, GM-SM, and GC-SC are common.

In addition to the borderline classifications within the coarse-grained division, borderline classifications also occur within the fine-grained division (par. 11 (c)).

Borderline classifications can also occur between coarse- and fine-grained soils; classifications such as SM-ML and SC-CL are common.

8. Descriptive Information for Coarse-Grained Soils.—The following information is required for a complete description of coarse-grained soils and should be recorded in the appropriate columns on figures 3-2 or 3-3. All of these descriptive data are not always needed. Judgment should be used to include pertinent information, to avoid negative information, and to eliminate repetition. However, items (1), (2), (3), (8), and (11) should always be included.

(1) Typical name.

(2) Maximum size, distribution, and approximate percentage of cobbles and boulders (particles larger than 3 inches) *in the total material.*

(3) Approximate percentage of gravel, sand, and fines in the *fraction of soil smaller than 3 inches.*

(4) For poorly-graded materials, statement of whether sand or gravel is coarse, medium, fine, or skip-graded.

(5) Shape of the grains; rounded, subrounded, angular, subangular.

(6) The surface coating, cementation, and hardness of the grains and possible breakdown when compacted.

(7) The color and organic content.

(8) Moisture conditions; dry, moist, wet, very wet (near saturation).

(9) Plasticity of fines; none, slight, medium, high plasticity.

(10) Local or geologic name.

(11) Group symbol.

9. Visual Procedure for Fine-Grained Soils.—If it has been determined that the soil is fine-grained, the soil is further identified by estimating the percentages of gravel, sand, and fines (silt- and clay-sized particles), and performing the manual identification tests for dry strength, dilatancy, and toughness. (See field identification procedures for fine-grained soils or fractions on fig. 3-1.) By comparing the results of these tests with the requirements given for the six fine-grained soil groups, the appropriate group name and symbol is assigned. The same procedures are used to identify the fine-grained fraction of coarse-grained soils to determine whether they are silty or clayey.

10. Manual Identification Tests.—The tests for identifying fine-grained soils are performed on that fraction of the soil finer than the No. 40 sieve size (about 1/64 inch).

The manual tests are considered to be performed on the "fines." The soil finer than the No. 40 includes the "fines" (minus No. 200) and fine sand (minus No. 40 to No. 200).

Select a small representative sample and remove by hand all particles larger than the No. 40 size and prepare two small specimens, each with a volume of about ½ cubic inch, by moistening until the specimens can easily be rolled into a ball. Perform the tests listed below, carefully noting the behavior of the soil pat during each test.

(*Note:* Operators with considerable experience find that it is not necessary in all cases to prepare two pats. For example, if the soil contains dry lumps, the dry strength can be readily determined without preparing a pat for this particular purpose.)

(a) *Dilatancy (Reaction to Shaking).*—Add enough water to nearly saturate one of the soil pats. Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. Squeeze the pat between the fingers. The appearance and disappearance of the water with shaking and squeezing is referred to as a reaction (fig. 3–4). This reaction is called (1) quick, if water appears and disappears rapidly, (2) slow, if water appears and disappears slowly, and (3) no reaction, if the water condition does not appear to change. Observe and record the type of reaction as descriptive information.

(b) *Toughness (Consistency Near Plastic Limit).*—Dry the pat used in the dilatancy test, subparagraph (a) above, by working and molding until it has the consistency of putty. The time required to dry the pat is an indication of its plasticity. Roll the pat on a smooth surface or between the palms into a thread about ⅛ inch in diameter. Fold and reroll the thread repeatedly to ⅛-inch diameter so that its water content is gradually reduced until the ⅛-inch thread just crumbles. The water content at crumbling stage is called the plastic limit, and the resistance to molding at the plastic limit is called the toughness.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles. If the lump can still be molded slightly drier than the plastic limit and if high pressure is required to roll the thread between the palms of the hands, the soil is described as possessing high toughness. Medium toughness is indicated by a medium tough thread and a lump formed of the threads slightly below the plastic limit will crumble; while slight toughness is indicated by a weak thread that breaks easily and cannot be lumped together when drier than the plastic limit. This test also provides approximate information on the plasticity index, *PI* (designation E-7), of the

REACTION TO SHAKING

REACTION TO SQUEEZING

Figure 3-4.—Reactions of a silty soil to shaking and squeezing (dilatancy test). PX-D-16335.

soil. The number of times the procedure can be repeated is an indication of the *PI* of the material.

Highly organic clays have a very weak and spongy feel at the plastic limit. Nonplastic soils cannot be rolled into a thread of ⅛-inch diameter at any water content. Observe and record the toughness as descriptive information.

(c) *Dry Strength (Crushing Resistance).*—Completely dry one of the prepared specimens. Then measure its resistance to crumbling and powdering between the fingers. This resistance, called dry strength, is a measure of the plasticity of the soil and is influenced largely by the colloidal fraction contained. The dry strength is designated as slight if the dried pat can be easily powdered, medium if considerable finger pressure is required, and high if it cannot be powdered at all. Observe and record the dry strength as descriptive information.

(*Note:* The presence of high-strength, water-soluble cementing materials, such as calcium carbonates or iron oxides, may cause high dry strengths. Nonplastic soils, such as caliche, coral, crushed limestone, or soils containing carbonaceous agents, may have high dry strengths, but this can be detected by the effervescence caused by the application of dilute hydrochloric acid (see acid test, subpar. (e) below).)

(d) *Organic Content and Color.*—Fresh, wet, organic soils usually have a distinctive odor of decomposed organic matter. This odor can be made more noticeable by heating the wet sample. Another indication of the organic material is the distinctive dark color. Dry, inorganic clays develop an earthy odor upon moistening, which is distinctive from that of decomposed organic matter.

(e) *Other Identification Tests.*—

(1) *Acid test.*—The acid test using dilute hydrochloric acid (HCl) is primarily a test for the presence of calcium carbonate. For soils with high dry strength, a strong reaction indicates that the strength may be due to calcium carbonate as cementing agent, rather than colloidal clay. The results of this test (no reaction to HCL should be reported) should be included in the soil description. (*Note:* Dilute solution (1:3) of hydrochloric acid is one part of concentrated hydrochloric acid to three parts of distilled water. Handle with caution. Rinse with tap water if it comes in contact with skin.)

(2) *Shine.*—This is a quick supplementary procedure for determining the presence of clay. The test is performed by cutting a lump of dry or slightly moist soil with a knife. A shiny surface imparted to the soil indicates highly plastic clay, while a dull surface indicates silt or clay of slight plasticity.

(3) *Miscellaneous.*—Other criteria undoubtedly can be developed by the individual as he gains experience in classifying soils. For

example, differentiation between some of the fine-grained soils depends largely upon the experience in the "feel" of the soils. Frequent checking by laboratory tests is necessary to gain this experience.

11. Silty and Clayey Soils.—Various combinations of results of the manual identification tests indicate which grouping is proper for the soil in question.

(a) The following three groups are soils possessing *slight to medium* plasticity (symbol L):

(1) ML has little or no plasticity and may be recognized by slight dry strength, quick dilatancy, and slight toughness.

(2) CL has slight to medium plasticity and may be recognized by medium to high dry strength, very slow dilatancy, and medium toughness.

(3) OL is less plastic than the clay (CL) and may be recognized by slight to medium dry strength, medium to slow dilatancy, and slight toughness. Organic matter must be present in sufficient amount to influence the soil properties in order for a soil to be placed in this group.

(b) The following three groups are soils possessing *slight plasticity to high plasticity* (symbol H):

(1) MH is generally very absorptive. It has slight to medium plasticity and may be recognized by low dry strength, slow dilatancy, and slight to medium toughness. Some inorganic soils (such as kaolin which is a clay from a mineralogical standpoint) possessing medium dry strength and toughness will fall in this group.

(2) CH possesses high plasticity and may be recognized by high dry strength, no dilatancy, and usually high toughness.

(3) OH is less plastic than the fat clay (CH) and may be recognized by medium to high dry strength, slow dilatancy, and slight to medium toughness. Organic matter must be present in sufficient amount to influence soil properties in order for a soil to be placed in this group.

(c) *Borderline Classifications for Fine-Grained Soils.*—Borderline classifications can occur within the fine-grained soil division, between low and high liquid limit soils, and between silty and clayey soils. The procedure is comparable to that given for coarse-grained soils in paragraph 7(c); that is, first assume a coarse soil, when there is a choice, and then a finer soil and assign dual group symbols.

Borderline classifications which are common are as follows: ML–MH, CL–CH, OL–OH, CL–ML, ML–OL, CL–OL, MH–CH, MH–OH, and CH–OH.

12. Peat or Very Highly Organic Soils (Symbol Pt).—These may be readily identified by color, odor, sponginess, or fibrous texture.

13. Descriptive Information for Fine-Grained Soils.—The following information is required for a complete description of fine-grained soils and should be recorded in the appropriate columns of the log forms shown on figures 3–2 or 3–3. All of these descriptive data are not always needed. Judgment should be used to include pertinent information, to avoid negative information, and to eliminate repetition. However, items (1), (2), (6), (7), and (9) should always be included.

(1) Typical name.

(2) Maximum particle size. Distribution, and approximate percentage of cobbles and boulders (particles larger than 3 inches) *in the total material.*

(3) Approximate percentage of gravel, sand, and fines in the *fraction of soil smaller than 3 inches.*

(4) Hardness of the coarse grains, possible breakdown into smaller sizes.

(5) Color in moist condition and organic content.

(6) Moisture and conditions; dry, moist, wet, very wet (near saturation).

(7) Plasticity characteristics; none, slight, medium, high plasticity.

(8) Local or geologic name.

(9) Group symbol.

14. Descriptive Information for Foundation Soils.—The inplace condition of soils which are to be utilized as foundations for hydraulic or other structures assumes primary importance in soil classification. Logs of foundation explorations and descriptions of *undisturbed* samples, therefore, must emphasize the inplace conditions of the soil. It is necessary to present a complete word picture describing the soil as it exists in the foundation, in addition to assigning a name and proper group symbol.

Judgment should be used to include all pertinent information, to avoid negative information, and to eliminate repetition.

(a) *Coarse-Grained Soils.*—Items in table 3–1 should always be included when applicable. The information requested for each item can be recorded on the preprinted log in the approximate sequence in table 3–1. The degree of compactness and structure usually cannot be ascertained when augering; an exposed test pit or trench wall is essential for describing natural subsoil conditions. An example of a field log is shown on figure 3–3.

(b) *Fine-Grained Soils.*—Items listed in table 3–2 should always be included when applicable. The information requested for each item can be recorded on the preprinted log in the approximate sequence shown in

Table 3-1.—Check list for description of coarse-grained soils

Items of descriptive data	Typical information desired for sand and gravel
Typical name	GRAVEL; SAND; Clayey GRAVEL; Silty SAND WITH COBBLES; (Add descriptive adjectives for minor constituents—example: approximately 15 percent slight plasticity fines, medium toughness.)
Gradation	Well-graded; poorly-graded (uniformly-graded or skip-graded); (Describe range of particle sizes, such as fine to medium sand or fine to coarse gravel, or the predominant size or sizes as coarse, medium, fine sand or gravel.)
Size distribution	Approximate percent of gravel, sand, and fines in the fraction finer than 3 inches.
Plasticity of fines	None; low; medium; high.
Maximum particle size	Note percent of boulders and cobbles (by volume) as well as maximum particle size.
Mineralogy	Rock hardness for gravel and sand. Note especially presence of mica flakes, shaly particles, organic matter, or friable particles.
Grain shape	Angular; subangular; subrounded; rounded.
Color	Use one basic color, if possible.
Odor	None; earthy; organic.
Moisture condition	Dry; moist; wet; saturated.
Degree of compactness	Loose; dense.
Structure	Stratified; lensed, nonstratified; heterogeneous.
Cementation	Weak; moderate; strong. Note reaction to HCl as: none; weak; moderate; or strong.
Local or geologic name	
Group symbol	GP, GW, SP, SW, GM, GC, SM, SC, or the appropriate dual symbol when applicable. Should be compatible with typical name used above.

table 3-2. The items of consistency, degree of compactness, and structure usually cannot be ascertained when augering; an exposed test pit or trench wall is essential for describing natural subsoil conditions. If hard rock such as siltstone is encountered, it should not be given a soil group symbol such as ML but should be designated as siltstone on the log An example of a field log is given on figure 3-3. The consistency of cohesive soils may be determined in place or on undisturbed samples in accordance with the identification procedure given in table 3-3.

The structural characteristics of intact soils provide important clues to their performance as foundation materials. Whenever undisturbed samples are available or when the soil profile may be inspected during sampling from a pit, the structural characteristics should be described. Stratified

Table 3-2.—Check list for description of fine-grained and partly-organic soils

Items of descriptive data	Typical information desired for silt and clay
Typical name	SILT: *Sandy SILT; CLAY; Lean or Fat CLAY; *Sandy CLAY; Silty CLAY; Organic SILT; Organic CLAY. * 25 percent or more sand must be present. "Gravelly" can be substituted for "Sandy" where applicable. Include cobbles and boulders in typical name when applicable.
Size distribution	Approximate percent of fines, sand, and gravel in fraction less than 3 inches in size. Must add to 100 percent.
Plasticity of fines	None; low; medium; high.
Dry strength	None; low; medium; high.
Dilatancy	None; very slow; slow; medium; quick.
Toughness near plastic limit	None; slight (low); medium; high.
Maximum particle size	Note percentage of cobbles and boulders (by volume) as well as maximum particle size.
Color	Use one basic color, if possible. Note presence of mottling or banding.
Odor	None; earthy; organic.
Moisture condition	Dry; moist; wet; saturated.
Consistency (see table 3-3) (for clay).	Very soft; soft; firm; hard; very hard.
Degree of compactness (for silt and fine sand).	Loose; dense.
Structure	Stratified; laminated (varved); fissured; slickensided; blocky; lensed; homogeneous. (The thickness, dip, and strike of layers should be included.)
Cementation	Weak; moderate; strong. Note reaction with HCl as: none; weak; moderate; or strong.
Local or geologic name	
Group symbol	CL, CH, ML, MH, OL, OH, Pt or the appropriate dual symbol when applicable. Should be compatible with typical name used above.

materials consist of alternating layers of varying types (or color). If the layers are less than about one-fourth inch thick, it may be described as laminated (or varved, if mostly fine-grained). Fissured materials break along definite planes of fracture with little resistance to fracturing. If the fracture planes appear polished or glossy, they should be described as slickensided. If a cohesive soil can be easily broken into small angular lumps which resist further breakdown, the structure may be described as

Table 3-3.—Identification of consistency of fine-grained soils

Consistency	Identification procedure
Very soft	Easily penetrate soil several inches using high thumb pressure or less.
Soft	Penetrate soil about 1 inch using high thumb pressure.
Firm	Soil indented less than ¼ inch using high thumb pressure.
Hard	Soil not indented using high thumb pressure. Readily indented by using thumbnail.
Very hard	Not indented with thumbnail.

blocky. A lensed structure is indicated by the inclusion of small pockets of different texture, such as small lenses of sand scattered through a mass of clay. The presence of special structural characteristics, such as root holes or porous openings, should also be noted. If no structural characteristics are apparent, the soil may be described as nonstratified or homogeneous.

Part B. Laboratory Method

15.—Apparatus.—Special apparatus is required as noted below:

(1) Equipment for performing the gradation test (see designation E-6).

(2) Equipment for performing the moisture limits tests (see designation E-7).

(3) A small bottle of dilute hydrochloric acid.

(4) Classification chart, figure 3-1.

16. Procedure.—The Unified Soil Classification System provides for precise delineation of the soil groups by using results of laboratory tests. For gradation and moisture limits, rather than visual estimates, see right-hand column of the classification chart, figure 3-1, entitled "Laboratory Classification Criteria." Classifying by these tests alone does not fulfill the requirements for complete classification as it does not provide an adequate description of the soil. Therefore, the descriptive information required for the visual method (pars. 8, 13, and 14) should also be included in the laboratory classification.

(a) *Preparation of Sample.*—Screen out the plus 3-inch fraction of the soil, noting the percentage.

(b) *Division between Fine- and Coarse-Grained Soils.*—Obtain the grain-size distribution of the minus 3-inch fraction by performing the laboratory gradation test. If the soil contains more than 50 percent by weight larger than the No. 200 sieve size, the soil is classified as coarse-grained; if less than 50 percent, it is classified as fine-grained.

17. Laboratory Procedure for Coarse-Grained Soils.—Coarse-grained soils are subdivided into GRAVEL and SAND (par. 7) by referring to the gradation curve instead of visually estimating the percentage of various sized particles present in the soil.

18. Gravelly or Sandy Soils.—Gravels or sands are further identified as being CLEAN or DIRTY by determining the amount of material finer than the No. 200 sieve. If less than 5 percent is finer than the No. 200 sieve, the soil will be classified as either:

(1) WELL-GRADED (GW or SW) if the coefficient of uniformity C_u is greater than 4 for gravels and 6 for sands, and the coefficient of curvature C_c is between 1 and 3; or

(2) POORLY-GRADED (GP or SP) if either one or both the C_u and C_c criteria for (1) above are not satisfied.

The coefficient of uniformity C_u and coefficient of curvature C_c are expressed as follows:

$$C_u = \frac{(D_{60})}{(D_{10})} \qquad C_c = \frac{(D_{30})^2}{(D_{10}) \times (D_{60})}$$

where:

D_{10}, D_{30}, and D_{60} are the grain-size diameters corresponding respectively to 10, 30, and 60 percent passing on the cumulative grain-size curve.

If more than 12 percent of the total soil is finer than the No. 200 sieve size, the soil will be classified as either:

(3) SILTY (GM or SM) if the results of the moisture limits tests show that the fines are silty—that is, the plot of liquid limit versus plasticity index falls below the "A" line (see plasticity chart, fig. 3-1)—or if the plasticity index is less than 4; or

(4) CLAYEY (GC or SC) if the fines are clayey—that is, the plot of liquid limit versus plasticity index falls above the "A" line and the plasticity index is greater than 7.

(a) *Borderline Classifications for Coarse-Grained Soils.*—Coarse-grained soils containing between 5 and 12 percent of fines are classified as borderline cases between the clean and the dirty gravels or sands as, for example, GW-GC or SP-SM. Similarly, borderline cases may occur in dirty gravels and dirty sands where the *PI* is between 4 and 7 as, for example, GM-GC or SM-SC. It is possible, therefore, to have a borderline case of a borderline case. The rule for correct classification in this case is to favor the nonplastic classification. For example, a gravel with 10 percent fines, a C_u of 20, a C_c of 2.0, and a *PI* of 6 would be classified GW–GM rather than GW–GC.

19. Laboratory Procedure for Fine-Grained Soils.—Soils containing more than 50 percent fines according to the grain-size curve are classified into one of the six fine-grained groups by the results of the moisture limits tests, as plotted on the plasticity chart, with attention being given to the organic content. Those with a liquid limit less than 50 are referred to as inorganic silts and clays of slight to medium plasticity, while those with a liquid limit greater than 50 are the elastic silts and fat clays of medium to high plasticity.

Organic silts and clays are usually distinguished from inorganic silts which have the same position on the plasticity chart by odor and color. However, when the organic content is doubtful, the material can be ovendried, remixed with water, and retested for liquid limit. The plasticity of fine-grained organic soils is greatly reduced on ovendrying owing to irreversible changes in the properties of the organic material. Ovendrying also affects the liquid limit of inorganic soils, but only to a small degree. A reduction in liquid limit after ovendrying to a value less than three-fourths of the liquid limit before ovendrying is positive identification of organic soils.

20. Subdivision of Fine-Grained Soils.—These soils are subdivided as follows:

(a) *Liquid Limit Less Than 50.*—

(1) ML has a liquid limit less than 50 and the plasticity index ranges from 0 to 22 (see area identified as ML or OL below the "A" line on the plasticity chart).

(2) CL has a liquid limit less than 50 and a plasticity index greater than 7 (see area identified as CL above the "A" line on the plasticity chart).

(3) OL contains sufficient organic material to affect the soil properties. The OL soils have liquid limits less than 50 and their plasticity indices range from 0 to 22 (see area identified as ML or OL below the "A" line on the plasticity chart).

(b) *Liquid Limit Greater Than 50.*—

(1) MH has a liquid limit greater than 50 and the plasticity index ranges from 0 to over 50 (see area identified as MH or OH below the "A" line on the plasticity chart).

(2) CH has a liquid limit greater than 50 and the plasticity index ranges from 22 to over 50 (see area identified as CH above the "A" line on the plasticity chart).

(3) OH has a liquid limit greater than 50 and the plasticity index ranges from 0 to over 50 (see area identified as OH or MH below the "A" line on the plasticity chart).

(c) *Borderline Classifications for Fine-Grained Soils.*—Fine-grained soils whose plot of liquid limit versus plasticity index falls on, or practically on, (1) "A" line or (2) the line LL=50 should be assigned the appropriate borderline classification. Soils which plot above the "A" line, or practically on it, and which have a plasticity index between 4 and 7 are classified ML–CL.

Part C. Graphic Symbols for Soils

21. Use of Symbols for Soils.—For pictorial presentation of a soil profile or log of exploration hole, graphic symbols for soils are sometimes advantageous and may be used. To avoid evaluation of a soil deposit on the basis of the graphic symbols, and for simplicity, graphic symbols are given only for the basic soil components: gravel, sand, silt, and clay (fig. 3–5). The graphic symbol for the noun in the group name assigned a soil should be used. For borderline soils the graphic symbols may be combined.

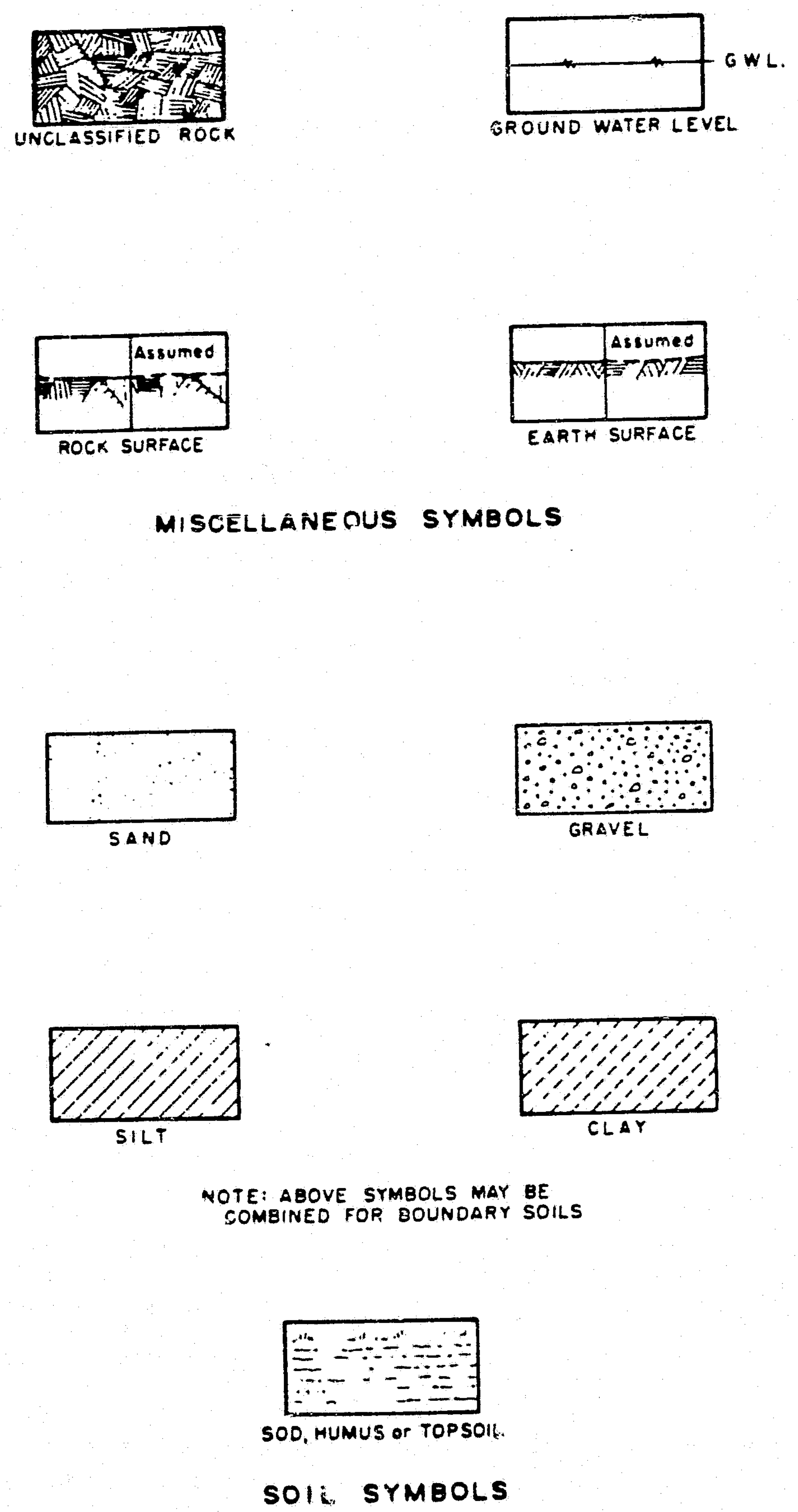


Figure 3-5.—Graphic symbols for soils. 103-D-345.

UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION

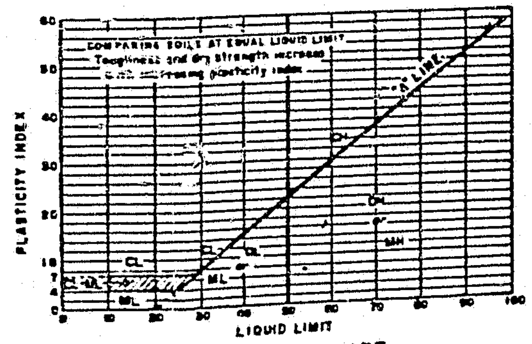
FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 3 inches and basing fractions on estimated weights)

Major divisions			Field identification procedures	Group symbols *	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 ** (The No. 200 sieve size is about the smallest particle visible to the naked eye)	GRAVELS — More than half of coarse fraction is larger than No 4 sieve size (For visual classifications, the 1/4" 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, max size, angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one 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.		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.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.	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		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one 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	EXAMPLE:- Silty sand, gravelly, about 20% hard, angular gravel particles 1/2-in maximum size, rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place, alluvial sand, (SM)	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 above "A" line with PI greater than 7	

Laboratory classification criteria, coarse grained soils: 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.

IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No 40 SIEVE SIZE

Major divisions		Dry strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group symbols *	Typical names	Information required for describing soils
FINE GRAINED SOILS — More than half of material is smaller than No 200 sieve size. (The No. 200 sieve size is about the smallest particle visible to the naked eye)	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, color in wet condition, odor 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	
		Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity.	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
	SILTS AND CLAYS — Liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays	EXAMPLE:- Clayey silt, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes, firm and dry in place, loess, (ML)
		Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity.	
HIGHLY ORGANIC SOILS		Readily identified by color, odor, spongy feel and frequently by fibrous texture			Pt	Peat and other highly organic soils	



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

ADOPTED BY - CORPS OF ENGINEERS AND BUREAU OF RECLAMATION - JANUARY 1952

* 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
** All sieve sizes on this chart are US standard

Figure 3-1.—Unified soil classification chart. From drawing 103-D-347.

APPENDIX E

GUIDE TO FIELD DESCRIPTION OF PERMAFROST

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1. WHAT IS MEANT BY "PERMAFROST"

Permafrost is defined as the thermal condition under which earth materials exist at a temperature below 32°F continuously for a number of years. Thus, all earth materials including bedrock, gravel, sand, silt, peat or mixtures of these materials may exist in the perennially below 32°F condition. Permafrost is defined exclusively on the basis of temperature, irrespective of texture, degree of induration, water content or lithologic character.

The term "perennially frozen", although cumbersome, is generally used to describe specific perennially frozen materials, e.g. perennially frozen silt, perennially frozen organic material. The presence of ice is not a necessary requisite of permafrost, but when ice is present it is of particular significance to engineers.

The term permafrost can also be used to describe the areal extent of the below 32°F condition. It has been found convenient to divide the permafrost region into two major zones—the continuous and the discontinuous. In the *continuous* zone permafrost is found everywhere under the ground surface to considerable depth; in the *discontinuous* zone permafrost is not as thick and exists in combination with areas of unfrozen material.

2. BASIS OF THE DESCRIPTIVE SYSTEM

Although permafrost is defined on a temperature basis, temperature is not a convenient or easily measured property for field description purposes. A more convenient approach to the field description of permafrost is to describe terrain features that may influence the existence of permafrost. Observations of terrain and the effects of construction on permafrost in northern Canada have suggested the following specific features of terrain that are of interest to engineers:

(1) Surface characteristics
- Vegetation cover
- Snow cover
- Relief and drainage

(2) Subsurface characteristics
- Depth of thaw
- Subsurface materials
  - Soil phase
  - Ice phase

Observations on these topics represent a minimum of field information that must be collected to describe permafrost adequately for an engineering appraisal of a site.

3. SURFACE CHARACTERISTICS

3.1 Vegetation Cover

The vegetative mantle of trees, shrubs, moss, lichen and other plants that covers much of the North acts as an insulator that protects and maintains permafrost. Vegetative cover is of additional interest in that it may indicate soil, ground water, wind and/or snow conditions. The major combinations of vegetation at a site should be delineated and described using the system outlined in the "Guide to a Field Description of Muskeg" (Technical Memorandum 44) published by the Associate Committee on Soil and Snow Mechanics.

3.2 Snow Cover

Although snow is basically a part of the climate, snow cover is generally considered as a terrain factor. The presence of snow reduces the depth of seasonal frost penetration during the winter and conversely inhibits thawing of frozen material in the spring. The type of snow, the depth of snow cover and their variability over a site throughout the winter season should therefore be observed.

3.3 Relief and Drainage

Terrain relief influences permafrost occurrence and since it is also a significant factor in drainage it is an important engineering consideration. Regional relief features should be described in addition to those observed at specific locations under investigation. Regional descriptions should include some indication of altitudes; whether the landscape is mountainous, hilly, undulating or flat; possible origin of the landform; and the regional drainage pattern. At specific sites small scale or micro features of relief and drainage should be noted. These small scale features are difficult to classify for descriptive purposes but would include details of patterned ground (sorted or unsorted circles, nets, polygons, steps and stripes), micro-drainage, slope and exposure to solar radiation.

A photographic record of the various surface characteristics (e.g. showing typical vegetation and snow cover, and relief and drainage features) is most valuable. A complete description of site conditions can be usefully summarized on a sketch map or air photograph of the area under investigation.

4. SUBSURFACE CHARACTERISTICS

4.1 Depth of Thaw

The seasonal depth of thaw and its variability within an area and from year to year has long been recognized as an important engineering consideration in permafrost areas. The depth of thaw refers to that portion below the ground surface at a specific location that is thawed at some time during the course of a summer. It increases progressively during the thawing season and therefore it is important to note the date on which a particular observation was made. When the seasonal thaw has reached its maximum depth (usually in the late fall) it then corresponds to the *"active layer"*. The *"active layer"* refers to the zone in which seasonal thawing *and* freezing occurs.

The depth and rate of thaw are affected by and closely related to terrain features. Any variations in an area are usually the result of differences in surface conditions such as vegetation, relief, drainage and snow cover and may also be related to changes in subsurface materials.

Initially, depth of thaw observations should be made in areas having different surface covers and then extended to locations within these areas that have noticeable changes in relief, drainage or subsurface materials. Appreciable differences in the depth of thaw for an area as small as 5 feet square are possible. It is important therefore to make many random observations at a site and to record not only the average but also the maximum and minimum depths of thaw for the area.

The depth of thaw can be conveniently measured using a probe that retains a sample of the thawed subsurface materials for examination. Records of the depth of thaw should also include notes on the date of observation, vegetation cover, relief, drainage, and a description of the subsurface materials in the various areas probed. Some assessment of the moisture content, density and ice segregation in the frozen soil underlying the thawed zone are of particular interest.

4.2 Subsurface Materials

The materials encountered in the frozen state vary, and can include bedrock, gravel, sand, silt, clay and organic material (peat). These frozen materials or combinations of them frequently contain considerable quantities of ice. Important engineering implications are involved when this condition occurs. It is important, therefore, to examine not only the soil but also the ice encountered in the soil. For engineering purposes, it is convenient to describe the soil and ice phases independently. At times a description of frozen bedrock may be required. It will be noted in the following paragraphs that the ice description system is based on the form of ice in frozen materials and is therefore applicable for either soils or bedrock.

4.2.1 Soil phase

The description of the soil phase applies to materials found in both the thawed and frozen states. Coarse- and fine-grained soils should be described according to the "Guide to a Field Description of Soils" (Technical Memorandum 37) published by the Associate Committee on Soil and Snow Mechanics. Partly organic soils, which are largely mineral types, are described as the predominant soil modified by the word "organic", e.g. organic silt. Soils that are mostly organic (peat), however, should be described according to the system outlined in "Guide to a Field Description of Muskeg" (Technical Memorandum 44).

4.2.2 Ice phase

The descriptive system for the ice phase is based on the *form* of ice found in frozen materials. It is not intended that this system be used to assess frozen materials according to properties or performance.

For descriptive purposes frozen materials are divided into three major groups in which the ice is:

not visible by eye,

visible by eye with individual ice layers *less* than 1 inch in thickness,

visible by eye with individual ice layers *greater* than 1 inch in thickness.

The major ice phase descriptive groups and their subdivisions are summarized in Table I. Letter symbols that suggest key descriptive terms of the ice forms for each subdivision have been included to help in the preparation of graphic logs or records. Written observations, however, are the fundamental feature of the descriptive system and the letter designations must be regarded only as a "short-hand" form. Guides for further descriptive details and illustrations of the basic types are included. It is not expected or intended that all of the detail shown in Table I should always be noted. In much engineering work only the most fundamental details need be recorded. Some definitions to clarify terms used in the ice phase descriptive system are given in Table II.

(A) Ice not visible

When ice is not discernible by eye its effectiveness as a cementing agent in bonding the mineral or organic portion is used as a further subdivision:

(a) ice that bonds or cements the subsurface materials into a *weak or friable mass*,

(b) ice that bonds the subsurface material into a *hard, solid mass*.

The presence of ice not generally discernible by eye may be revealed within the voids of the material by crystal reflections or by a sheen on fractured or trimmed surfaces. The impression to the unaided eye is that none of the frozen water occupies space in excess of the original voids in the soil. The opposite is true of soils where the ice segregation is visible by eye.

In some cases, particularly in materials well bonded by ice, a large portion of the material may actually be ice, even though it is not discernible by eye. When visual methods are inadequate, a simple field test to aid evaluation of the volume of excess ice can be made by placing a chunk of the material in a small jar, allowing it to thaw, and observing the quantity of water as a percentage of the total volume. If free water is noted it is termed "excess".

(B) Visible ice segregation less than 1 inch thick

When ice is discernible by eye and is less than 1 inch thick further subdivision is based on the form and orientation of the ice concentrations:

(a) individual ice crystals or inclusions,

(b) ice coatings on particles,

(c) random or irregularly oriented ice formations,

(d) stratified or distinctly oriented ice formations.

(C) Visible ice segregation greater than 1 inch thick

For descriptive purposes, ice formations greater than 1 inch thick may be considered as ICE. Two types of ice strata are recognized at present:

(a) ice with soil inclusions,

(b) ice without soil inclusions.

In some cases the occurrence of stratified or distinctly oriented ice formations in frozen soil increases to such an extent that the frozen material approaches "ice with silt lenses". Although the absence or inclusion of soil in ice is a first subdivision, the over-all form of the ice mass should also be included. Common forms of such "massive ice" are: random or irregularly oriented layers, vertical, wedge-shaped sheets, and variable chunks or blocks sometimes hundreds of square feet in area.

5. FIELD INVESTIGATIONS AND RECORDS

The scope of field investigations of permafrost and the amount and type of information required will depend largely upon the use for which it is intended. A discontinuity in the occurrence of permafrost (areas free of permafrost or large variations in the depth to permafrost) has many implications to construction. Accordingly, a sufficient number of observations must be made at a site so that the areal occurrence of permafrost is adequately delineated. This is particularly important in the discontinuous zone where permafrost occurs in scattered patches or "islands" in combination with areas of thawed ground.

All information collected should be recorded on data sheets. Typical sheets of recorded information have been included. Most of the specific details required are noted; other pertinent information may be added.

TABLE I

ICE DESCRIPTIONS

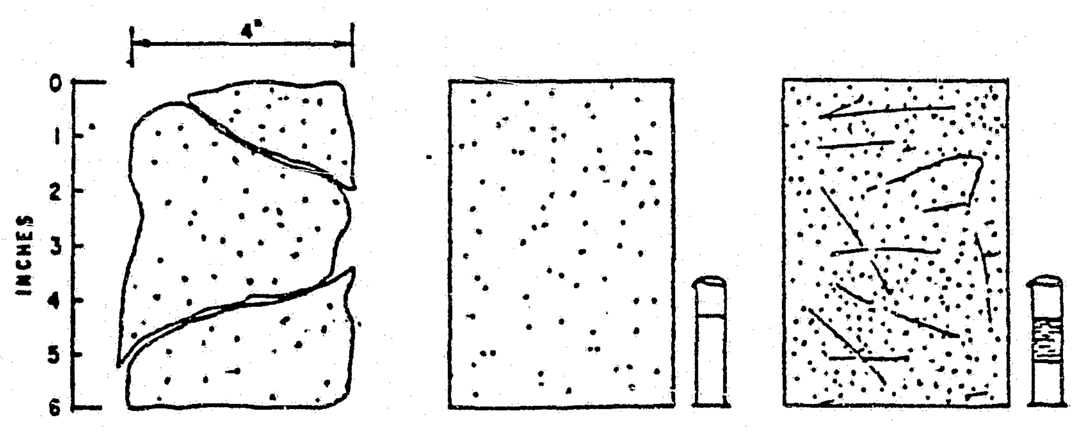
A. ICE NOT VISIBLE[a]

Group Symbol	Subgroup Description	Subgroup Symbol		Field Identification
N	Poorly bonded or friable	Nf		Identify by visual examination. To determine presence of excess ice, use procedure under note[b] and hand magnifying lens as necessary. For soils not fully saturated, estimate degree of ice saturation: medium, low. Note presence of crystals or of ice coatings around larger particles.
	Well-bonded: No excess ice	Nb	Nbn	
	Well-bonded: Excess ice	Nb	Nbe	

[a] *Frozen soils in the N group may, on close examination, indicate presence of ice within the voids of the material by crystalline reflections or by a sheen on fractured or trimmed surfaces. The impression received by the unaided eye, however, is that none of the frozen water occupies space in excess of the original voids in the soil. The opposite is true of frozen soils in the V group (see p. 14).*

[b] *When visual methods may be inadequate, a simple field test to aid evaluation of volume of excess ice can be made by placing some frozen soil in a small jar, allowing it to melt, and observing the quantity of supernatant water as a percentage of total volume.*

FIG A. ICE NOT VISIBLE



Nf — POORLY BONDED

Nbn — WELL BONDED - NO EXCESS ICE

Nbe — WELL BONDED - EXCESS ICE

LEGEND: SOIL - ▭ ICE - ■ or ▭

TABLE I (cont'd)

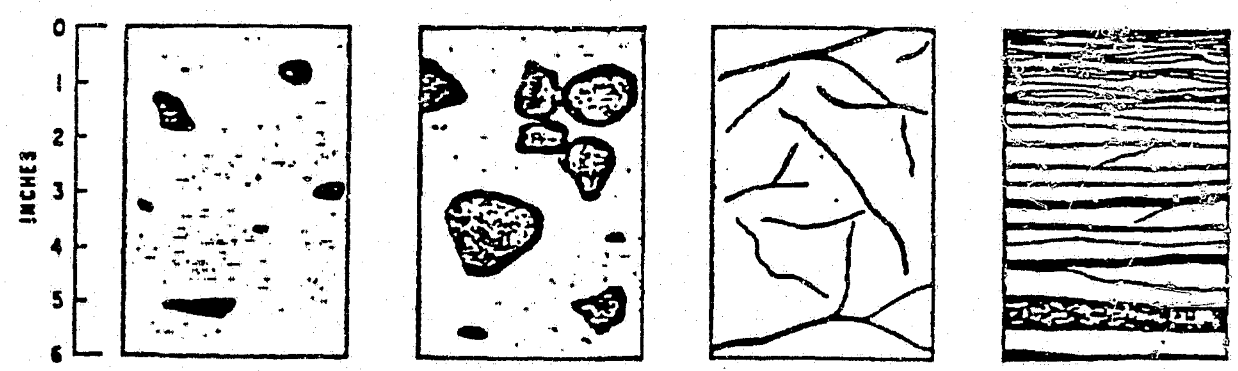
ICE DESCRIPTIONS

B. VISIBLE ICE—LESS THAN 1 INCH THICK[a]

Group Symbol	Subgroup Description	Subgroup Symbol	Field Identification
V	Individual ice crystal or inclusions	Vx	For ice phase, record the following when applicable: Location — Size Orientation — Shape Thickness — Pattern of arrangement Length Spacing Hardness, Structure, Colour } per Group C (see p. 16) Estimate volume of visible segregated ice present as percentage of total sample volume.
	Ice coatings on particles	Vc	
	Random or irregularly oriented ice formations	Vr	
	Stratified or distinctly oriented ice formations	Vs	

[a] *Frozen soils in the N group may, on close examination, indicate presence of ice within the voids of the material by crystalline reflections or by a sheen on fractured or trimmed surfaces. The impression received by the unaided eye, however, is that none of the frozen water occupies space in excess of the original voids in the soil. The opposite is true of frozen soils in the V group.*

FIG B. VISIBLE ICE LESS THAN ONE INCH THICK



Vx — INDIVIDUAL ICE INCLUSIONS

Vc — ICE COATINGS ON PARTICLES

Vr — RANDOM OR IRREGULARLY ORIENTED ICE FORMATIONS

Vs — STRATIFIED OR DISTINCTLY ORIENTED ICE FORMATIONS

LEGEND: SOIL – ▭ ICE – ■

TABLE I (cont'd)

ICE DESCRIPTIONS

C. VISIBLE ICE—GREATER THAN 1 INCH THICK

Group Symbol	Subgroup		Field Identification
	Description	Symbol	
ICE	Ice with soil inclusions	ICE + soil type	Designate material as ICE[a] and use descriptive terms as follows, usually one item from each group, when applicable: Hardness: HARD, SOFT (of mass, not individual crystals) Structure[b]: CLEAR, CLOUDY, POROUS, CANDLED, GRANULAR, STRATIFIED Colour (Examples): COLOURLESS, GRAY, BLUE Admixtures (Examples): CONTAINS FEW THIN SILT INCLUSIONS
	Ice without soil inclusions	ICE	

[a] Where special forms of ice such as hoarfrost can be distinguished, more explicit description should be given.
[b] Observer should be careful to avoid being misled by surface scratches or frost coating on the ice.

FIG C. VISIBLE ICE GREATER THAN ONE INCH THICK

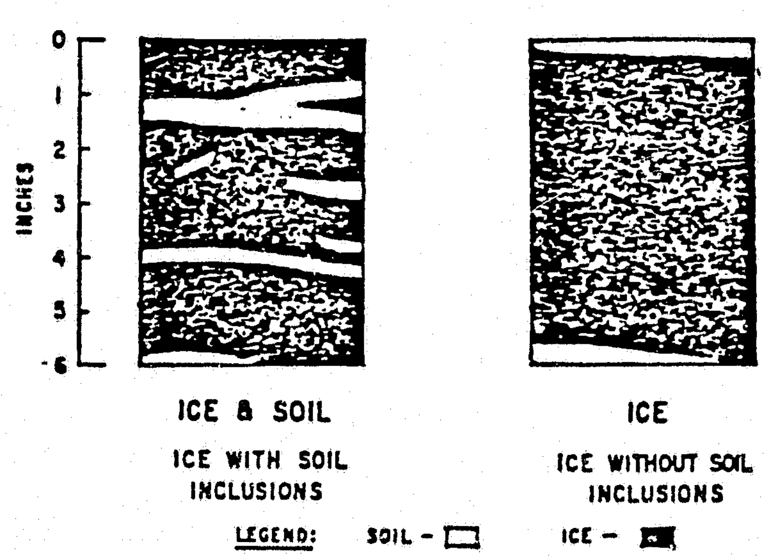


TABLE II

TERMINOLOGY

Ice Coatings on Particles are discernible layers of ice found on or below the larger soil particles in a frozen soil mass. They are sometimes associated with hoarfrost crystals, which have grown into voids produced by the freezing action.

Ice Crystal is a very small individual ice particle visible in the face of a soil mass. Crystals may be present alone or in combination with other ice formations.

Clear Ice is transparent and contains only a moderate number of air bubbles.

Cloudy Ice is relatively opaque due to entrained air bubbles or other reasons, but which is essentially sound and non-pervious.

Porous Ice contains numerous voids, usually interconnected and usually resulting from melting at air bubbles or along crystal interfaces from presence of salt or other materials in the water, or from the freezing of saturated snow. Though porous, the mass retains its structural unity.

Candled Ice is ice that has rotted or otherwise formed into long columnar crystals, very loosely bonded together.

Granular Ice is composed of coarse, more or less equidimensional, ice crystals weakly bonded together.

Ice Lenses are lenticular ice formations in soil occurring essentially parallel to each other, generally normal to the direction of heat loss and commonly in repeated layers.

Ice Segregation is the growth of ice as distinct lenses, layers, veins, and masses in soils commonly but not always, oriented normal to direction of heat loss.

Well-bonded signifies that the soil particles are strongly held together by the ice and that the frozen soil possesses relatively high resistance to chipping or breaking.

Poorly-bonded signifies that the soil particles are weakly held together by the ice and that the frozen soil consequently has poor resistance to chipping or breaking.

Friable denotes extremely weak bond between soil particles. Material is easily broken up.

Excess Ice signifies ice in excess of the fraction that would be retained as water in the soil voids upon thawing.

For a more complete list of terms generally accepted and used in current literature on Frost and Permafrost see Hennion, F. "FROST AND PERMAFROST DEFINITIONS", Highway Research Board, Bulletin 111, 1955.

APPENDIX F

GUIDE TO FIELD DESCRIPTION OF SOILS

LIST OF TABLES

NATIONAL RESEARCH COUNCIL

CANADA

ASSOCIATE COMMITTEE ON
SOIL AND SNOW MECHANICS

GUIDE

to the

FIELD DESCRIPTION

of

SOILS

for

Engineering Purposes

Technical Memorandum 37

OTTAWA

Guide to the Field Description of Soils for Engineering Purposes

The purpose of this document is to enable field men to describe soils as they are encountered and used for engineering purposes. It is not intended to be a soil classification system. Wherever possible the terms conform with those of the Unified Soil Classification System (in use in the United States) and with the British Standard Code of Practice for Site Investigation.

2. WHAT IS MEANT BY "SOIL"

The word soil, as used in an engineering sense, refers to that portion of the earth's crust which is fragmentary, or such that some individual particles may be readily separated by the agitation in water of a dried sample. Soil has been derived from bed-rock or organic matter by natural processes of chemical decomposition and physical disintegration and may have been subsequently modified by atmospheric or biological agencies.

3. MAJOR SOIL DIVISIONS

Soil may be grouped into three major divisions: coarse-grained, fine-grained, and organic.

(a) *Coarse-grained soils* may be described briefly as those soils made up largely of particles visible to the naked eye. Further subdivisions may be made according to the particle size as follows:

Cobbles and Boulders.—particles larger than 3 inches in diameter:
Cobbles 3 to 12 inches; boulders greater than 12 inches;

Gravel.—particles smaller than 3 inches in diameter and larger than the No. 4 sieve (approx. ¼ inch);

Sand.—particles smaller than the No. 4 sieve and larger than the No. 200 sieve (particles smaller than the No. 200 sieve are not visible to the naked eye).

(b) *Fine-grained soils* are made up of particles not visible to the naked eye. Plasticity and particle size, therefore, cannot be judged accurately without the use of refined testing techniques. For field identification, fine-grained soils may be classed as silt or clay by their behaviour in a few simple indicator tests described later (under the field identification procedures).

(c) *Organic soils* are placed in a separate group because of their appreciable content of organic matter. Soils which are mostly organic may be described as organic material, a term which includes peat, muskeg and peat moss. Partly organic soils which are largely mineral types are described as the predominant soil modified by the word "organic" e.g., organic silt.

4. DESCRIPTION OF SOILS

(a) *Coarse-grained soils*

For adequate description of coarse-grained or cohesionless soils, reference should be made to the *density, grading,* and *grain shape* of the soil.

Density.—Density is described by the terms "dense", "medium dense" and "loose" and should refer only to the density in place (i.e. in the ground). It is difficult to drive a 2- by 2-inch wooden picket into dense soil for more than a few inches. A 2- by 2-inch picket can be easily driven into loose soils. If the grains are "cemented" together, density cannot be estimated by this simple method.

Grading.—Grading is the term applied to the particle-size distribution of the soil. A uniform soil has a predominance of particles of one size, whereas a well-graded material has sizes assorted over a wide range, with no one size predominating. The word "uniform" is applied where it is obvious that one size is predominant, and "graded" if this is not the case.

Grain shape.—The terms used to describe grain shape are "angular", "subangular" and "rounded".

Angular particles have sharp edges and relatively plane sides with unpolished surfaces; subangular particles are similar to angular but have rounded edges; rounded particles have smoothly curved sides and no edges.

Additional descriptive notes.—Note should be made if the soil is stratified or contains any organic matter. If the soil contains some fine material, but not sufficient to cause cohesion, this should also be noted.

(b) *Fine-grained soils*

The descriptive terms for fine-grained or cohesive soils are obtained by reference to *consistency* in the undisturbed and remoulded states, *plasticity, structure, colour,* and *odour.*

Consistency.—Consistency varies mainly with water content and density and is described by the adjectives "hard", "stiff", "firm", and "soft". Occasionally, cohesive soils are "sensitive", i.e., they undergo a great loss of strength when disturbed or remoulded. It is necessary, when describing consistency, to state whether it is consistency in the undisturbed or remoulded states. The proper adjectives for consistency may be determined by attempting to penetrate the soil with the thumb. It is difficult to indent hard clays or silts with the thumb-nail. Stiff soils are readily indented with the thumb. Firm soils can be penetrated by moderate thumb pressure. Soft soils are penetrated easily with the thumb, and can be remoulded under light finger pressure.

Plasticity.—Plasticity is the ability to change shape and to retain the impressed shape when the stress is removed. The degree of plasticity of soils is the range in moisture content through which the soil remains plastic or is capable of being moulded. An indication of plasticity can be gained by manipulating the soil with the fingers when it is near the plastic limit. The plastic limit of a soil is defined as the moisture content at which a thread of soil one-eighth inch in diameter will begin to crumble when rolled further. Near the plastic limit, highly plastic soils will require considerable pressure to roll threads by hand, medium plastic soils a noticeable pressure, and soils weakly plastic can be rolled with little effort. The dry strength test is another indication of plasticity. Highly plastic soils are very hard when dry and cannot be broken by finger pressure. Medium plastic soils have a medium dry strength and can be crumbled only with difficulty. Weakly plastic soils have low dry strength and can be easily crumbled between thumb and forefinger.

Structure.—Structure is the term applied to the nature of the soil mass. The following terms are commonly used in describing special soil structures: "stratified", "fissured", "lensed", and "friable" or "blocky". The appearance of a fresh fracture may be used as an indication of structure. Stratification is evident when the soil has definite bedding planes and when these bedding planes are roughly parallel to one another. When there are definite stratifications, closely spaced, of alternating material the structure of the mass is described as "varved" or "laminated". Fissures are indicated when the soil breaks along definite planes of fracture, developing very little strength in fracturing. Near the surface, fissures may be indicated by slight discoloration along the planes. When the soil breaks along a fissure, the surface of the fracture will be very clean and glossy. A lensed structure is caused by the inclusion of small pockets of foreign material. For instance, a clay may have small lenses of sand scattered throughout. A friable or blocky structure is that found when a cohesive soil can be broken into small lumps easily with the lumps themselves more difficult to break.

Colour.—Colour indicates the depth of weathering in a soil and may also be helpful in identifying similar soils in the same region.

Odour.—Odour of the soil will normally indicate the presence of organic matter.

(c) *Organic soils*

The descriptive terms used for inorganic soils can be used to describe partly organic soils. For organic material, a separate classification system is necessary. This will be described in a booklet similar to this, based upon studies of Dr. N. W. Radforth.

5. FIELD IDENTIFICATION PROCEDURE

Most soils consist of mixtures of various particle sizes. Therefore the first step is to decide which of the principal fractions or characteristics predominate, then to decide which of these acts as a modifier. For example, a sand containing some silt would be called a silty sand. Table I lists the principal soil divisions with their characteristics which lead to identification.

Boulders, cobbles, gravel, and sand are identified by visual examination as all their particles are visible to the naked eye. Size is the criterion of identification.

Fine-grained soils can only be identified by more indirect means. The tests listed below may be used to establish the identity of these soils:

(a) *Shaking test*

When a wet pat of soil is shaken vigorously in the hand, the surface will become glossy and show free water. If the pat of soil is then squeezed in the fingers, the free water may disappear and the surface become dull, i.e. dilates. With clay soils this phenomenon will not be noticeable but with silts and fine sands a rapid or good reaction will be exhibited;

(b) *Shine Test*

If a moist lump of soil is stroked with considerable pressure with the flat of a pen knife blade or finger-nail, the type of surface imparted is an indication of the soil: if a shiny surface results, the presence of clay is indicated; silt is indicated if a dull surface is produced;

(c) *Dry Strength Test*

If a small piece of dry fine-grained soil is broken or crushed with the fingers, the breaking strength is an indication of the relative amounts of silt or clay. Very low dry strength is indicated when the soil powders readily in the fingers, and may be taken as an indication of a sandy silt or silt. Medium dry strength is shown by difficulty in powdering the soil by finger pressure, but the soil can be broken into small pieces without great difficulty. This state indicates silty clays and clays of medium plasticity. High dry strength is indicated when the pat of dry soil cannot be broken with the fingers. A highly plastic clay is indicated by this condition.

In addition to the tests mentioned above, clay sticks to the fingers when wet, and does not wash off readily, whereas silt will wash away easily or brush off if dry. When a small amount of soil is placed between the teeth, the presence of grit will indicate silt or sand, but if no grit is detected a pure clay is present.

Organic soils are very compressible and spongy. Purely organic soils are easily recognized by their matted or fibrous structure. Partly organic soils may behave as a silt or clay, but are very compressible and usually have a characteristic odour.

6. PARTICULAR SOIL NAMES AND CONDITIONS

Each soil has a definite origin, and many of its characteristics depend upon the environment under which it was formed. In some cases, the geological origin can only be determined after study by the specialist. In other cases, the nature of the soil is indicative of the origin, and the soil can be described most adequately by using a special name.

(a) *Topsoil*

Topsoil is the layer of soil on the surface which will support plant life. It is characterized by the presence of organic material. Topsoil should be modified by reference to the predominant inorganic soil.

(b) *Fill*

Fill is a man-made deposit of natural soils or waste materials. It can usually be identified by the inclusion of grass, twigs, cinders, bricks, glass, etc., and by a layer of topsoil or profile development under the fill. To describe fill, an adjective indicating the predominant soil should be used, i.e. sand and gravel fill, clay fill, rubbish fill, or cinder fill.

(c) *Local names*

Frequently soils in one area are given local names by the inhabitants. These names give a vivid description of the soil, e.g. "bull's liver". To promote uniformity in soil terminology, such local names should be omitted or used only to supplement the description of the soil.

(d) *Permafrost*

In northern parts of Canada, the soil remains perennially frozen. These areas are known as permafrost regions. In such regions, the same soils exist as in other areas, but it is necessary not only to identify the soil, but to note the presence of permafrost, and if possible the depth of the "active zone", i.e., the depth to which the soil thaws during the summer, and the thickness of organic cover if any.

7. OTHER FACTORS IN SOIL DESCRIPTION

If the vertical section of a boring or test pit is being examined, such data as the date of observation, depth below surface, elevation of surface, level of groundwater, and location of the boring or test pit must be recorded. A brief description of the method of sampling is necessary to show whether the sample can be regarded as undisturbed.

8. CHECK LIST FOR FIELD DESCRIPTION OF SOILS

General

The check list below may be used as a guide in a soil description. It includes the terms necessary for an adequate description of the soil. Any additional descriptive terms, which the user may think necessary, should be included to give a more complete description.

(a) *Environmental*

Sample No.		*Site*	
Detailed Location		*Date*	
Depth Below Surface		*Surface Elevation*	
Boring	*Test Pit*	*Excavation*	*Other*
Remarks on Method of Sampling			
Groundwater Level			

(b) *Check list for coarse-grained soils*

Soil Subdivision	Boulders and Cobbles, Gravel, Sand		
Size of Maximum Particles			
Grain Shape	Angular	Subangular	Rounded
Grading	Uniform	Graded	
	Fine	Medium	Coarse
Density	Loose	Medium	Dense
Structure	Stratified	Nonstratified	
Colour			
Odour			
Organic Material			
Presence of Fines			

(c) *Check list for fine-grained soils*

Soil Subdivision

Sandy Silt	Silt	Clayey Silt	Silty Clay	Clay

Consistency	Hard	Stiff	Firm	Soft
Dry Strength	None	Low	Medium	High
Reaction to Shaking Test	Rapid	Slow	None	
Reaction to Shine Test	No clay		Clay present	
Reaction to Taste Test	Silt or sand present		No Silt or Sand	
Toughness at Plastic Limit	Weakly plastic	Medium plastic	Highly plastic	
Structure	Stratified		Fissured	
	Friable or Blocky	Lensed	Nonstratified	
Odour				
Colour	Mottled			

TABLE 1

General Basis for Field Description of Soils

Major Divisions	Subdivisions	Field Identification	Information for Description
COARSE-GRAINED SOILS	COBBLES AND BOULDERS	Larger than 3 inches diameter —cobbles 3 to 12 inches —boulders greater than 12 inches	Density Particle Shape
	GRAVEL	Smaller than 3 inches but larger than No. 4 sieve (approx. ¼ inch)	Grading Density Particle Shape Stratification
	SAND	Smaller than No. 4 sieve but larger than No. 200 sieve. Particles smaller than No. 200 sieve are not visible to the naked eye.	Grading Density Particle Shape Stratification Organic Matter
FINE-GRAINED SOILS	SILT	Exhibits dilatancy (reacts to the shaking test). Powders easily when dry only slight dry strength. Gritty to the teeth. Dries rapidly. No shine imparted when moist and stroked with knife blade.	Consistency Undisturbed Remoulded Plasticity Dry Strength Structure
	CLAY	Not dilatant. Possesses appreciable dry strength. When moist, sticks to fingers and does not wash off readily. Not gritty to the teeth. When moist a shiny surface is imparted when stroked with knife blade.	Consistency Undisturbed Remoulded Plasticity Dry Strength Structure
ORGANIC SOILS	PARTLY ORGANIC —organic clay —organic silt etc.	Depending on amount of organic material, these soils usually have some of the characteristics of their inorganic counterparts: usually highly compressible (spongy) usually have characteristic odour	Consistency Undisturbed Remoulded Plasticity Dry Strength Structure
	ORGANIC MATERIAL	Fibrous structure—usually brown or black when moist. Spongy. Usually has characteristic odour.	Organic terrain including muskeg, peat and peat moss

NOTE: Please also see EXHIBIT 9 for a more detailed chart showing the Unified Soil Classification System.

APPENDIX G

LUGEONS

TABLE OF CONTENTS

1.0 LUGEONS MEASURE HYDRAULIC PRESSURE TESTING

One of the most commonly used units for measuring the ability of water to pass through bedrock is the "Lugeon" criteron developed and much utilized in Europe since 1933. This relatively simple method yields numerical values acceptable for evaluating the resistance of rock foundations to the effects of groundwater flowing through open or partially open channels resulting from joints, bedding-plane partings, faults and other recurrent fractures or fissures.

The Lugeon value is measure of permeability obtained by pumping water into a bedrock foundation. To go a step further and calculate Lugeons into "velocity" type permeability units such as feet/year or cm./sec. is not realistic. This is because bedrock foundations owe their ability to pass water to discontinuous openings in the rock mass. The nonuniformity of the aperture and spacing of these openings (fractures) in most rock foundations do not obey Darcey's Law. The presence of absence of a network of fissures, at any particular location, will result in a profound difference in velocity-permeability magnitudes.

The basic definition of the hydraulic pressure test, in terms of Lugeons, is a water take of 1 litre of hole per minute at 10 Lars (150 psi) pressure or in more customary units:

$$\frac{1820 \times \text{Rate of less in gal. per min.}}{\text{interval tested (ft.)} \times \text{Net pressure (psi)}} = \text{Lugeon Value}$$

To get a sense of proportion for the Lugeon unit it might be noted that:

1 Lugeon indicates virtually a tight bedrock foundation.

10 Lugeons usually warrents some remedial treatment.

100 Lugeons definitely requires corrective measures as heavily jointed or fractured bedrock with relative open joints or sparsely cracked rock with wide open joints, is indicated.

As can be seen from the above examples, the Lugeon scale decreases in sensitivity as the values increase. The greatest sensitivity and importance are in the low values form 1 to 5. When values in the range of 50 are reached, an accuracy in the order of ± 10 units is all that is warranted, and when 100 units are reached, ± 30 units is all the accuracy needed. Although the scale has no upper limit, beyound 100 units the values become meaningless.

ROUTINE INTERPRETATION OF THE LUGEON WATER-TEST

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SUMMARY

A relatively simple routine method of calculating and interpreting the "modified" Lugeon water test is described. The method is used for assessing the need for foundation grouting at dam sites; it comprises calculations of lugeon values for each of five test runs at increasing and then decreasing pressures, followed by interpretation of the pattern of results, and hence selection of an appropriate representative permeability.

The relative frequency of different behaviour patterns is indicated by reference to 811 actual tests.

1.1 Development of the method

The method and interpretations presented here evolved as a result of various difficulties and complications. This evolution is believed to have been generally concurrent with the corresponding development of similar methods by other people in this field, but because little of this can be found in the technical press (an exception is Lancaster Jones 1975), it has been set down here. Others may be able to improve on it.

The method, as described here, reached its present state of development in 1970, and has since been extensively used on a number of dam sites in investigation work preliminary to grouting design.

1.2 The Method

1. Five consecutive water (pump-in) tests are done, each of ten minutes duration; the
   - 1st 10 minutes is at a low pressure—(pressure "a")
   - 2nd 1 minute run is at a medium pressure—(pressure "b")
   - 3rd 1 minute run is at a peak pressure—(pressure "c")
   - 4th 1 minute run is at a medium pressure—(pressure "b" again)
   - 5th 1 minute run is at a low pressure—(pressure "a" again)

2. A single lugeon value is then calculated for each one of these five tests, using the formula:

$$\text{lugeon value} = \text{water taken in test (litres/metre/min)} \times \frac{10 \text{ (bars)}}{\text{test pressure (bars)}} \quad (1)$$

3. Having calculated the five lugeon values, they are inspected and compared and an appropriate decision can then be made as to which of the five values is accepted as the permeability reported from the test.

Table 1 shows the interpretations placed on the various patterns of the five values, and then the values that are used as the reported permeability, an elaboration of each of the patterns, followed by the interpretations placed on each, follows.

1.3 Explanation of the Calculation

= 1 MPa.

Lugeon (1933) in his standard test, specified a pressure of 10 bars (150 p.s.i.; 1,000 kPa). The "modified" test usually uses lower pressures than this because:

(i) a range of pressures (rather than a single pressure) is desirable, as discussed in this paper.
(ii) use of a pressure as high as 10 bars is not always advisable, particularly at shallow depths in the weaker rocks.
(iii) satisfactory results can be readily obtained with lower pressures.

When using the "modified" test, it is necessary to convert the results into values which would have been (supposedly) obtained if the "definition" pressure (10 bars) had been used. Lugeon values then result from this conversion.

Formula (1) carries out this conversion. However, it presumes direct proportion when relating the pressures: this presumption is only valid if flow through open jointing is laminar. Accordingly, when significantly different lugeon values result from the calculation carried out for the five test runs, it becomes immediately apparent that the flow is not laminar, or that some other factor is exerting an influence.

1.3.1 Group A–Laminar flow

Laminar flow is regarded as being indicated when the lugeon values calculated for the five tests are all about the same.

The reported permeability of the stage is taken as the average of the five values (to the nearest whole number).

1.3.2 Group B–Turbulent flow

When the lugeon value calculated for the peak pressure ("c") is less than those for the two medium pressure tests, and also when those for the low pressure tests are approximately

TABLE I: *Lugeon patterns for various occurrences during testing: Their interpretation and percentage occurrences*

TEST PRESSURES (SHOWN IN APPROX. RELATIVE MAGNITUDE)	LUGEON PATTERN — LUGEONS CALCULATED FOR EACH 10 MINUTE RUN* (RELATIVE MAGNITUDES SHOWN IN GENERALIZED PATTERNS; ACTUAL MAGNITUDES CAN VARY WIDELY)	CHARACTERISTICS OF THE PATTERN, & INTERPRETATION OF IT	WHICH OF THE LUGEON VALUES IN THE PATTERN SHOULD BE USED AS THE REPORTED PERMEABILITY?	PERCENTAGE OF ACTUAL CASES IN EACH GROUP (SUMMARISED FROM TABLE VI) WHEN THE REPORTED PERMEABILITY IS: 1, 2, OR 3 LUGEONS	4 OR MORE LUGEONS
GROUP A - LAMINAR FLOW					
1ST TEN MINUTE RUN a; 2ND b; 3RD c; 4TH b; 5TH a		ALL 5 LUGEONS ARE ABOUT EQUAL HENCE LAMINAR FLOW	USE THE AVERAGE OF THE FIVE LUGEONS (TO THE NEAREST WHOLE NUMBER)	78%	5%
GROUP B - TURBULENT FLOW					
1ST TEN MINUTE RUN a; 2ND b; 3RD c; 4TH b; 5TH a		LOWEST LUGEON VALUE OCCURRING AT HIGHEST PRESSURE HENCE TURBULENT FLOW	USE THE LUGEON VALUE FOR THE HIGHEST PRESSURE	13%	53%
GROUP C - DILATION					
1ST TEN MINUTE RUN a; 2ND b; 3RD c; 4TH b; 5TH a		HIGHEST LUGEON OCCURRING AT HIGHEST PRESSURE HENCE DILATION	USE THE LUGEON VALUE FOR THE LOWEST (OR MEDIUM) PRESSURES	1%	9%
GROUP D - WASH-OUT, ETC					
1ST TEN MINUTE RUN a; 2ND b; 3RD c; 4TH b; 5TH a		LUGEONS INCREASING AS TEST PROCEEDS HENCE THE TEST IS CAUSING CHANGES TO THE FOUNDATION	USE THE HIGHEST LUGEON VALUE UNLESS SPECIAL REASONS REQUIRE OTHERWISE	2%	21%
GROUP E - VOID FILLING					
1ST TEN MINUTE RUN a; 2ND b; 3RD c; 4TH b; 5TH a		LUGEONS DECREASING AS TEST PROCEEDS HENCE THE TEST IS GRADUALLY FILLING EXTENSIVE VOIDS	USUALLY USE THE FINAL LUGEON VALUE	6%	12%

*CALCULATED USING DIRECT PROPORTION TO RELATE TO LUGEON'S "DEFINITION" PRESSURE OF 10 BARS (150 P.S.I.)

THIS TABLE GIVES GENERAL PATTERNS ONLY. DISCRETION IS NECESSARY IN ADAPTING THESE TO THE INEVITABLE VARIATIONS MET IN PRACTICE equal in value, the flow is classed as "turbulent". The two medium pressure values are usually equal to each other, and are slightly less than the low pressure values.

Guerra *et al.* (1968), Arhippainen (1970), Lancaster-Jones (1975) and others, have discussed the detection of turbulent flow. They have shown that for flow which is solely turbulent test pressures have a square root relationship, compared with the direct relationship of laminar flows. They have also given instances of cases where the relationship was neither a square root one, nor a direct one, but was intermediate.

Accepting that turbulent flow is indicated by the square root relationship, and that laminar flow follows a direct relationship, it follows that if the five lugeon values calculated by formula (1) are not generally equal, but instead show a lower value for the peak pressure than for the medium ones, while the low pressure ones are equal, then the flow is non-laminar. It is conveniently designated as "turbulent"; this is, however, not an accurate description, but suffices.

Because a test stage usually cuts many open cracks of various sizes, and because the finer ones are liable to exhibit laminar flow, and the wider ones will have turbulent flow, the overall effect is likely to be a mixture of both types of flow. Hence it is not surprising that the pure square root relationship is not solely experienced as the only alternative to laminar flow. Thus "turbulent" is a convenient all-embracing designation for all flows apart from solely laminar flow.

The reported permeability of a stage which exhibits such "turbulent" flow is taken as that calculated for the peak pressure. It can be argued, with some justification, that if the flow is in the vicinity of genuine turbulent flow, the lugeon value should be recalculated on a square root basis. This would produce a slightly lesser value. However, the proportion of solely turbulent flow cannot be reliably assessed when the usual mixture of laminar and turbulent flow exists. There seems little realism therefore in recalculating on a turbulent basis unless the result is quoted as a range, with the solely turbulent value at one extreme, and the solely laminar flow at the other. Rarely is this warranted.

1.3.3 Group C–Dilation

When the lugeon value for the peak pressure ("c") is greater than for the two low pressure tests, and when these two low pressures have produced approximately equal values, the occurrence of temporary dilatancy of the rock mass is inferred. This pattern of values could be regarded as the reverse of that for Group B–turbulent flow.

The high value for the peak pressure (it sometimes also occurs to a lesser extent on the medium pressures), is interpreted as the result of fissures opening (temporarily) or materials being compressed by the test water. This temporary condition is distinguished from permanent movements of the same type by the return of lugeons at the final (low) pressure test to the value obtained at the initial (low) pressure test.

The dilatancy effect, because of its temporary nature, is usually disregarded, and accordingly the reported permeability of the stage is that obtained for the lowest pressures, or alternatively for the medium pressures if these are less than for the low pressures (indicating that "turbulent" flow was occurring prior to the dilation).

1.3.4 Group D–Wash-out of joint filling materials etc.

A progressive increase in the five lugeon values, without any return to pre-peak pressure values after the peak has been passed, is regarded as indicative of permanent washing-out of joint filling materials, or permanent rock movements caused by the testing. Too much of this sort of thing is a warning that test pressures are too high!

The reported permeability is usually taken as that measured for the final run (a low pressure run); this presumes that the peak pressure bears some resemblance to eventual in-service pressures which would produce generally similar washing-out or dilation.

Group E–Void filling

A progressive decrease in the five lugeon values is regarded as an indication that the test is gradually filling empty voids, joints, etc., which are semi-blind (i.e. water cannot easily escape from them). A properly conducted test, where the foundation is fully saturated before test readings are commenced, avoids this problem, but is not always possible to organize.

Guerra *et al.* (1968) in discussion of this type of flow, suggest that it may be partly due to capilliary resistance to penetration in fine cracks.

The adopted permeability for reporting purposes is the value obtained for the final run. However, where possible, an extended test is preferred, in which readings are not commenced until all voids are filled.

1.3.5 Applications of test results

The main use for the permeability information thus obtained is for the assessment of foundations to decide when grouting is warranted. Figure 1 shows an outline of how this is done, in relation to various lugeon values. This figure also shows, incidentally, criteria for evaluating (during the course of a grouting operation) when sufficient grouting has been done; the water test used for this evaluation is not as elaborate as the investigatory one described in this paper.

The need for grouting is not solely decided upon the permeability considerations shown in Fig. 1, geological and other local factors are also considered, but these issues are beyond the scope of this present paper. They are dealt with in some detail in Houlsby (1976).

Rarely, in Australian dam sites, is clay or chemical grouting used for rock grouting. Neat cement grout is almost solely used. It is generally found that if cracks are too fine for effective penetration of cement grout, then grouting is unnecessary.

A partial difference in purpose is therefore apparent between the author's usage of the permeability information, and the usage applied to essentially similar information by Guerra *et al.* (1968).

The difference might be summarized as follows:

The *magnitude* of the reported lugeon values is used, both by the author and Guerra *et al*, to decide whether grouting is necessary, but, the *type of grout* is decided by Guerra *et al.* from considerations of whether flow is laminar or turbulent (or occurs in some combination of them), whereas the author has no need to make this decision because of the virtually exclusive use of cement grout. Instead, he uses these same considerations whether flow is laminar, turbulent, etc.) to assess testing conditions, and hence to decide which test values to accept as the reported permeabilities.

Laminar flow is interpreted by Guerra *et al.* as indicating the presence of granular material in rock joints, and hence a need, where grouting is required, for some use of chemical grout. The author, however, for Australian sites, has frequently experienced laminar flow in fine cracks, quite free of granular materials. These cracks have, on occasions, been inspected by borehole periscopes and similar devices, and are frequently groutable at reasonable, non-dilating pressures, with cement grout, without recourse to finer grouts.

It has been noticed that laminar flow predominates where the take is 1, 2 or 3 lugeons, and that for 4 or more lugeons, "turbulent" flow is the commonest type. This behaviour is compatible with the relatively fine cracks usually encountered where the permeability is 1, 2 or 3 lugeons, and with the mixture of (generally wider) crack sizes experienced in sites where the permeability exceeds 4 lugeons.

In accordance with the criteria of Fig. 1, grouting of foundations with permeabilities of 1 to 3 lugeons is commonly unnecessary, this being the range of solely laminar flow. Grouting is commonest in the mixture of crack sizes having greater permeability than 3 lugeons, and having flow designated as "turbulent".

Fracture Porosity

Snow (1968) has presented a method for estimating "fracture porosity". As noted by Houlsby (1969) the method appears of doubtful validity for prediction of cement grout takes in small areas. Snow (1969) has emphasized that this method is primarily relevant to chemical grouting. Therefore, for reasons which include these, Snow's method does not appear suited for permeability assessments related to cement grouting.

3. Geological effects

Geological factors, such as the degree of roughness of crack walls, crack frequency, orientations, straightness, and so forth, obviously affect the hydraulic flow during the water testing. They also similarly affect subsequent grouting, though with modifications due to the differing rheology of the grout.

These geological factors can usually be observed during investigation operations, and some adjustment can, if desired, be made for them. This adjustment need be no more than a mental one, applied to the grouting criterion, but is rarely warranted because of the commonly encountered scatter of water test results. This scatter is due to a variety of causes, including geology, and is usually manifest by a considerable scatter of lugeon values determined even from neighbouring holes in fairly uniform areas. To make use of

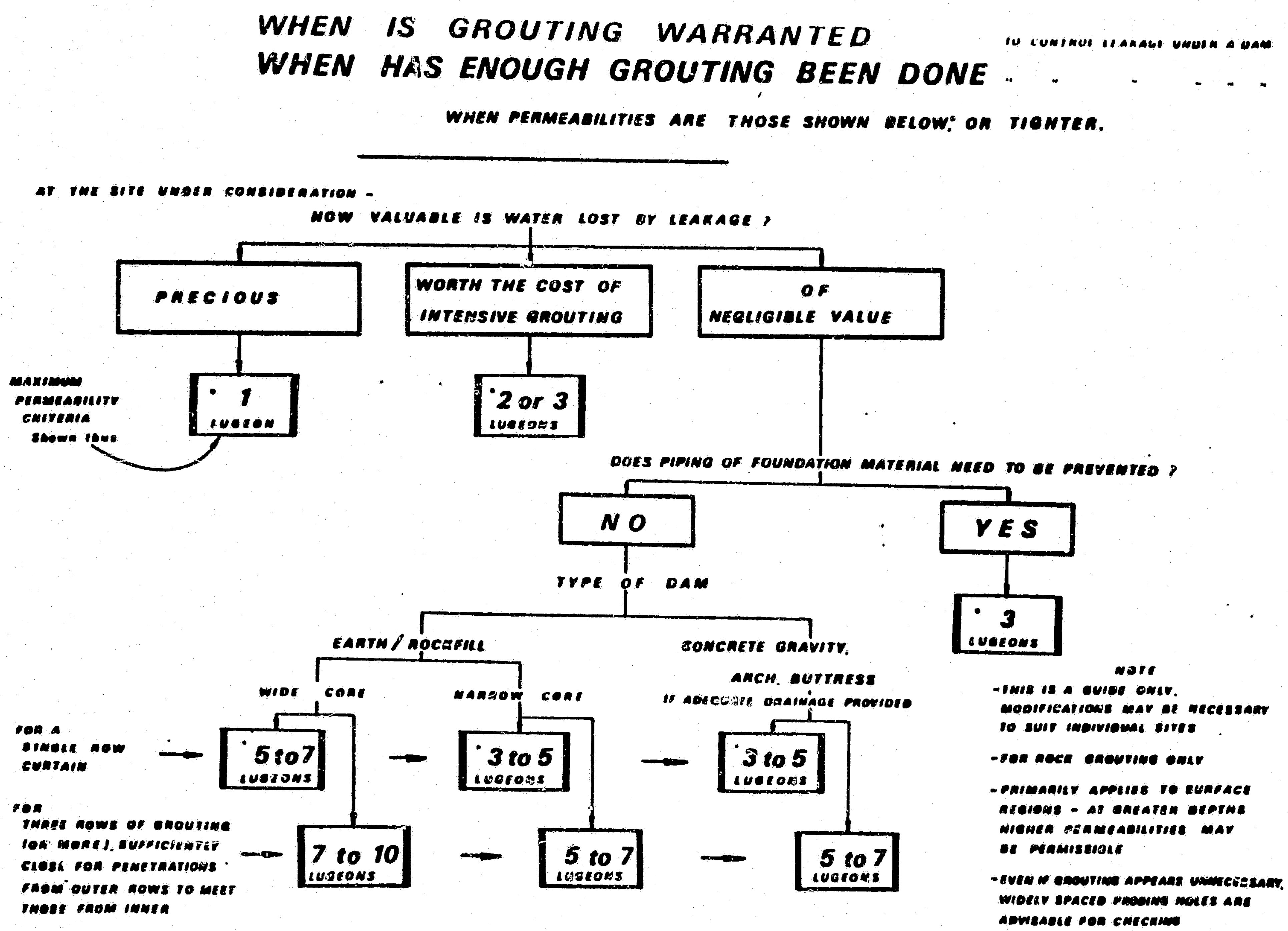
WHEN IS GROUTING WARRANTED
WHEN HAS ENOUGH GROUTING BEEN DONE
WHEN PERMEABILITIES ARE THOSE SHOWN BELOW* OR TIGHTER.
AT THE SITE UNDER CONSIDERATION -
HOW VALUABLE IS WATER LOST BY LEAKAGE ?
PRECIOUS
WORTH THE COST OF INTENSIVE GROUTING
OF NEGLIGIBLE VALUE
MAXIMUM PERMEABILITY CRITERIA Shown thus
* 1 LUGEON
* 2 or 3 LUGEONS
DOES PIPING OF FOUNDATION MATERIAL NEED TO BE PREVENTED ?
NO
YES
* 3 LUGEONS
TYPE OF DAM
EARTH / ROCKFILL
CONCRETE GRAVITY, ARCH, BUTTRESS
IF ADEQUATE DRAINAGE PROVIDED
WIDE CORE
NARROW CORE
FOR A SINGLE ROW CURTAIN
* 5 to 7 LUGEONS
* 3 to 5 LUGEONS
* 3 to 5 LUGEONS
FOR THREE ROWS OF GROUTING (OR MORE), SUFFICIENTLY CLOSE FOR PENETRATIONS FROM OUTER ROWS TO MEET THOSE FROM INNER
7 to 10 LUGEONS
5 to 7 LUGEONS
5 to 7 LUGEONS
NOTE
- THIS IS A GUIDE ONLY. MODIFICATIONS MAY BE NECESSARY TO SUIT INDIVIDUAL SITES
- FOR ROCK GROUTING ONLY
- PRIMARILY APPLIES TO SURFACE REGIONS - AT GREATER DEPTHS HIGHER PERMEABILITIES MAY BE PERMISSIBLE
- EVEN IF GROUTING APPEARS UNNECESSARY, WIDELY SPACED PROBING HOLES ARE ADVISABLE FOR CHECKING

FIGURE 1

such scatters of results for the purpose of Fig. 1, a representative value is used (rather than mathematical averaging, which can be very misleading when used with lugeons, for the significance of individual tests depends on their numerical value), taking cognisance of geological boundaries, and of the numerical values of the lugeons involved. This assessment requires experience. Much of the difficulty arises because the lugeon scale does not have linear significance. In the lower values of the range (say below 7 lugeons) every unit change is of relevance. As values get higher, two-unit changes (in the range 7–15 lugeons, say) widen out to five (15–50 lugeon range) and then to ten (50–100 range) for equal significance. Beyond 100 lugeons, values become impossibly high to distinguish and are merely grouped as "greater than 100". Hence mere mathematical averaging of lugeons, without some weighting to suit the significance of individual values, should be avoided.

The various types of rock in which testing has been carried out indicate that the test method is generally applicable, virtually irrespective of rock type. However, caution is desirable at each fresh site in case there is some (very unusual) geological factor which may be important.

4. Examples of the flow groups

In order to give an appreciation of the relative frequency of actual occurrences of the various flow groups, Table 1 shows percentages of actual cases. These are grouped in

1, 2 or 3 lugeon cases

4 lugeons or greater cases.

The percentages are taken from 811 different test stages at 4 dam sites. Table 2 gives a more detailed breakdown of the same figures: they are generally representative of many other dam sites.

As commented previously, laminar flow is predominant in the 1, 2 or 3 lugeon cases, and turbulent flow is commonest when the permeability exceeds 4.

Dilation (Group C) is of minor occurrence (certainly less in these cases than Sabarly (1968) and others have implied).

Wash-out (Group D) has occurred in a significant number of cases when the permeability exceeds 4.

Void-filling (Group E) is of minor occurrence.

In these figures, stages marked as "tight" on Table 2 are those where tests showed nil take or takes as high as 0.5 lugeons (because lugeons are rounded off to the nearest whole number 0.5 lugeons is reported as zero.)

5. Test pressures, reporting method

The pressures used for the results quoted vary with the depth, and are taken generally from the relationship:

Low pressure "a" (at surface) in p.s.i. = 0.4 × depth in ft (max = 50 p.s.i.)

Medium pressure "b" (at surface) in p.s.i. = 0.7 × depth in ft (max = 100 p.s.i.)

TABLE 2

Number off and Percentages of tests coming within the various Groups

	SITE A Breccias, Greywacke, Siltstone. Very hard, sound. Fracturing moderate.			SITE B Granite. Very tough, massive. Negligible jointing or fracturing.			SITE C Shales, Sandstone, Grits. Very weak to very tough. Some beds highly fractured.			SITE D Jasper, Siltstone, Tough. Highly fractured and brittle.			Totals for the four sites		
	No off	% overall		No off	% overall		No off	% overall		No off	% overall		No off	% overall	
TIGHT	133		30	37		17	8		8	8		22	186		23
1, 2 or 3 Lugeons		%			%			%			%			%	
Group A-Laminar Flow	150	78	33	111	85	52	12	46	11	6	60	16	279	78	34
" B-Turbulent Flow	31	16	7	9	7	5	5	19	5	0	0	0	45	13	5
" C-Dilation	1	1	0	3	2	1	0	0	0	1	10	3	5	1	1
" D-Wash-out, etc.	3	2	1	1	1	0	3	12	3	1	10	3	8	2	1
" E-Void Filling	7	3	2	6	5	3	6	23	6	2	20	5	21	6	3
Totals	192	100		130	100		26	100		10	100		358	100	
4 or More Lugeons		%			%			%			%			%	
Group A-Laminar Flow	2	2	0	7	15	3	2	3	2	3	16	8	14	5	2
" B-Turbulent Flow	65	50	14	23	49	11	49	68	46	4	21	11	141	53	17
" C-Dilation	12	9	2	5	10	2	5	7	5	3	16	8	25	9	3
" D-Wash-out, etc.	35	27	8	6	13	3	8	11	7	6	31	16	55	21	7
" E-Void Filling	15	12	3	6	13	3	8	11	7	3	16	8	32	12	4
Totals	129	100		47	100		72	100		19	100		267	100	
Overall Totals	454		100	214		100	106		100	37		100	811		100

Peak pressure "c" (at surface) in p.s.i. = 1.0 · depth in ft (max = 150 p.s.i.) or their metric equivalents.

Figure 2 is an example of a typical (metric) report form; it shows a number of cases from various holes illustrating the various Groups, and shows the calculated lugeon values and the representative values selected for each stage.

Graphical plotting of lugeon values is no longer required.

Omission of pressure corrections

It is not the author's practise for the test pressures used for calculations by formula (1) to be corrected for head losses in testing equipment, nor corrected for the position of the water table. Although Lugeon (1933) and others have applied corrections for these, the

REPORT FORM & CALCULATIONS

DATE	STAGE	TESTING TIMES				GAUGE PRESSURE BARS		WATER INJECTED INTO HOLE IN TESTING PERIOD (TOTAL QUANTITY READ FROM WATER METER)	LUGEON VALUE		GROUP
19....	METRES	REQUIRED MINS.	ACTUAL MINS.	CLOCK TIMES FROM	CLOCK TIMES TO	REQUIRED	ACTUAL	LITRES	FOR EACH TEST	REPRESENTATIVE FOR TESTED STAGE	
	FROM	10	10	0820	0830	2 8	2 8	29	2		A LAMINAR
	27 67	10	10	0830	0840	5 0	5 0	46	1		
	TO	10	10	0840	0850	7 8	7 8	81	2	2	
	33 87	10	10	0850	0900	5 0	5 0	54	2		
		10	10	0900	0910	2 8	2 8	25	1		
	FROM	10	10	1230	1240	2 8	2 8	120	7		B TURBULENT
	24 08	10	10	1240	1250	4 5	4 5	180	6		
	TO	10	10	1250	1300	6 8	6 8	198	5	5	
	30 26	10	10	1300	1310	4 5	4 5	178	6		
		10	10	1310	1320	2 8	2 8	150	9		
	FROM	10	10	0855	0905	0 7	0 7	20	5		C DILATION
	6 20	10	10	0905	0915	1 7	1 7	57	6		
	TO	10	10	0915	0925	2 7	2 7	122	8		
	12 20	10	10	0925	0935	1 7	1 7	76	7		
		10	10	0935	0945	0 7	0 7	20	5	5	
	FROM	10	10	1315	1325	2 0	2 0	62	5		D WASH-OUT
	15·16	10	10	1325	1335	3·0	3 0	114	6		
	TO	10	10	1335	1345	4 8	4 8	198	7		
	21·29	10	10	1345	1355	3 0	3 0	153	8		
		10	10	1355	1405	2 0	2 0	119	10	10	
	FROM	10	10	1140	1150	2 8	2 8	118	5		E VOID FILLING
	22·57	10	10	1150	1200	4 5	4 5	151	4		
	TO	10	10	1200	1210	6 8	6 8	208	4		
	30 42	10	10	1210	1220	4 5	4 5	94	3	3 or 1	
		10	10	1220	1230	2 8	2 8	28	1		
	FROM	10	10	1512	1522	0 8	0 8	269	56		*
	3 60	10	10	1522	1532	1·4	1 2	432	60		
	TO	10	6	1532	1538	2 4	1 2	259	60	60	
	9·60	10	10	1538	1548	1 4	1 2	432	60		
		10	10	1548	[illegible]	[illegible] 8	0 8	279	58		

*PARTLY BEYOND PUMP CAPACITY

FIGURE 2

author, in common with Arhippainen (1970) has found that with flow channels of reasonable size, head losses are not worth correcting for within the degree of realism actually needed, and the position of the water table can realistically be presumed to be almost at the surface. A series of water tests, together with drilling water, usually so charges low-permeability foundations with water, that if the water table were not at the surface prior to testing it is soon raised to that position.

6. Conclusions

The routine method described for the interpretation of the "modified" lugeon water tests comprises:

—testing at five different pressures, with rising, then falling pressures
—calculation of five lugeon values by direct proportion
—inspection of the five values to detect the nature of the flow behaviour during the tests
—selection of an appropriate lugeon value to be the reported permeability for the stage under test.

The information obtained from the tests is used to decide when grouting is required, but is not used for determining the suitability of different types of grout.

Results from using the method on 811 cases at differing dam sites suggest predominantly laminar flow in the fine crack openings commonly encountered when the permeability is 1, 2 or 3 lugeons; when the permeability exceeds this, "turbulent" flow appears to predominate. This latter designation is also used to denote various proportions of turbulent flow and laminar flow in one stage.

There are an insignificant number of cases of temporary rock dilation or movement during testing. There is, however, a significant proportion of cases where the testing has caused permanent movement or washing-out of joint infill material.

7. References

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APPENDIX H

SITE GEOLOGY

TABLE OF CONTENTS

1.0 LANDFORM CLASSIFICATION UNITS

Bedrock. In place rock that is overlain by overburden material or exposed at the surface. The following modifiers will be used for all types of bedrock whether igneous, sedimentary or metamorphic. Wheathered, highly fractured or poorly consolidated bedrock should be indicated. Unweathered, bedrock should also be indicated.

Colluvial Deposits. Deposits of widely varying composition that have been moved downslope chiefly by gravity. Fluvial slopewash deposits are ususally intermixed with colluvial deposits.

Talus. Deposits of angular rubble and rock fragments accumulated by gravity at the base of cliffs and steep slopes.

Fluvial Deposits. Marterials deposited by running water, such as rivers and streams.

Floodplain. Deposits laid dow by a river or stream and flooded during periods of highest water in the present stream regimen. Floodplains are composed of two major types of alluvium: (1) Generally granular riverbed (lateral accrection) deposits laid down above the riverbed and deposits due to bank overflow (flood stages).

Old Terrace. An old, elevated floodplain surface no longer flooded. Occurs as horizontal benches above present floodplains.

Glacial Deposits. Deposits formed in direct contact with glacial ice. These heterogencous deposits are laid down by glacial ice and composed of materials varying from clay to boulders.

Ablation Till. Material is transported upon or within the glasier and is deposited during the downwasting of the glacier. As a consequence the material is loose, non-compact generally washed the clasts are less abraded.

Basal Till. Material deposited at the base of the glacier. The till is compact, non-sited, and the clasts are crushed and abraded. Generally deposited in sheets. Generally frozen with high silt and ground ice content.

Glaciofluvial Deposits. Coarse, granular deposits laid down by streams flowing on, in or from glaciers.

Outwash. Relatively level flood plain fromed a stream flowing frm a glacier.

Lacustrine. Fine grained lake deposits.

2.0 OVERBURDEN

The overburden in the vicinity of the "Fins" consists of a sequence of surficial deposits which include ice disintegration, outwash, lacustrine, till, and alluvial materials. A summary of the overburden stratigraphy is presented in Table H-1.

The overburden in the vicinity of th Fingerbuster - Powerhouse area has been found to consist of talus which is interlayed with alluvuim near the base of the valley slopes, adjacent to the river.

3.0 PLUTONIC ROCKS

At the Watana site, the bedrock is nearly continuously exposed in outcrops along the south bank between Elevations 1650 and 1900. On the north bank, outcrops are generally smaller and less frequent. The rock is primarily diorite and quartz diorite, with lesser amounts of monzonite and grandodiorite. Thses varied lithologies are probably the result of magnatic differentiation within the parent magma. A 20-foot-wide gradational contact between the diorite and quartz diorite is exposed at river level on the south bank approximately 1,000 feet upstream from the dam centerline. Contacts are found in boreholes BH-6 and BH-8 over 0.3 feet at Elevation 1594 and 3.8 feet at Elevation 1708, respectively.

The diorite is a crystalline igneous rock whic is predominatly medium greenish gray, but varies to light gray and light to medium greenish gray in the granodiorite and quartz diorite phases, respectively. The texture is massive with no foliation. Grain size varies from fine (less tham 1mm) to medium (1-5mm) but is generally medium grained. The diorite is generally composed of 60 to 80 percent feldspar, 0 to 10 percent quartz, and 20 to 30 percent mafics.

The feldspar consists primarily of medium grained, euhedral plagioclase with minor amounts of fine grained anhedral orthoclase. Quartz, when present, is fine grained and intergrown between the feldspar crystals. Mafic minerals, consisting of biotite and hornblende, are generally fine grained. The hornblende is often partically chloritized. Trace amounts of sulphides and carbonate also occur within the diorite. Inclusions of argillite have been observed in the diorite in "the Fins" and the "Fingerbuster" area.

The diorite is generally fresh and hard to very hard. The rock is slightly weathered along the joint surfaces ot depths of about 50 to 80 feet. There is generally a very hin (less than 2 inches) weathering rind on most outcrops.

The pluton has been intruded by both mafic and felsic dikes which are discussed below.

Zones of hydrothermal alteration occur within the diorite. The alteration has caused the chemical breakdown of the feldspars and mafic minerals. The feldspars have altered to kaolinite and the mafics have altered to chlorite. Hydrothermal alteration is discussed in more detail in a later section.

4.0 ANDESITE PORPHYRY

The name andesite porphyry is used for a varied group of apparently related extrusive rock types. The andesite porphyry occurs along the western side of the diorite pluton and is exposed in outcrops on both sides of the Susitna River. On the south bank, outcrops occur across from the "Fingerbuster" and at approximate Elevation 1750 immediately downstream from the dam centerline. andesite porphyry was drilled in boreholes BH-4, BH-8, and BH-2 to depths of 96.0, 43.0 and 103.0 feet, respectively. (see Exhibit 2) Borehole DH-28 bottomed at 125 feet in the porphyry Andesite porphyry dikes are also found intersprersed in the diorite. On the north bank, the andesite is exposed at river level in the "Fingerbuster" area and in scattered outcrops to about Elevation 2350.

The andesite porphyry is a light to medium, dark greenish gray volcanic rock similar in composition to the diorite pluton. The color becomes lighter with increasing amounts of lithic inclusions. The groundmass is aphanitic (grains visible only with the aid of a microscope) with generally 10 to 30 percent of fine to medium grained plagioclase feldspar phenocrysts. Lithic inclusions are found thorughout the andesite porphyry but are most concentrated near the contract with the diorite. Concentrations of subrounded to subangular frangments, up to 6 inches in diameter, of quartz diorite, argillite and volcanic rocks were found above the diorite contact in BH-8. The andesite porphyry is fresh to slightly weathered and hard. Hydrothermal alteration is not common in the andesite porphyry.

The andesite porphyry appears to contain layers or zones of dacite and latite. These varied rock types appear to be irregular and discontinuous in the site area and could not be mapped over large areas. The term andesite porphyry has been used as a general term for all of these volcanic units.

5.0 DIKES

The diorite pluton has been intruded by both mafic and felsic dikes. No dikes were deleniated i the andesite porphyry. Their small size precludes their delineation as a mappable unit.

Felesic dikes are found in outcrops and in boreholes. Felsic dikes are light gray and aphanitic to medium grained, but generally fine grained. The felsic dikes are composed primarily of feldspar (plagioclase and orthoclase) with up to 30 percent quartz and less than 10 percent mafics. Contacts with the diorite are tight and "welded". The felsic dikes are hard, fresh, and unfractured. Dike widths are up to 6 feet bu generally less than 0.5 feet. Felsic dikes have been found offset up to 16 inches by shears and healed shears in outcrop and in boreholes.

Mafic dikes are less common at the site than the felsic dikes. They are rarely seen in outcrop but were found locally in boreholes BH-1, BH-2, BH-8, and BH-12. The mafic dikes, consisting of andesite or diorite, are dark green to dark green gray. Grain size is aphanitic to very fine, with fine to medium grained plagioclase phenocrysts. The mafic dikes are hard and fresh with tight contacts. Dike widths are generally less than 1 foot, although in BH-2, an andesite dike was drille from 245.8 to 277.8 feet. Diorite inclusions were also found in this dike.

6.0 SHEARS, FRACTURE, AND ALTERATION ZONES

This section defines and discusses shears, fracture zones, and alteration zones.

6.1 Shear Zones

Shear are defined as a surface or zone of rock fracture along which there has been measurable displacement or is characterized by breccia, gouge, and/or slickensides indicating relative movement.

The primary type of shear found at the site is common to all rock types and consists of unhealed breccia and/or gouge. The breccia consists of coarse to fine sand-size rock fragments in a silt or clay matrix. The gouge is primarily slightly to moderately plastic. Both the breccia and gouge are soft and friable. Thicknesses of these shears vary from less than 0.1 inch up to 10 feet, but are generally less than 1 foot. Carbonate and chlorite mineralization are commonly associated with these shears. Some shears are partially to completely filled with carbonate. slickensides are found in may shear and occur on both the carbonate and chlorite surfaces. The shears are often associated with alteration zones.

Healed shears and breccias were found in virtually all boreholes. In all cases,these zones were found to be competent with high RQDs and high core recoveries. These features are interpreted to be emplacement type shears which formed during the last phases of plutonic activity.

6.2 Fracture Zones

Fracture zones are areas of very closely to closely spaced (less than 1 foot) jointed erock with no apparent relative movement: Fracture zones in outcrop were found to range form 6 inches up to 30 feet in width but are generally less than 10 feet wide. In the boreholes, fracture zones were found to range from less than 1 foot up to ore than 100 feet wide as measured in BH-2. However, for the most part in boreholes and outcrop, th fracture zones are less than 5 feet wide.

Where exposed, the fracture zones are easily eroded and form topographic lows or gullies, which have become filled with talus. The fracture surfaces are generally ironoxide stained. A coating of white carbonate is also commonly found on the fracture surface.

6.3 Alteration Zones

Alteration zones are areas where hydrothermal solutions have caused the chemical breakdown of the feldspars and mafic minerals. The common byproducts of alteration are kaolinite from feldspar, and chlorite from mafic minerals. These zones are found in both the diorite and andesite porphyry, but appear to be less common in the adesite porphyry.

Most of the information regarding alteration zones is form the boreholes. The degree of alteration is highly variable ranging form slight, where the feldspars show discoloration, to complete where

the feldspars and mafics are completely altered to clay and chlorite. In slightly altered diorite, the rock is bleached to a yellowish-green or whitish-gray and is generally hard to moderately hard as seen in BH-3 from 933.2 to 948.9 feet. The slightly altered zones have approximately 10 to 25 percent of the feldspars stained or altered to clay. In completely altered diorite, the rock is bleached to whitish gray or very light yellowish gray. The rock fabric is preserved; however, the material is soft and friable. The completely altered zones are less common, and when encountered, are generally 1 to 2 feet wide. Most alteration zones found in the boreholes are slightly to moderately altered. These zones are moderately hard with some thin soft zones.

Widths of these alteration zones range up to 10 feet but are generally under 5 feet. an exception is in BH-12 on the south bank which drilled over 300 feet into an alteration zone GF 8. Several shear/alteration zones are exposed in "The Fins" and range up to 10 feet wide.

The carbonate, which is also associated with the alteration zones, occurs as veins or joint filing generally up to 0.5 inch thick. Occasionally, sulphide mineralizatin and iron oxide stainingare also found inthese zones.

No increase in joint frequencyis evident in association withthese alteration zones. Numerous thin (less than 2 inches) shears are associated with the alteration zones. RQDs are generally low, because only fresh to slightly altered rock i.e. competent rock, is considered in RQD measurements. Core recovery is generally more than 90 percent within the alteration zones. The transition from fresh to altered rock is gradational, generally occuring over less than 1 foot.

TABLE H-1 WATANA RELICT CHANNEL STRATIGRAPHY – GENERALIZED DESCRIPTION OF PROPERTIES[1]

GRAPHIC SYMBOL	UNIT	TYPE OF DEPOSIT	DESCRIPTION	GEOLOGICAL AND ENGINEERING REMARKS [2]	UNIFIED SOIL CLASSIFICATION [3]	THICKNESS RANGE IN FEET [4]
	A/B	Surficial Deposit	Organics, peat, silt and boulders raised by frost action.	Organic mat which includes localized boulder fields and bogs. Within the active layer/seasonal frost penetration zone.	OL, PT, SM	0.5 - 6.0
	C	Ice Disintegration	Grey brown, gravelly sand to silty sand with little to some gravel and cobbles. Coarse fraction subangular to subrounded.	Hummocky, knob and kettle topography. Variable density Permafrost detected in 1 out of 16 possible borings. No groundwater detected.	SM, SC	2.5 - 38.0
	M	Basal Till	Grey to dark grey silty sand to clay with little angular to subrounded gravel and cobbles, occasional boulder. Very dense, hard. Poorly sorted.	Gravel and cobbles are striated. Limited in areal extent to near the Susitna River Valley. Similar to unit G', it is overconsolidated. Permafrost detected in 1 out of 4 possible borings. No groundwater detected.	SM, SC, CL	14.5 - 79.0
	D	Alluvium	Grey stratified sand, gravel and cobbles. Very dense.	Localized fluvial event, reworking of the underlying outwash, unit E/F, found in topographic lows on top of outwash. No permafrost or groundwater detected.	SM, SP, SC	1.5 - 55.0
	D'	Lacustrine	Grey to dark greyish brown, laminated clayey silt to clayey silty sand. Very dense, hard. Sorted to partly sorted.	Thin laminated deposit, limited in areal extent. Permafrost detected in 1 out of 4 possible borings. No groundwater detected.	ML, CL, (SC, SM)	3.5 - 23.0
	E/F	Outwash	Olive brown to grey brown, silty sand with little gravel and cobbles to a silty sandy gravel with occasional cobbles and boulders. Coarse fraction subangular to subrounded. Dense to very dense. Poorly sorted.	In places the unit gets coarser with depth, higher energy environment. Thick continuous deposit. Density is loose to medium dense in active frost zone, up to 15 feet deep. Permafrost detected in 3 out of 31 borings. Groundwater detected in 4 out of 15 possible borings.	SM, GM, SC	10.0 - 131.0
	G	Lacustrine and/or Waterlain Till	Dark grey to olive grey, laminated, sandy silt to silty clay, little or no gravel, little to some sand. Very dense. Poorly sorted.	Thin clay, silt and sand interlaminations. Organics and wood present. Overconsolidated. Permafrost detected in 2 out of 17 possible borings. No groundwater was detected. Together with unit G', forms a prominent marker bed.	ML, CL, SM	8.3 - 73.5
	G'	Basal Till	Olive grey to very dark grey, clayey silty sand with trace to little gravel to gravelly silty or clayey sand. Coarse fraction subangular to subrounded and includes occasional cobbles and boulders. Very dense. Poorly sorted.	Gravels and cobbles are striated and polished. Overconsolidated. Permafrost detected in 1 out of 15 possible borings. Groundwater was detected in 1 out of 9 possible borings. Forms a marker bed with unit G.	SM, SC, (ML, CL, GC)	7.0 - 231.0
	H	Alluvium	Grey brown to olive grey, silty sand and sand with little or no gravel to sandy gravel. Coarse fraction subangular to rounded, slightly oxidized. Very dense. Sorted to partly sorted.	Rounded particles, sorted, relatively clean lenses or layers, possibly stratified. Localized fluvial event, reworking of the underlying outwash, found in topographic lows of unit I. Groundwater detected in 4 out of 6 possible borings. No permafrost detected.	SM, SP, GW-GM	2.0 - 41.0
	I	Outwash	Olive grey, silty sand with little gravel to sandy gravel with little fines. Coarse fraction subangular to subrounded trace rounded; some cobbles, particles oxidized. Very dense. Poorly sorted.	Oxidation on particles, indicative of age and weathering. Organics found in the upper horizon. Trace striations on gravel. Thick nearly continuous deposit. Groundwater detected in 3 out of 6 possible borings. No permafrost detected.	SM, GW-GM, SW, GM, (CL, ML)	6.0 - 77.0
	J'	Lacustrine and/or Stratified Deposits	Olive grey to olive brown, silty sand, trace subangular gravel with some sandy gravel (?). Oxidized and weathered particles, some cobbles and boulders(?). Very dense. Sorted to partly sorted.	Moderately oxidized and weathered, generally sorted, possibly stratified. Overconsolidated. Localized deposit. No permafrost or ground water detected. Mud loss of 50 gals/ft over 25 foot interval in DR-22.	SM, SW, SC	3.0 - 57.7
	J	Basal Till	Olive grey to dark grey, clay to clayey sand little to no subangular to subrounded oxidized(?) gravel. Dense, very hard. Poorly sorted.	Gravels are striated and polished. Overconsolidated. Probable lacustrine or waterlain till at base of unit. No permafrost or groundwater detected.	CL, SM, SC	6.0 - 62.0
	K	Alluvium	Olive grey, silty sandy gravel to sandy gravel with cobbles and boulders (?) Coarse fraction subangulars to rounded, oxidized. Very dense. Sorted.	Rounded particles, sorted, relatively clean. Found only along the main thalweg to date. No permafrost or groundwater detected. Mud loss of 14 gal/ft over an 85 foot interval in DR-22.	GM, GP, GW	36.0 - 161.0

[1] Modified After Acres American, Inc., 1982

[2] Remarks on permafrost are based on Acres Summer 1982 and Harza-Ebasco Winter 1983 Exploration. Remarks on groundwater are based on the 1983 Winter Exploration.

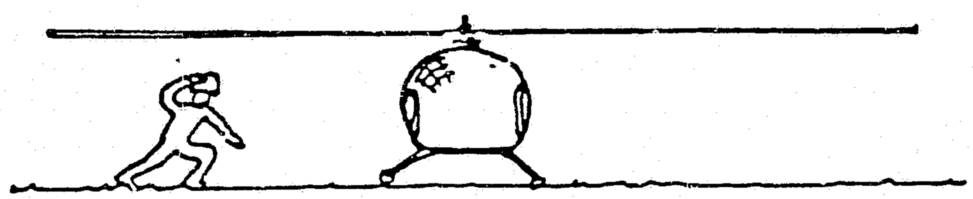
[3] Classification is based on the primary soils types in decreasing order of occurence. Those in parentheses are key secondary types.

[4] Thickness ranges are based on outcrop exposures and drilled thicknesses.

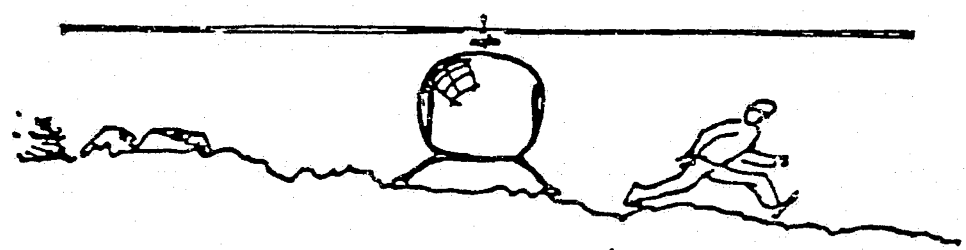
HARZA-EBASCO
SUSITNA JOINT VENTURE

APPENDIX I

HELIOCOPTER SAFETY MANUAL

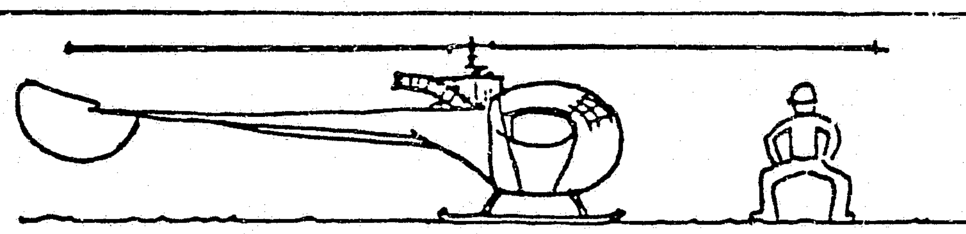


Always wear a hat when approaching or leaving a helicopter. If no chin strap is used, hold on to the hat.



Approach or leave on the downslope side (to avoid rotor).

Never walk up slope from helicopter.



Do not touch bubble or any moving parts (tail rotor, exposed linkage, etc.).

SMOKING
AT PILOT'S
DISCRETION

NO SMOKING IN THE CABIN ON TAKE OFF OR LANDING.

SMOKING IN FLIGHT WITH PILOT'S PERMISSION ONLY.

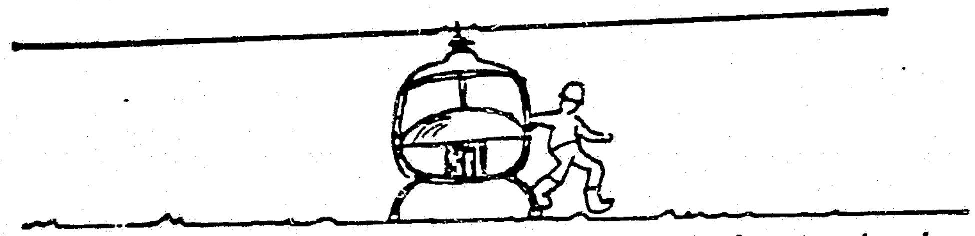
ASH TRAYS ARE INSTALLED IN ALL HELICOPTERS. USE THEM.

NO SMOKING IN THE LANDING AREA.

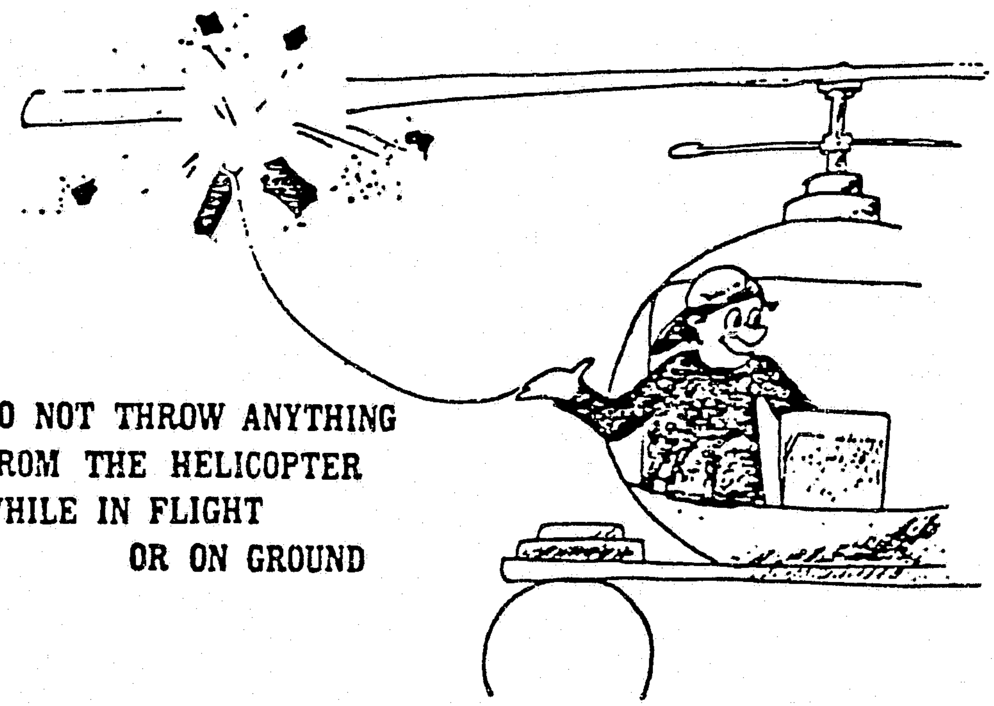
Fasten seat belt upon entering helicopter and leave it buckled until pilot signals you to get out.

Remain in seats and position in helicopter.

DO NOT SLAM THE HELICOPTER DOORS

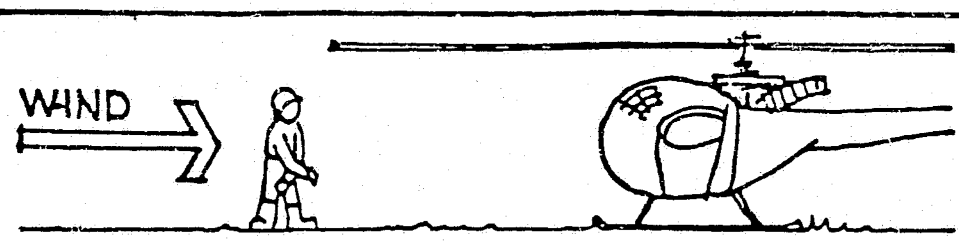


Never leave the helicopter while it is in a hover. Get out and off in one smooth, unhurried motion.



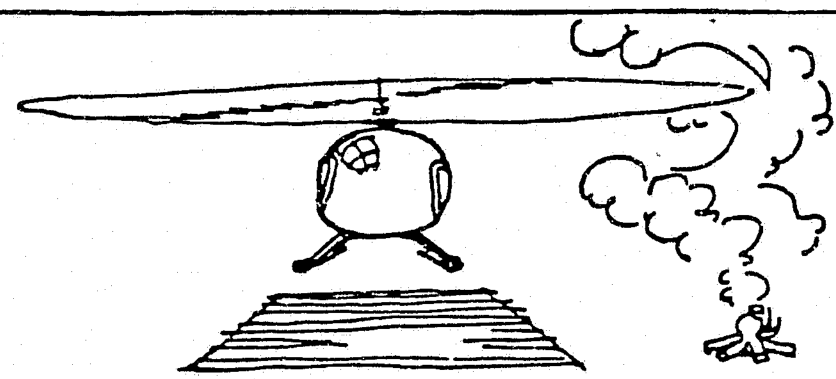
DO NOT THROW ANYTHING
FROM THE HELICOPTER
WHILE IN FLIGHT
OR ON GROUND

KEEP THOSE FINGERS OFF THE BUBBLE

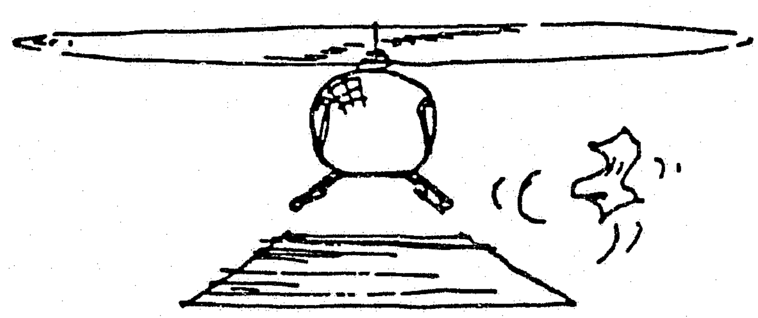


When directing landing, stand with arms crossed in front of body and point downward with back to wind.

All wires, ropes, aerials should be well marked and never erected near landing area or approaches to the landing area.



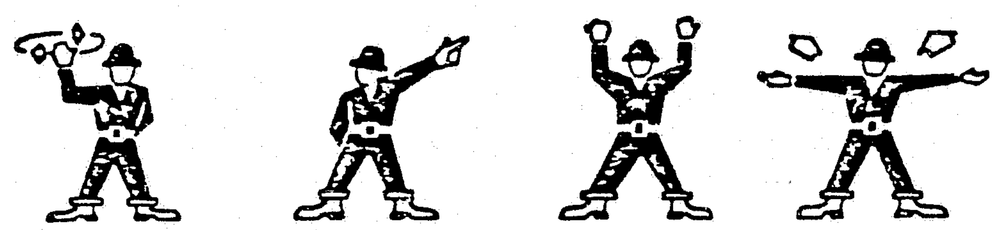
Keep cooking and heating fires well clear of helispot. This area is unsafe when the helicopter is landing.



Keep heliport clear of loose articles (water bags, empty cans, etc.). Keep personnel away during take-offs and landings.

HELICOPTER HAND SIGNALS

THESE SIGNALS ARE ADVISORY AND THE PILOT IS UNDER NO OBLIGATION TO OBEY THEM. CONDITIONS BEYOND THE CONTROL OF THE PILOT OR FACTORS UNKNOWN TO THE GROUND SIGNALMAN MAY MAKE IT NECESSARY OR ADVISABLE TO DISREGARD THE SIGNALS. WHEN THESE SIGNALS ARE USED IT IS IMPORTANT THAT THE SIGNALMAN POSITION HIMSELF BEYOND THE PATH OF THE MAIN ROTOR WHERE HE MAY BE READILY OBSERVED BY THE PILOT.

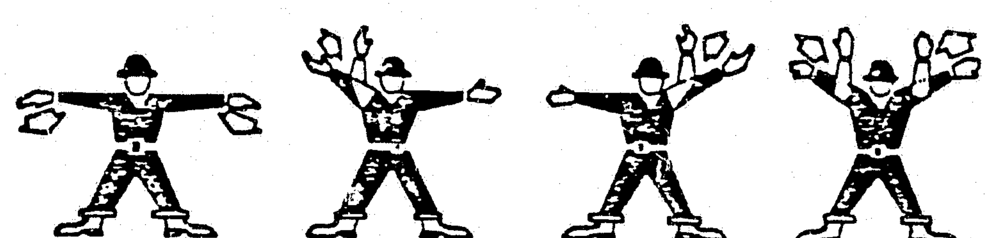


CLEAR TO START ENGINE

TAKEOFF RIGHT HAND BEHIND BACK LEFT HAND POINTING UP

HOLD-HOVER PLACE ARMS OVER HEAD WITH CLENCHED FISTS

MOVE UPWARD ARMS EXTENDED, SWEEPING UP



MOVE DOWNWARD ARMS EXTENDED, PALMS DOWN, ARMS SWEEPING DOWN

MOVE RIGHT LEFT ARM HORIZONTAL RIGHT ARM SWEEPS UPWARD TO POSITION OVER HEAD

MOVE LEFT RIGHT ARM HORIZONTAL LEFT ARM SWEEPS UPWARD TO POSITION OVER HEAD

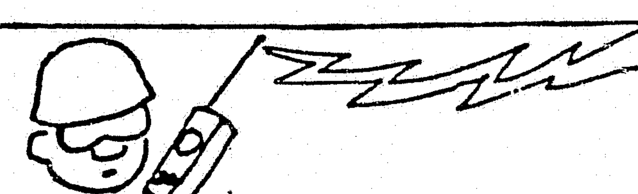
MOVE FORWARD COMBINATION OF ARM AND HAND MOVEMENT IN A COLLECTING MOTION PULLING TOWARD BODY

MOVE REARWARD HANDS ABOVE ARM, PALMS OUT USING A SHOVING MOTION

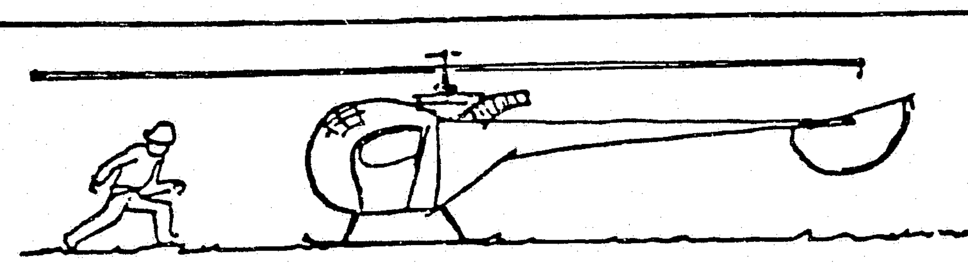
RELEASE SLING LOAD LEFT ARM DOWN AWAY FROM BODY. RIGHT ARM CUTS ACROSS LEFT ARM IN A SLASHING MOVEMENT FROM ABOVE

LAND ARMS CROSSED IN FRONT OF BODY AND POINTING DOWNWARD WITH BACK TO WIND

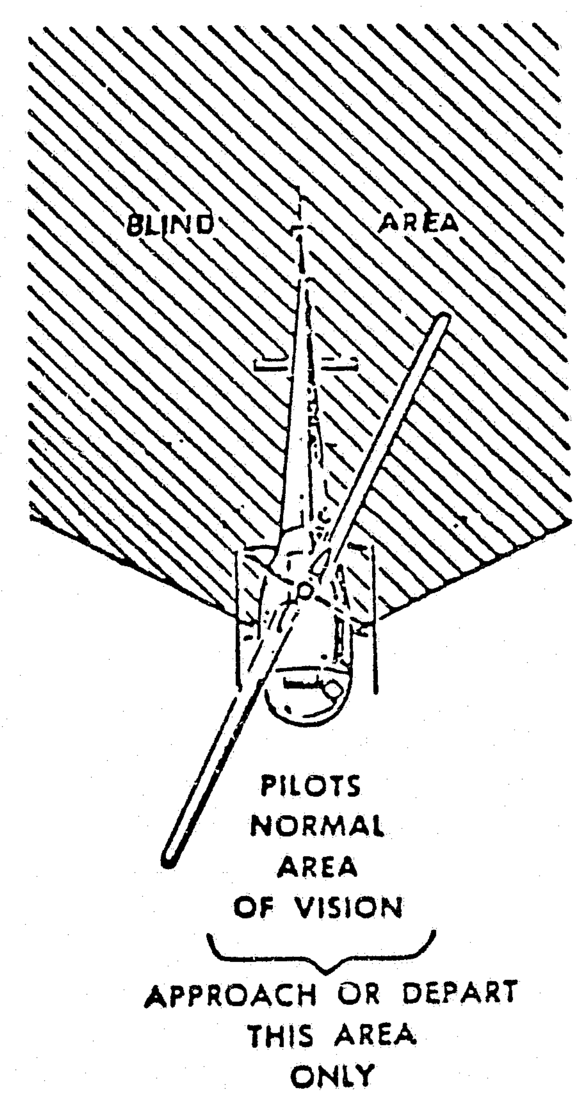
SHUT OFF ENGINE SLASH ACROSS THROAT

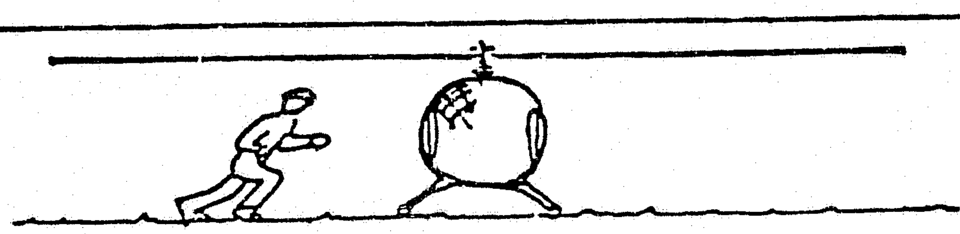


When directing pilot by radio, give no landing instructions that require acknowledgement as pilot will have both hands busy



Approach or leave in pilot's field of vision (to avoid the rotor).





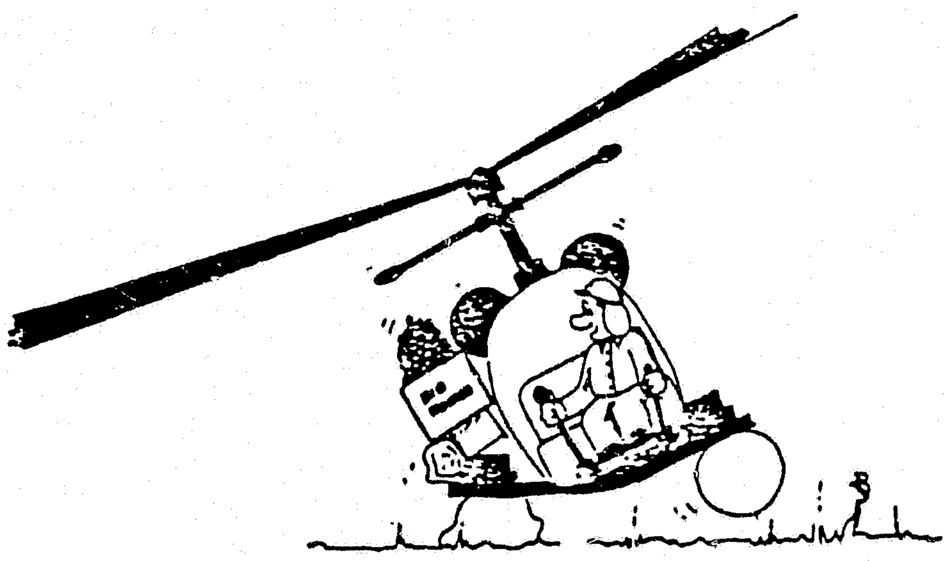
Approach or leave machine in a crouching manner (for clearance from main rotor).

Blade tips may come within 5 feet of level ground.

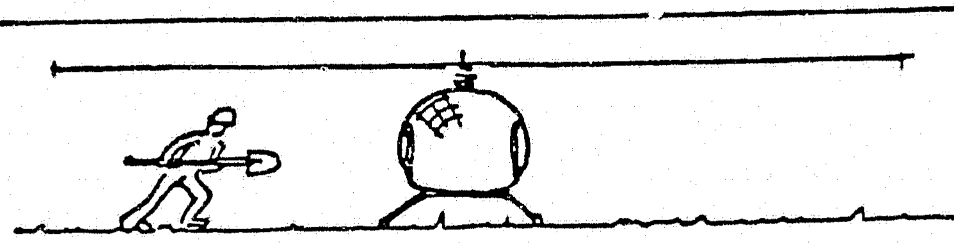
LOADING AND UNLOADING

Cargo should be loaded carefully and not thrown, dropped or jammed in cabin. Load and secure as the pilot instructs. Unload next to helicopter. Never throw anything away from 'copter. Never carry anything on your shoulder or over your head when near the helicopter. Loaded firearms are not allowed on board the helicopter. Weight down light equipment and loose articles, then leave landing area before take-off.

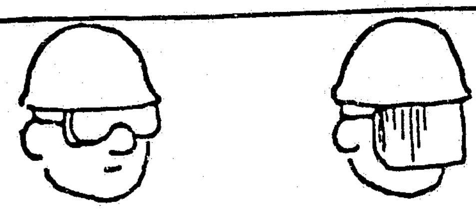
DO NOT SLAM THE HELICOPTER DOORS



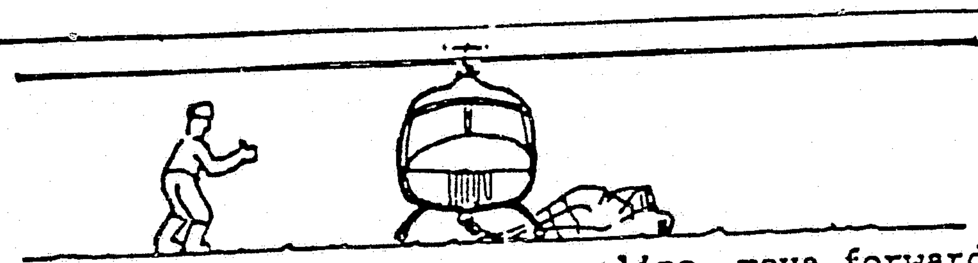
ALWAYS LOAD THE HELICOPTER EVENLY



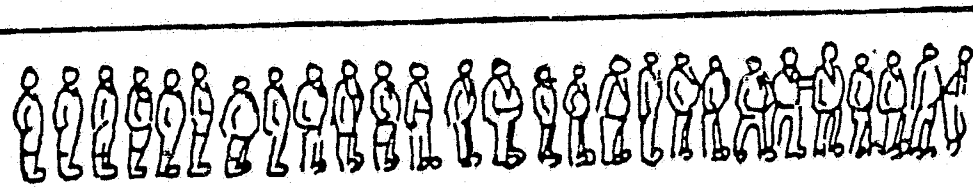
Carry tools horizontally, below waist level (never upright or over shoulder).



Loading personnel should always wear eye protectors.



After hooking up cargo sling, move forward and to the side to signal pilot (to avoid entanglement and getting struck with loaded sling).



When moving larger crews:

(a) Brief them on safety as above.
(b) Keep them together and well back at side of landing zone (this gives the pilot a chance in the event he has to land suddenly during either takeoff or landing.
(c) Have them face away from the machine during takeoff and landing.
(d) Have each man look after his own personal gear.
(e) Have men organized for loading and ready to board on signal.