RAY TABER FOUNDATION
ANNUAL
DRILL SEMINAR

Drill Class Manual

Drilling and Sampling
CPT
Soil/Rock Logging

TABLE OF CONTENTS

Ray Taber Foundation 3

Safety 6

Reference Websites 16

Practical Considerations 18

Drilling Techniques 21

Tools / Specifications 33

Logging Techniques 37

Sampling Techniques 57

Hammer Energy Measurements 67

In-Situ Testing – CPT 73

In-Situ Testing – DMT 95

In-Situ Testing – FVST 129

Instrumentation – Inclinometers 134

Instrumentation – Time Domain Reflectometry 141

APPENDIX - Army Corps Publication “Drilling in Earth Embankment Dams & Levees” 157

FERC – “Guidelines for Drilling In and Near Embankment Dams and Their Foundations” 174

Ray Taber Foundation

H. R. Taber

Ray Taber Foundation

RAY TABER FOUNDATION

Purpose: To pursue the creation of a not-for-profit California education-focused foundation that advances the two passions of Ray Taber, the science of combining geology and civil engineering and sustainable small ranch farming. The primary purpose of the foundation would be to serve as a vehicle for providing educational scholarships to students in these specific areas of study, but also as support for specific research endeavors and other activities which will aid the general advancement of these areas of specialization.

Background: H. Ray Taber (1927-2011) was a pioneer consultant in the field of the application of professional geologic opinion to civil engineering design which is known as the practice of "engineering geology", now a subset of Geotechnical Engineering. He also came from a pioneer ranching family that arrived in California in the 1850's and settled in the Capay Valley. He and his family have continuously practiced sustainable small ranch farming on this same land since 1867.

Career Highlights: Ray Taber and partner Ret Moore literally "wrote the book" on bridge foundation investigation protocols while with the Bridge Department of the State of California (1950's), he was a founding member of AEG an international organization for engineering geologists (1960's), he developed a risk-based geologic hazard assessment system for evaluation of potential construction sites (1970's) and he helped develop a roller-plate super-structure-to-foundation connection to mitigate fault movement between bridge supports (1980's). In the ranching community he has made contributions to the development of a new almond variety called the "Merlin" and to the development of a sustainable water supply and delivery system model for a small ranch setting.

Mission: To advance the science of integrating geology and civil engineering in their application to the built world and to contribute to the historical preservation and advancement of economically sustainable small ranch farming.

Initiatives:
1. Provide scholarships to students pursuing studies in the areas of our mission.
2. Provide support for specific research endeavors which will advance our mission.
3. Pursue other collaborative activities within the scientific and farming communities which both align with and effectively advance our mission.

Income Sources:
1. Corporate Donations
2. Private Party Donations
3. Professional Organization Donations
4. Professional Organization Grants
5. Government Grants
6. Private Foundation Grants
7. Stakeholder Support Share from Participation in Mission-Related Special Events
8. Stakeholder Support Share from Participation in Mission-Related On-Going Product or Services Sales Activities

Ray Taber Foundation

RAY TABER FOUNDATION - - BOARD OF DIRECTORS

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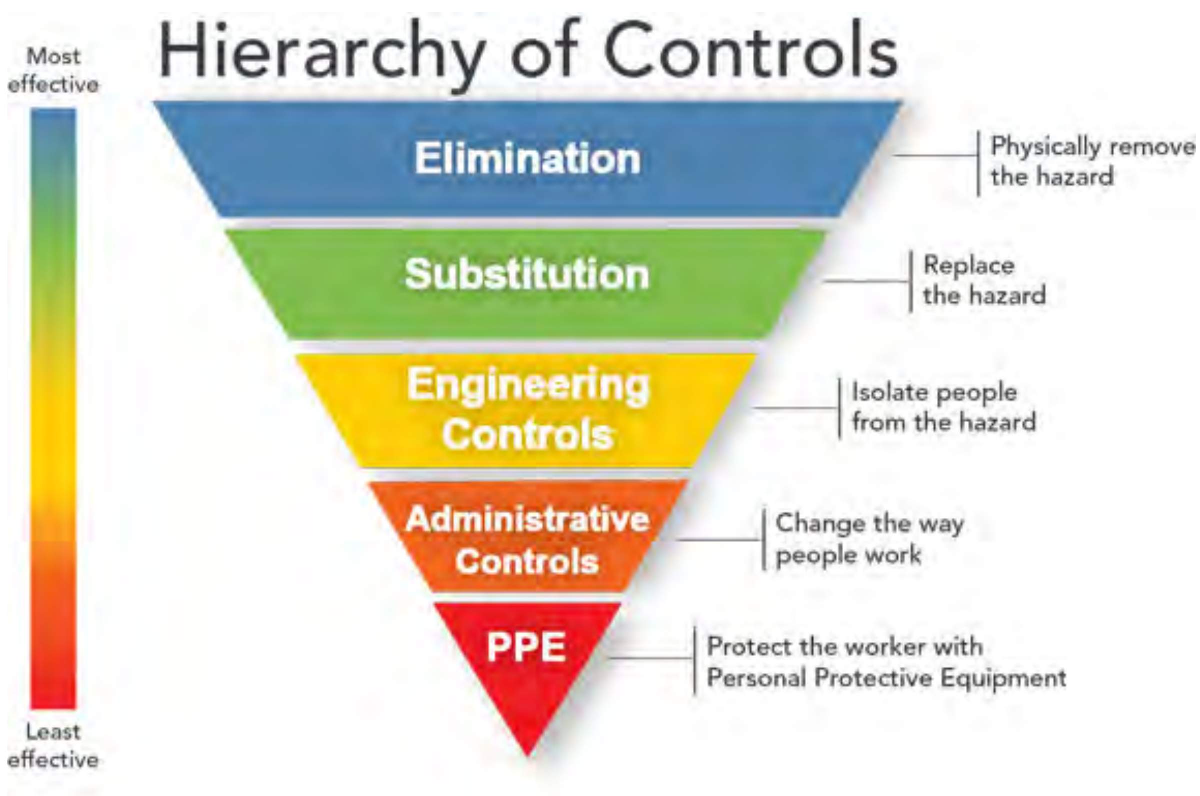
Safety

Safety Brief

Safety plays an integral part geotechnical field and should always be the number one priority. As one becomes indoctrinated into this field, you will be required to follow, institute, and teach safety measures. Whether it be compiling a Job Safety Analysis for the site, speaking at tailgate safety meeting, or comprising a Site Health and Safety Plan for the project, the following pages serve as a reference for safety tools, protocols, and best practices for safety in geotechnical work.

Please note that the following safety examples are for reference only and may not meet the minimum safety requirements as required by specific client standards or whatever jurisdiction you may be in.

The below graph is a reference to be used to mitigate safety hazards and/or use as a method for determining what Risk Control Measures to use.



Job Safety Analysis Worksheet

Company name:		Date:	JSA No.
Site name:		Permit to work requirement:	Yes ☐ No ☐
Contractor:		Approved by:	
Activity:			

Activity List the tasks required to perform the activity in the sequence they are carried out.	**Hazards** Against each task list the hazards that could cause injury when the task is performed.	**Risk control measures** List the control measures required to eliminate or minimise the risk of injury arising from the identified hazard.	**Who is responsible?** Write the name of the person responsible (supervisor or above) to implement the control measure identified.

Remember: Each JSA must be site specific.
Include all workers in the development of this JSA.

WorkSafe VICTORIA

WorkSafe Victoria is a division of the Victorian WorkCover Authority

USA North

UNDERGROUND SERVICE ALERT
*** Call Before You Dig ***
1-800-227-2600

UNDERGROUND SERVICE ALERT is a free, convenient information and notification service supported by utilities like PG&E, SMUD, AT&T, Sprint, etc. When you place a toll-free call to USA, an operator will ask you where you plan to dig and will contact all the appropriate utilities, who will mark any underground installations they have at the excavation site.

Information to have ready:

- When the work begins (date & time)
- Location of markings
- What the area is marked with
- Are you using explosives?
- Will you use vacuum equipment?

The Color of Safety

These colors and symbols have been adopted by all subscribing to the UNDERGROUND SERVICE (USA). Look for painted stripes or markers before you begin to dig.

Color	Type	Symbol	Facility
Blue	Water	W	Water
Orange	Communications	FA	Fire Alarm
		Tel	Telephone
		R	Railroad
		TV	Television
		WU	Western Union
Green	Sewer/Storm	S	Sewer
	Drain	D	Storm Drain
Red	Electric	L	Street Lightning
		E	Electric
		T	Traffic Signals
Yellow	Oil/Gas/Steam/	G	Gas
	Chemical	Co. Name	Oil & Chemical
Pink			Survey Marking
White			Proposed Excavation

Activity Hazard Analysis (AHA)

Activity/Work Task: Mobilization	Overall Risk Assessment Code (RAC) (Use highest code)	L

Risk Assessment Code (RAC) Matrix

Severity	Probability				
	Frequent	Likely	Occasional	Seldom	Unlikely
Catastrophic	E	E	H	H	M
Critical	E	H	H	M	L
Marginal	H	M	M	L	L
Negligible	M	L	L	L	L

Step 1: Review each **"Hazard"** with identified safety **"Controls"** and determine RAC (See above)

	RAC Chart
"Probability" is the likelihood to cause an incident, near miss, or accident and identified as: Frequent, Likely, Occasional, Seldom or Unlikely.	
"Severity" is the outcome/degree if an incident, near miss, or accident did occur and identified as: Catastrophic, Critical, Marginal, or Negligible	**E = Extremely High Risk** **H = High Risk**
Step 2: Identify the RAC (Probability/Severity) as E, H, M, or L for each "Hazard" on AHA. Annotate the overall highest RAC at the top of AHA.	**M = Moderate Risk** **L = Low Risk**

Job Steps	Hazards	Controls	RAC
Mobilization from home base to the job site	1. Highway accidents 2. Equipment shift during travel 3. Exposed to traffic hazards	1. Defensive driving 2. Secure equipment during transport 3. Wear high visibility apparel	L L L

Equipment to be Used	Training Requirements/Competent or Qualified Personnel name(s)	Inspection Requirements
1. Trucks, trailers and track-mounted equipment 2. Tie-downs, straps and ropes	1. All employees are required to be licensed by the State of California 2. Trained on DOT requirements 3. Wear seatbelt	1. DOT requirements 2. Conduct walk around vehicle inspection

Activity Hazard Analysis (AHA)

Activity/Work Task: Job Hazard Analysis and Assessment	Overall Risk Assessment Code (RAC) (Use highest code)	L

Risk Assessment Code (RAC) Matrix

Severity	Probability				
	Frequent	Likely	Occasional	Seldom	Unlikely
Catastrophic	E	E	H	H	M
Critical	E	H	H	M	L
Marginal	H	M	M	L	L
Negligible	M	L	L	L	L

Step 1: Review each **"Hazard"** with identified safety **"Controls"** and determine RAC (See above)

	RAC Chart
"Probability" is the likelihood to cause an incident, near miss, or accident and identified as: Frequent, Likely, Occasional, Seldom or Unlikely.	E = Extremely High Risk H = High Risk
"Severity" is the outcome/degree if an incident, near miss, or accident did occur and identified as: Catastrophic, Critical, Marginal, or Negligible	
Step 2: Identify the RAC (Probability/Severity) as E, H, M, or L for each "Hazard" on AHA. Annotate the overall highest RAC at the top of AHA.	M = Moderate Risk L = Low Risk

Job Steps	Hazards	Controls	RAC
1. Conduct Job Hazard Analysis to each specific site to determine possible hazard (e.g. Chemical or Biological) 2. Continually conduct safety observations and Job Hazards Assessment before, during, and after all on site operations	1. Encountering contaminates while drilling 2. Biological (e.g. poisonous snakes and spiders) 3.Thermal stress 4. Spills and leaks	1. Stop drilling immediately to assess situation and notify the on-site Site Safety Officer 2. Work area will be inspected prior to starting operations 3. Wear proper clothing for the weather conditions. Keep hydrated. All employees shall have an "Oasis" to get out of the heat per OSHA regulations. The Site Safety Officer will implement the Heat Illness Prevention Plan. 4. Wear hard hat, steel toe boots, safety glasses, hearing protection & disposable plastic gloves as needed. Use spill kit when appropriate which includes absorbent towels, absorbent pads, drip pans, buckets, and shovel	L L L L

Equipment to be Used	Training Requirements/Competent or Qualified Personnel name(s)	Inspection Requirements
1. P.P.E. as deemed necessary and applicable to the job site by the Site Safety Officer	1. All people shall be notified of standards that are applicable OSHA requirements 2. All crews shall be aware of possible Biological Hazards prior to the beginning of operations	1. Work area will be inspected daily prior to operations

Activity Hazard Analysis (AHA)

Activity/Work Task: Site-Setup	Overall Risk Assessment Code (RAC) (Use highest code)	L

Risk Assessment Code (RAC) Matrix

Severity	Probability				
	Frequent	Likely	Occasional	Seldom	Unlikely
Catastrophic	E	E	H	H	M
Critical	E	H	H	M	L
Marginal	H	M	M	L	L
Negligible	M	L	L	L	L

Step 1: Review each **"Hazard"** with identified safety **"Controls"** and determine RAC (See above)

	RAC Chart
"Probability" is the likelihood to cause an incident, near miss, or accident and identified as: Frequent, Likely, Occasional, Seldom or Unlikely.	E = Extremely High Risk
"Severity" is the outcome/degree if an incident, near miss, or accident did occur and identified as: Catastrophic, Critical, Marginal, or Negligible	H = High Risk
Step 2: Identify the RAC (Probability/Severity) as E, H, M, or L for each "Hazard" on AHA. Annotate the overall highest RAC at the top of AHA.	M = Moderate Risk L = Low Risk

Job Steps	Hazards	Controls	RAC
1. On-Site preparation 2. Equipment test 3. Utility clearance	1. Tip over 2. Electrocution / explosion 3. Slips, trips, falls 4. Struck by	1. Set up on level surfaces using all leveling jacks 2. Inspect for buried and overhead utilities. Do not break ground without clearance permits 3. Wear proper P.P.E, such as hard hat and steel toe boots 4. Clear traffic areas 5. Use proper lifting techniques 6. Tools shall properly be secured during transport 7. Secure footing	L L L L L

Equipment to be Used	Training Requirements/Competent or Qualified Personnel name(s)	Inspection Requirements
1. Trucks, trailers, and track-mounted equipment 2. Tie-downs, straps, and ropes 3. Mechanical winches	1. On the job training and in house training 2. Trained on DOT requirements 3. All crews shall wear high visibility apparel / PPE, at a minimum meeting ANSI / ISEA 07-2004 performance class 2 requirements	1. DOT requirements 2. Conduct walk around vehicle inspection 3. Check for hydraulic leaks and wear tear on cables

Activity Hazard Analysis (AHA)

Activity/Work Task: Traffic Control	Overall Risk Assessment Code (RAC) (Use highest code)	L

Risk Assessment Code (RAC) Matrix

Severity	Probability				
	Frequent	Likely	Occasional	Seldom	Unlikely
Catastrophic	E	E	H	H	M
Critical	E	H	H	M	L
Marginal	H	M	M	L	L
Negligible	M	L	L	L	L

Step 1: Review each **"Hazard"** with identified safety **"Controls"** and determine RAC (See above)

	RAC Chart
"Probability" is the likelihood to cause an incident, near miss, or accident and identified as: Frequent, Likely, Occasional, Seldom or Unlikely.	
"Severity" is the outcome/degree if an incident, near miss, or accident did occur and identified as: Catastrophic, Critical, Marginal, or Negligible	E = Extremely High Risk H = High Risk
Step 2: Identify the RAC (Probability/Severity) as E, H, M, or L for each "Hazard" on AHA. Annotate the overall highest RAC at the top of AHA.	M = Moderate Risk L = Low Risk

Job Steps	Hazards	Controls	RAC
Provide reasonably safe and effective movement of road uses through or around Temporary Traffic Control (TTC) zones while reasonably protecting road users, workers, and responders to traffic incidents and equipment.	1. Workers not visible to drivers 2. Workers unfamiliar with specific traffic control requirements 3. Inadequate TTC for advance warning of work zones	1. All workers shall wear high visibility safety apparel 2. The SSO shall be 8-hr Traffic Control and Flagging trained and shall brief employees every morning prior to work 3. Work zones will be established using barricades, traffic cones, and/or caution tape as appropriate to alert passing traffic to workers in or near roadway	L L L

Equipment to be Used	Training Requirements/Competent or Qualified Personnel name(s)	Inspection Requirements
1. High visibility class 3 PPE 2. Traffic control lights, barricades, road markings, signs, and signalpersons for the safe movement in accordance with DOT Federal Highway Administration's "MUTCD".	1. The SSO shall be 8-hr Traffic Control Flagging trained	1. Conduct daily tailgate meetings prior to the start of work to ensure all workers are wearing Class 3 high visibility PPE and are aware and understand all traffic control measure AHAs for the site.

Activity Hazard Analysis (AHA)

Activity/Work Task: Drilling Procedures	Overall Risk Assessment Code (RAC) (Use highest code)	L

Risk Assessment Code (RAC) Matrix

Severity	Probability				
	Frequent	Likely	Occasional	Seldom	Unlikely
Catastrophic	E	E	H	H	M
Critical	E	H	H	M	L
Marginal	H	M	M	L	L
Negligible	M	L	L	L	L

Step 1: Review each **"Hazard"** with identified safety **"Controls"** and determine RAC (See above)

	RAC Chart
"Probability" is the likelihood to cause an incident, near miss, or accident and identified as: Frequent, Likely, Occasional, Seldom or Unlikely.	
"Severity" is the outcome/degree if an incident, near miss, or accident did occur and identified as: Catastrophic, Critical, Marginal, or Negligible	**E = Extremely High Risk** **H = High Risk**
Step 2: Identify the RAC (Probability/Severity) as E, H, M, or L for each "Hazard" on AHA. Annotate the overall highest RAC at the top of AHA.	**M = Moderate Risk** **L = Low Risk**

Job Steps	Hazards	Controls	RAC
1. Operating drilling equipment	1. Struck by 2. Pinch and grab points (loose clothing, hands and feet catching in the drill head) 3. Loud noise from drilling equipment above 90 db (not expected) 4. Ergonomic hazards	1. Make sure all swivels are firm before lifting. Make sure to have proper footing before swinging hanging objects 2. Always be aware when drill head is spinning and stay out of grabbing range. Make sure to always keep hands and clothing out of pinch points. Loose gloves and long shoe-laces are not to be used near drill operations 3. PPE -- wear hard hat, safety glasses, steel toe boots and hearing protection as needed 4. Always use proper lifting, standing, and sitting techniques while on the job	L L L L

	Training Requirements/Competent or Qualified Personnel name(s)	Inspection Requirements
1. Mechanical winches 2. Hydraulic down feeds	1. Vehicles are to meet DOT regulations 2. On the job training and in house training 3. All crews shall wear high visibility apparel / PPE, at a minimum meeting ANSI / ISEA 07-2004 performance class 2 requirements	1. Check for hydraulic leaks before daily operations

Activity Hazard Analysis (AHA)

Activity/Work Task: Maintenance	Overall Risk Assessment Code (RAC) (Use highest code)	L

Risk Assessment Code (RAC) Matrix

Severity	Probability				
	Frequent	Likely	Occasional	Seldom	Unlikely
Catastrophic	E	E	H	H	M
Critical	E	H	H	M	L
Marginal	H	M	M	L	L
Negligible	M	L	L	L	L

Step 1: Review each **"Hazard"** with identified safety **"Controls"** and determine RAC (See above)

	RAC Chart
"Probability" is the likelihood to cause an incident, near miss, or accident and identified as: Frequent, Likely, Occasional, Seldom or	
"Severity" is the outcome/degree if an incident, near miss, or accident did occur and identified as: Catastrophic, Critical, Marginal,	**E = Extremely High Risk** **H = High Risk**
Step 2: Identify the RAC (Probability/Severity) as E, H, M, or L for each "Hazard" on AHA. Annotate the overall highest RAC at the	**M = Moderate Risk** **L = Low Risk**

Job Steps	Hazards	Controls	RAC
1. All mechanical and hydraulic motors are to be maintained	1. Equipment malfunction 2. Fire 3. Hydraulic blow out	1. All equipment must be maintained and in a proper functioning condition 2. All motors must be shut off when making repairs and key removed. Additionally, an out of service placard is placed over the ignition 3. All motors must be shut off during refueling. An A-B-C fire extinguisher must be maintained near all excavation motorized equipment 4. Hydraulic pump and lines are not to exceed operating pressures. Lines must be secured to excavation equipment	L L L L

Equipment to be Used	Training Requirements / Competent or Qualified Personnel name(s)	Inspection Requirements
1. Excavation equipment 2. Fire extinguishers 3. Hydraulic pump	1. On the job training and in house training 2. Inspected and replace per OSHA regulations	1. Check for fluid leaks before daily operations 2. Extinguishers are inspected and replaced every year 3. Check for fluid leaks before daily operations

Reference Websites

WEBSITES FOR REFERENCE

Geoprofessional Business Association
(ASFE)
http://www.geoprofessional.org

American Society of Civil Engineers
(ASCE)
http://www.asce.org

California GeoProfessionals Association
(Cal Geo)
http://www.cgea.org

Geological Society of America
(GSA)
http://www.geosociety.org

Association of Engineering and Environmental
Geologists (AEG)
http://aegweb.org

American Society for Testing and Materials
(ASTM)
http://www.astm.org

California Department of Transportation
(Cal Trans)
http://dot.ca.gov

Practical Considerations

PRACTICAL CONSIDERATIONS

1. SAFETY – "SEE IT, SAY IT" – EVERYONE

2. FIELD RELATIONSHIPS ARE KEY
   (SCHLOSSER SYNDROME)

3. PLAYERS HAVE ROLES - RESPECT THEM
   - PRODUCTIVITY
   - SAFETY

4. CONTRACTOR SELECTION IS LIKE RIG SELECTION
   - STUDY GOALS
   - FIELD CONDITIONS ABOVE GROUND
   - FIELD CONDITIONS BELOW GROUND
   - $ BUDGET
   - AVAILABILIY
   - ALSO "HISTORY" REPUTATION)

5. TYPES OF CONTRACTS
   - BY FOOT (SPEED)
   - BY HOUR (CONTROL OF WORK)
   - LUMP SUM (CONTROL OF $)

WHICH RIG?

STUDY GOALS

FIELD CONDITIONS (ABOVE & BELOW)

$ BUDGET

AVAILABILITY

Drilling Techniques

Introduction to
Solid Flight Auger and Mud Rotary Techniques

Why are we drilling holes in the ground?

To assess subsurface conditions while keeping in mind specific aspects of your project.

What types of conditions/characteristics do you look for?

- Foundation Conditions
- Liquefaction potential
- Settlement potential
- Weak layers
- Very Dense or hard layers
- Groundwater
- Boulders, cobbles, Rock??
- Topography
- Geology

What method of drilling do you use to make your hole and get the samples You need?

- Solid Flight Auger
- Mud Rotary with Drag Bit, Tri-cone Bit
- Casing Advancement / Wire-line methods
- Diamond Coring
    - Types of tools for various methods
    - Lengths of Tools
    - Can your tools fit inside the casing?

What is that thing on the back of the truck?

KNOW the Rig.

- Where's the hammer and what type is it (Cathead & Rope or automatic)?
- Where's the kelly? The kelly swivel??
- Where's the mud tank?
- Where's the BEAN 35?
- Where's the gimble joint?
- Where's the drill rods?
- The rig isn't dangerous is it??

Introduction to
Solid Flight Auger and Mud Rotary Techniques

Continued…

What do you look for while drilling? What is important?

- How difficult is it to drill?
- Is there any chatter while drilling? I.e. are you drilling through gravels?
- Do you have any hole problems? Is the hole tight? Does it cave in or bridge when you pull the auger out of the hole? Is your drill fluid cleaning the hole properly?
- If using rotary methods… Are you getting good circulation? If not, how much mud are you losing?
- What do the cuttings look like?
- How deep is the groundwater?

How do you know you have exhausted a method of drilling? Where do you go from there?

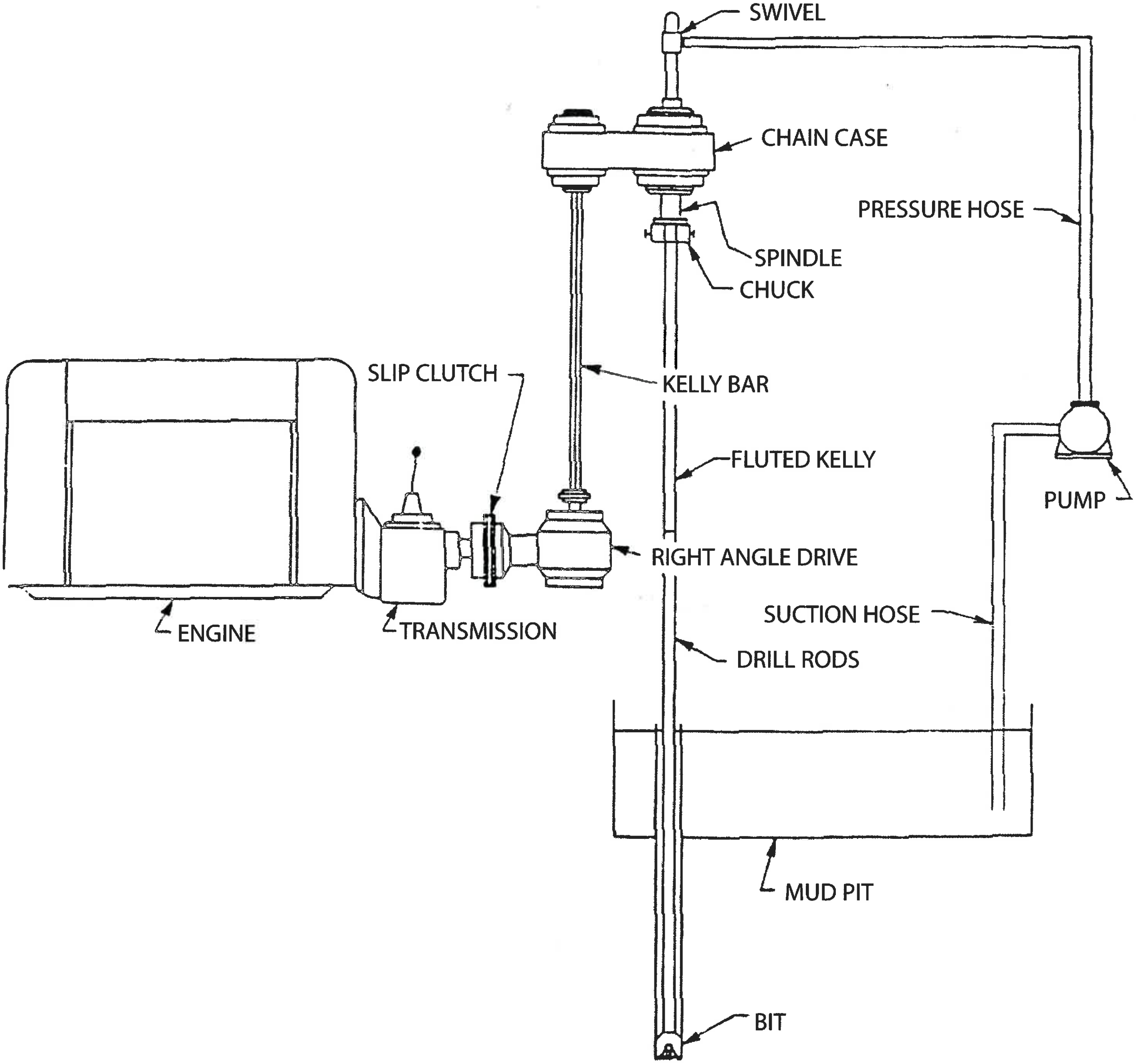
- You are not advancing the hole…
- Caving problems are preventing progress…
- Loss of Circulation (especially in sands and gravels)

Try another method or use additional tools to advance the hole.

- Casing Advancement/Wire-line methods
- Insert more casing in the hole
- Air Drilling
- Diamond Coring

Problems with Mud Rotary Drilling

- Groundwater measurements
- Set-up time
- Removal or disposal of cuttings
- Site Restoration/Clean-up
- Wet cuttings

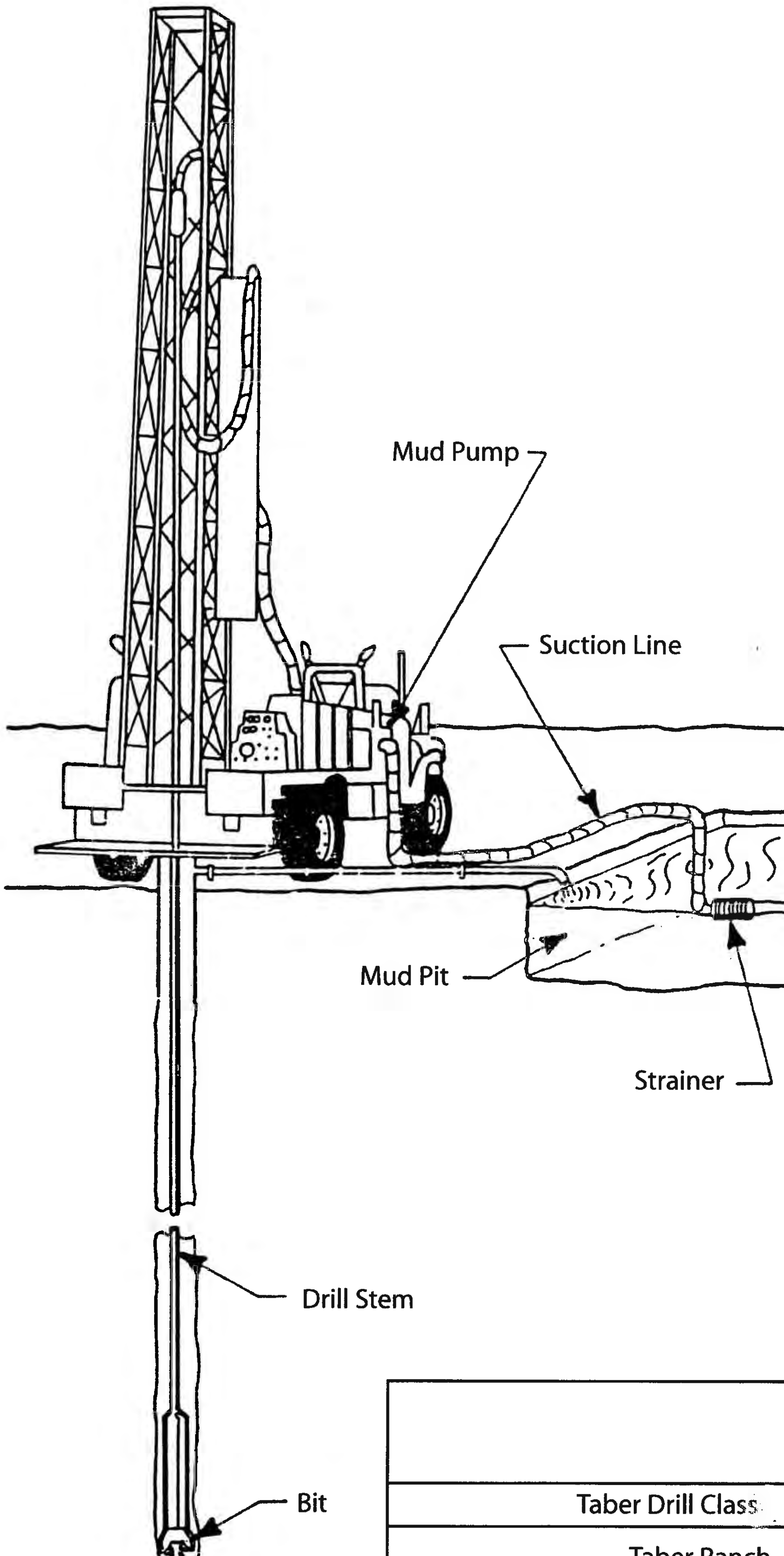


Taber

Taber Drill Class

Taber Ranch
Capay Valley, California

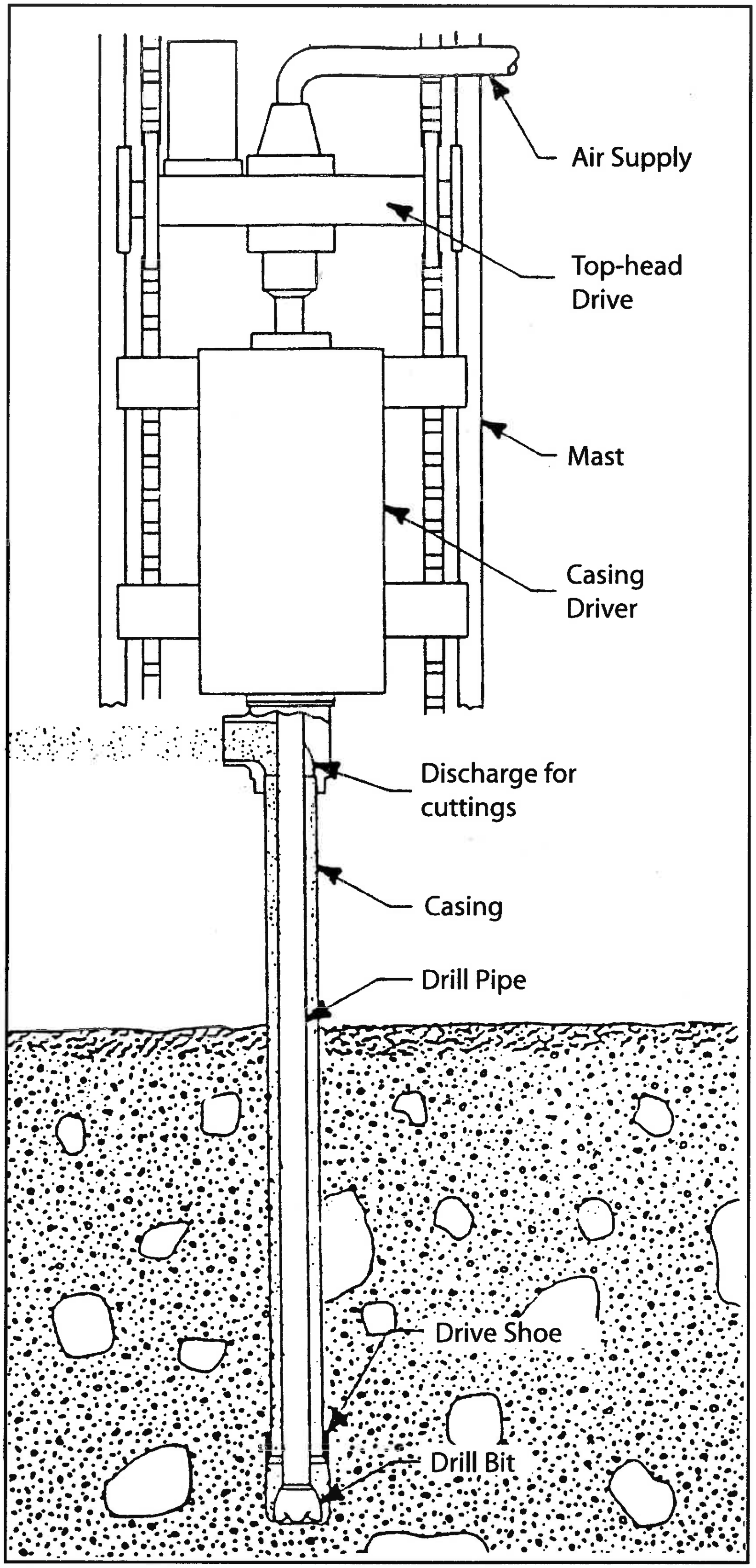
Typical Drill Rig Drive Train



Taber Drill Class

Taber Ranch
Capay Valley, California

Fluid Circulating System for Mud Rotary Drilling



Casing drivers can be fitted to top-head drive rotary rigs to simultaneously drill and drive casing.

Introduction to
Hollow-Stem Flight Auger Techniques

Differences Between Solid-Stem And Hollow-Stem Flight Auger Drilling

- What Are The Parts Of The Hollow-Stem Auger System?
- How Does It Work?
- How Deep Can You Drill?
- How Is Sampling Different?
- How Is Logging Different?
- What Are Common Reasons For Using Hollow-Stem Auger?
- What Is A Bad Reason For Using Hollow-Stem Auger?

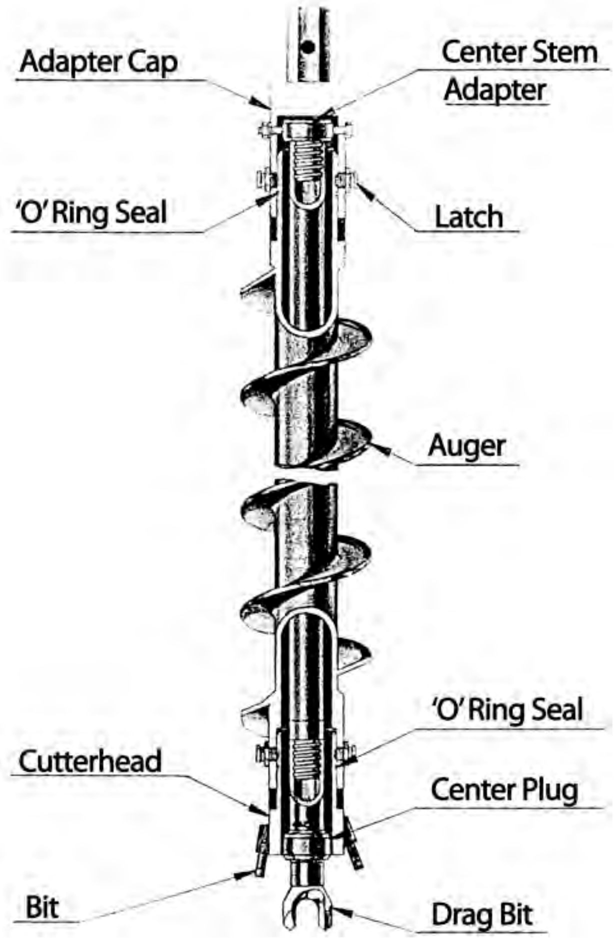
Problems With Hollow Stem Auger Drilling

- Groundwater Measurements
- Too Many Cuttings
- Not Enough Cuttings
- Effects on Sample

Tricks of the trade

- Know Why You Are Here
- Involve Your Driller
- Get The Data Now
- Interpret The Data Now
- Not Enough Sample
- Too Much Sample
- Retaining Sample; Preserving The Sample

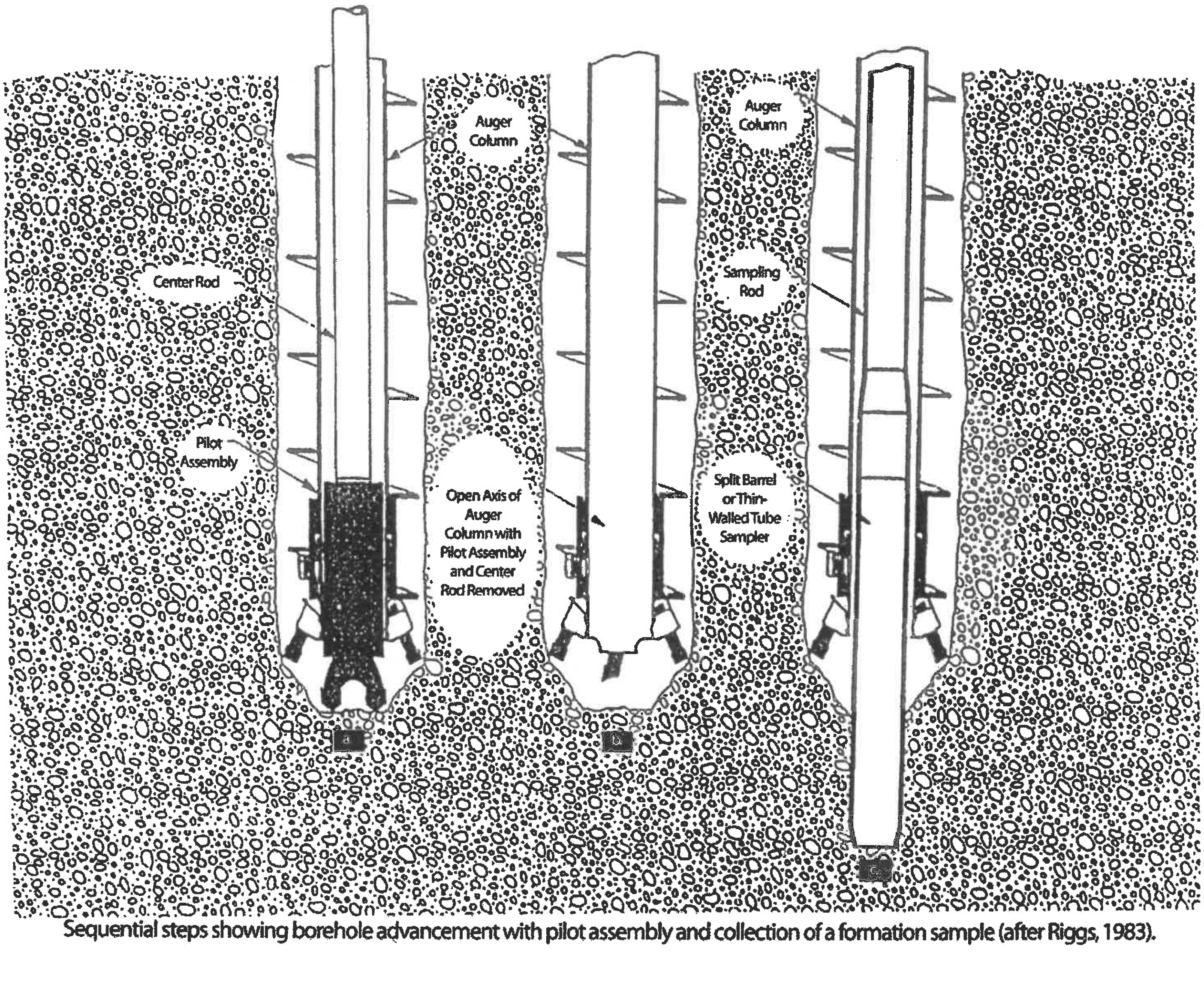
Hollow Stem Auger



Auger Size Chart (inches)

Hole Size	Flight OD	Auger Axle ID	Sample Tools	Core Barrels
6 ¼	5	2 ¼	2	AWG
6 ¾	5 ¾	2 ¾	2 ½	BWG
7 ¼	6 ¼	3 ¼	3	NWG
13 ¼	12	6	Denison	4x5 ½ CBLS

Hollow Stem Auger and Sampling



Sequential steps showing borehole advancement with pilot assembly and collection of a formation sample (after Riggs, 1983).

SONIC Drilling

Method of Operation of the Sonic Drill

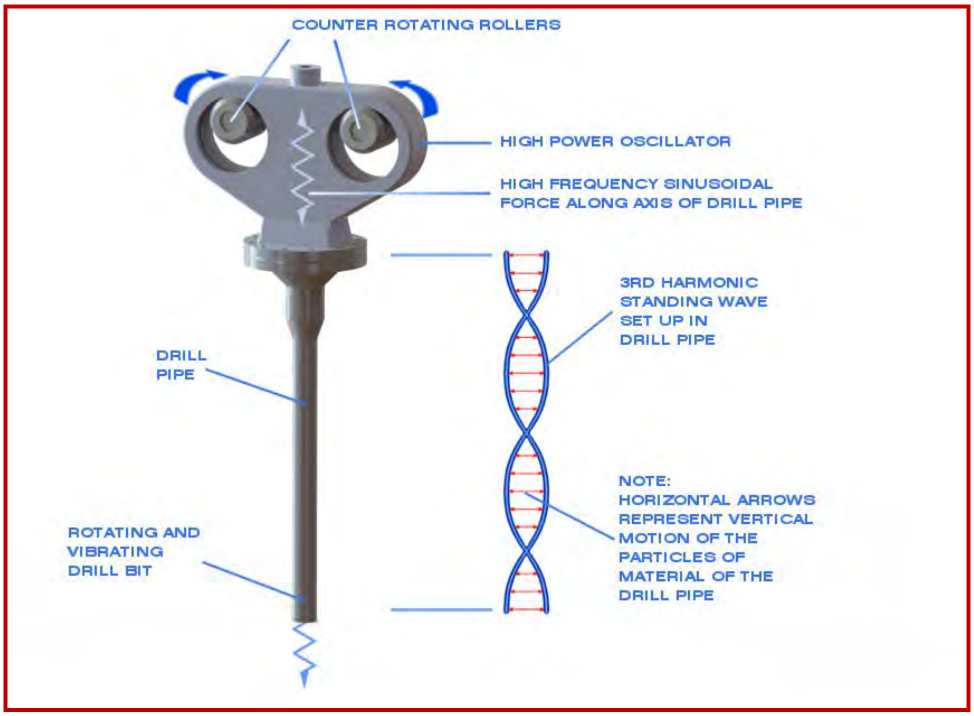
A sonic drill is more precisely identified as a rotary-vibratory drill. It is capable of high drilling speeds as well as accomplishing tasks, such as continuous coring.

At first glance, a sonic drill rig looks very much like a conventional air or mud rotary drill rig. The biggest difference is in the drill head, which is slightly larger than a standard rotary head. The head contains the mechanism necessary for rotary motion, as well as an oscillator, which causes a high frequency force to be superimposed on the drill string. The drill bit is physically vibrating up and down in addition to being pushed down and rotated. These three combined forces allow drilling to proceed rapidly through most geological formations including most types of rock.

In overburden, the vibratory action causes the surrounding soil particles to liquefy, thereby allowing effortless penetration. In rock, the drill bit causes fractures at the rock face, creating rock dust and small rock particles, which facilitates advancement of the drill bit. In many instances the drilling and coring of rock and earth can be accomplished without the use of any drilling fluid whatsoever. This is an important requirement for environmental drilling projects. Compressed air, drill mud, or plain water can be utilized to remove the cuttings and speed up the operation further, depending on the application that the machine is used for.

The oscillator is driven by a hydraulic motor and uses out of balance weights to generate high sinusoidal forces that are transmitted to the drill bit. An air spring is also incorporated in order to confine the alternating forces to the drill string. The frequency can be varied to suit operating conditions and is generally between 50 and 120 hertz (cycles per second). As a comparison, ordinary household current in many countries alternates at 60 hertz. This frequency range falls within the lower range of sound vibrations that the human ear is capable of hearing. Thus the term ‘sonic drill’ has been applied to this class of rotary-vibratory drilling machine.

While the principle behind the sonic drill appears complicated, the machine is actually very simple to operate. The driller only adds vibratory energy to the normal rotary motion. He simply chooses a frequency that gives him the best drilling rate or best core recovery, as the case may be.



Tools / Specifications

TECHNICAL PAGE

Depths/elevations of various tools and volumes/quantities of various materials must be calculated during and after the drilling. Therefore, equipment specifications should be measured and recorded prior to their use. Information such as the following should be included, if/as applicable.

Drill Rig Specifications
Drill Rig Manufacturer
Drill Rig Model
Drill Method (e.g. auger, rotary, air rotary, etc.)

Drill Crew
Lead Driller
Helper(s)

Hammer Specifications
Lift Type (e.g. cathead, winch, automatic)
Drop Weight
Drop Height

Auger Specifications
Auger type (e.g. hollow auger, solid flight-auger, etc.)
OD
ID (hollow auger only)
Flight Length
Auger Bit Length

Drill Rod Specifications
Designation (e.g. A, AWJ, etc.)
OD
ID
Section Length

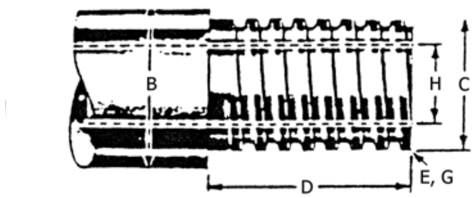
Drill Bit Specifications
Drill Bit Type (e.g. side-discharge rotary, tricone, etc.)
Drill Bit Material (e.g. tungsten-carbide, etc.)
Designation (e.g. NX, BX, etc.)
OD
Drill Bit Length

Drill Casing Specifications
Designation (e.g. NX, BX, etc.)
OD
ID
Section Length
Shoe Length

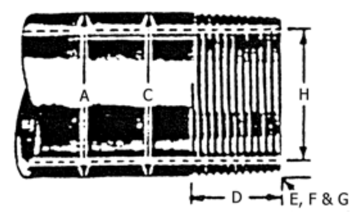
Sampler Specifications
Sampler Type (e.g. Standard Penetration, etc.)
OD
ID
Liner (type, length)
Length

Level Survey Data
Survey Benchmarks
Location of Benchmark and datum
Equipment used
Survey Personnel

DRILL ROD THREAD INFORMATION



FLUSH JOINT STRAIGHT THREAD

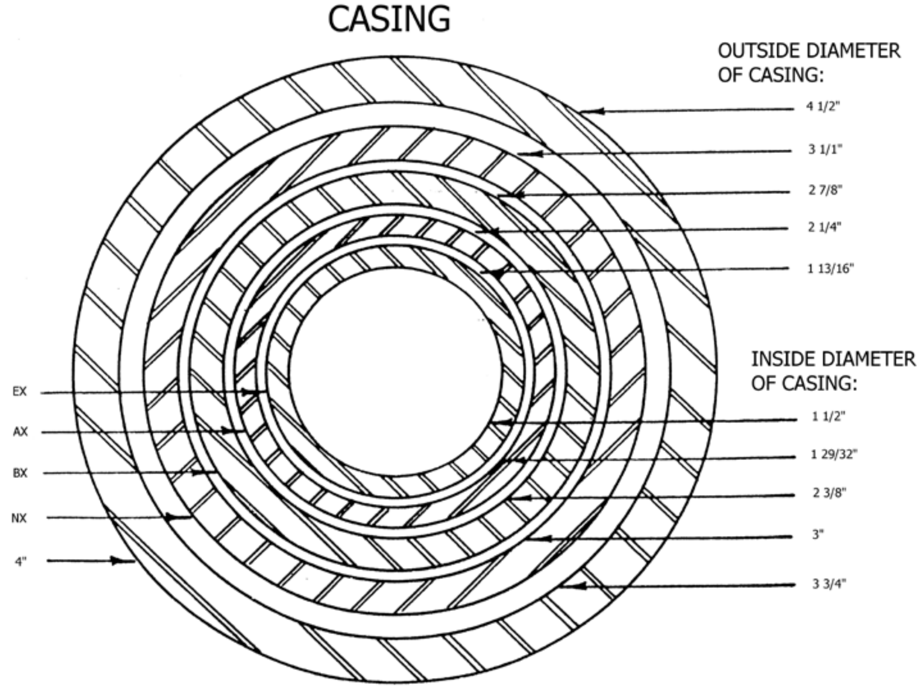


LINE PIPE

Name	A	B	C	D	E	F	G	H
FLUSH JOINT STRAIGHT THREAD[3]								
E-Rod[4]	1 5/16	1 5/16	1	1 1/2	3		Sq. Thd.	7/16
EW-Rod[*4]	1 3/8	1 3/8	1 1/16	1 9/16	3		Sq. Thd.	7/16
A-Rod[4]	1 5/8	1 5/8	1 17/64	1 17/64	3		Sq. Thd.	9/16
AW-Rod[4]	1 3/4	1 3/4	1 3/8	1 3/8	3		Sq. Thd.	5/8
B-Rod[4]	1 29/32	1 29/32	1 13/32	1 13/32	5		Sq. Thd.	5/8
BWRod[*4]	2 1/8	2 1/8	1 11/16	1 11/16	3		Sq. Thd.	3/4
N-Rod[4]	2 3/8	2 3/8	1 7/8	1 7/8	4		Sq. Thd.	1
NW-Rod[*4]	2 5/8	2 5/8	2 7/32	2 7/32	3		Sq. Thd.	1 3/8
Carey Modified 3-Thd.	2 3/8	2 3/8	2 1/16	2 1/16	3		Sq. Thd.	1 1/2
N-Rod Failing Type	2 3/8	2 3/8	1 7/8	1 7/8	3		Sq. Thd.	1 1/8
Petty Geophysical	1 1/2	1 1/2	1 1/4	1 1/4	6		Sq. Thd.	3/4

[3] Pipe diameter same as joint diameter.
[4] American Diamond Core Drill Standards.
[*] American Standards Show 5 Modified Thread.

CASING



AUGER AND SAMPLING EQUIPMENT

SOLID STEM AUGER

4″ O.D., 6″ O.D., 8″ O.D., 10″ O.D., 12″ O.D., 16″ O.D.

HOLLOW STEM AUGER

3 1/4″	I.D.	X	7″	O.D.
4 1/4″	I.D.	X	8″	O.D
5 5/8″	I.D.	X	11″	O.D
8 1/4″	I.D.	X	13″	O.D

DRIVE SAMPLING EQUIPMENT

1 3/8″	I.D.	X	2″	O.D.	Split Spoon (SPT)
2″	I.D.	X	2 1/2″	O.D.	Split Spoon (California)
2 1/2″	I.D.	X	3″	O.D.	Split Spoon (Modified California)

MOSS SYSTEM: CONTINUOUS CORE SAMPLING

2 1/2″	I.D.	X	3″	O.D. X 5′ L	Split Spoon

SHELBY TUBES

2″ O.D.
3″ O.D.
4″ O.D.

BOART LONGYEAR CORE BARREL SYSTEM SIZES

SIZE	CORE SAMPLE Ø	HOLE Ø
NQ3™	45 mm (1-3/8 in)	75.7 mm (3 in)
HQ™	63.5 mm (2-1/2 in)	96 mm (3-3/8 in)
HQ3™	61.1 mm (2-3/8 in)	96 mm (3-3/8 in)
PQ™	85 mm (3-3/8 in)	122.6 mm (4-7/8 in)

Logging Techniques

TYPE: * SURFACE ELEVATION: *

PID (ppm)	* (tsf)	OTHER TESTS	DRY DENSITY (pcf)	Moisture (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (inches)	SAMPLE No.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION SYSTEM	
pid	*ucs*	*other*	*dd*	*mc*	*blows*	*size*	*no*	##		*usc*	*description*

well description

--CONTINUED--

THE BORING LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOGGED BY: * DATE: *

	BORING LOG	Project No.
	Client	*Proj #*
	Proj Name Loc, CA	*Date*
		Boring *

INTRODUCTION

Q: What is a "Log"?

- A log is a record of processes, measurements, events, materials, and characteristics considered relevant and important to the objective of the project.

  A "Geotechnical Boring Log" records such as information as soil description/characteristics, field tests, groundwater depth, and drilling methods.

Q: What is the Purpose of "Logging"?

- The purpose of logging is to accurately and legibly record all pertinent data and information for future interpretation, reference, or use.

- The purpose of a "Geotechnical Boring Log" is to provide information for use in geotechnical analysis, earthwork, monitor wells, construction, etc.

Q: What is the Importance of Thorough "Logging"?

- A log containing a <u>complete</u> and <u>legible</u> record provides the most useful information for interpretation, reference, and use.

- Missing or inaccurate information can lead to inaccurate interpretation and poor performance from the facility being designed.

- If you don't record information in the field, it's gone forever! And, it costs a lot to go back out and obtain it a second time.

Soil Classification and Description

(Reference ASTM D 2487 and ASTM D 2488)

The "Unified Soil Classification System" (USCS) is a standardized system for classifying soils based on laboratory testing, and is specified by ASTM D 2487. Description and identification of the USCS soil classification by "Visual-Manual" methods is covered by ASTM D 2488. Identification of the USCS classification in the field, as a result of conditions and time constraints, frequently must be estimated based on experience and other subjective means, guided by the "Visual-Manual" method. When soil samples are obtained, laboratory testing may be performed to confirm field classification. Once laboratory testing is complete, the final soil classifications can be made in accordance with ASTM D2487.

Soil Classification

- Grain Size
  - Sieve (Gravel and Sand gradation plus total "fines" – Lab or Field estimate)
  - Hydrometer (Silt and Clay gradation – Lab only)
  - Visual-Manual (Visual, Dry Strength, Dilatancy, etc. – Field or Lab)
  - Estimation (Visual, Tactile, etc. – Field, verify in lab)
- Plasticity
  - Atterberg Limits (Liquid Limit and Plasticity Index – Lab only)
  - Visual Manual (Dilatancy, Toughness, etc. – Field or Lab)
  - Estimation (Visual, Tactile, etc. – Field, verify in lab)

Soil Description is information supplemental to soil classification. Many standardized systems for soil description have been developed (e.g. Visual-Manual method) but most are relatively subjective. In all cases, soil desctipions should be systematic and complete, and preferably referenced to some standard methodology (refer to firm or regional practice).

Soil Description

- Consistency (Fine-Grained Soil Only)
  - Thumb Penetration (per Visual-Manual procedure)
  - Torvane Shear (field or lab)
  - Pocket Penetrometer (field or lab)
  - Relative Density (Coarse-Grained soils only)
  - Blow Count ("Standard Penetration Test," ASTM D 1586)
  - Estimation
- Color (by Color Chart or Estimation)
- Moisture Condition
- Range of Particle Sizes
- Angularity of Particles (Coarse sand and larger)
- Shape of Particles (Gravel and larger)
- Other (Odor, HCl Reaction, Cementation, Structure, Oxidation etc.)

Typical field classifications and description might appear as follows:

Dense brown silty fine to medium sand with fine gravel, SM, moist.

or, Soft dark gray to black silty clay with peat stringers, CL, wet, slight organic odor, very slight dilatency, moderate dry strength.

Soil Layer Breaks

Where the soil classification or description changes in a boring, either within a drilling interval or sampling interval, the depth/location and type of transition or "break" between layers should be noted in the boring log. Accuracy of the break location is desirable to within 6 inches between samples and within 1 inch within a sampler. If the break is noted within a recovered sample or the depth can otherwise be accurately determined, its location is indicated as definite, and commonly shown on the graphic log as a solid line between layers. All other breaks are noted as approximate and commonly shown on the graphic log using a dashed line.

Two types of layer breaks are typical of geotechnical test boring logs:

Unconformable Layer Break represents a discontinuity in the succession of rock or soil layers, containing a gap in the geologic record. Examples of unconformable layer breaks are fill overlying alluvium or alluvium overlying bedrock. On the graphic log an unconformable layer break might be shown as a wavy line (~) similar to an unconformity on a geologic structure section.

Conformable Layer Break represents continuity in the succession of rock or soil layers without a gap in the geologic recorded. An example of a conformable layer breaks is an alluvial sequence of sand, silt, and clay layers.

The transition of gradation, color, and other properties between adjacent soil layers can be slow or abrupt. There are various schemes for describing the distinctness of the transition between layers, but none can be considered "typical practice." Commonly, contacts are described as "gradational" where the soil classification and/or description changes slowly over some un-described distance. Otherwise, the change is commonly un-described but assumed to be distinct.

Fill Materials

Definition of "FILL" Any material below the ground surface that is not present due to natural processes.

- Material placed to raise the level of an area
- Material placed to support a structure or act as a barrier
- Material placed to fill an excavation
- Material placed for disposal of excess material
- Material placed to drain groundwater or seepage around or under man-made structures of facilities.

The Importance Of Fill

The strength, compressibility, permeability, and other characteristics of a fill depend on the type of fill material, the methods used to construct the fill, and the moisture content of the fill material at the time of fill construction.

FILL MATERIALS
Continued…

A well-constructed (engineered) fill built with the proper materials can adequately serve the design purpose, such as:

- Support structures or other facilities – Buildings, Bridges, Retaining Walls, Roadways, Parking Areas, etc. (typically desire granular soils for this purpose, however, silty or clayey soils are adequate in many cases)
- Act as an "Impermeable" Liner – Landfills, Water Retention Basins, etc. (typically use clayey soils for this purpose)
- Act as a Barrier – Dams, Levees, and etc. (Typically desire best soil mixture available to provide strength and "impermeability")
- Act as a Drainage Layer – Dams, Retaining Walls, Trenches, Roadways, etc. (exclusive use of granular materials for this purpose)

A poorly constructed fill, using improper materials or poor methods of construction can potentially lead to excessive settlement, expansion/shrinkage, undesired permeability rates, low soil strength, slope instability, and the failure of the facility relying on that fill.

Identifying Fill Materials

Any one or combination of the following characteristics might indicate the presence of fill materials, but not necessarily!! – USE GOOD JUDGEMENT or as for a second opinion from senior personnel!!!

1. Records indicated that an area has been filled
2. An area is higher than the surrounding (natural) landscape
3. Man-Made objects are present within the soil (Refuse, concrete, asphalt, deleterious materials etc.)
4. The soil consistency (blow count) differs from the layer immediately below
5. The soil type differs from the layer immediately below
6. The "large-scale" structure of a fill layer often exhibits a mixture of soil colors, soil types, and an arbitrary orientation of soil layers (i.e. naturally deposited soils most often exhibit uniformly oriented – near level – layers of a single color per layer). Layers (or lifts) of fill may be discernable in cuts or open excavations.

Please note that in many cases, a fill is constructed with soils excavated (cut) close-by, therefore, the soil-type of fill materials is often nearly identical to undisturbed native soils immediately below. In such cases, characteristics other than "soil type" must be used to identify a material as "fill".

DEPTH MEASUREMENTS – DRILLING AND SAMPLING

When drilling a test boring or monitoring well, the depth of the drilling and sampling tools must be known to a pre-specified accuracy. In geotechnical and environmental work, depth measurements with accuracy of 0.1-ft are typically sufficient.

To figure the depth of the drilling or sampling tool below the ground surface, add the lengths of all the tools inserted into the hole, then subtract the amount remaining above a fixed point – such as ground surface. (Caution: Do not use the top of hollow auger or drill casing as a "fixed" reference point for measuring depth since either might be inserted further into the hole!).

Hypothetical Augering Example: A test boring is drilled with hollow auger. As documented on the technical page of your notes, each flight of auger is 5.0-ft long and the auger bit is 1.2-ft long. So far, the auger bit and five flights of auger have been assembled and drilled into the ground. After drilling and removing the auger cap, you measure 1.7-ft of auger above the ground surface.

Q. How deep is the tip of the auger bit where you wan to sample now?
A. The answer is the sum of the auger lengths (5 x 5.0-ft) plus the length of the auger bit (1.2-ft) minus the length of auger above ground surface (1.7-ft).

Auger Tip Depth = 25.0 ft + 1.2 ft – 1.7 ft = 24.5-ft depth below ground surface.

Hypothetical Rotary Drill Example: A test boring is drilled with rotary drill tools. As documented on the technical page of your notes, each section of drill rod is 5.0-ft long, the drill bit is 0.7-ft long, each section of casing is 5.0-ft long and the casing shoe is 1.0-ft long. So far, the casing show is six sections of casing have been driven into the ground with 1.1-ft of casing remaining above the ground surface. The drill bit and seven sections of drill rod have been assembled and drilled into the ground through the casing, however, when drilling is completed, 5.9-ft f rod remains above ground surface.

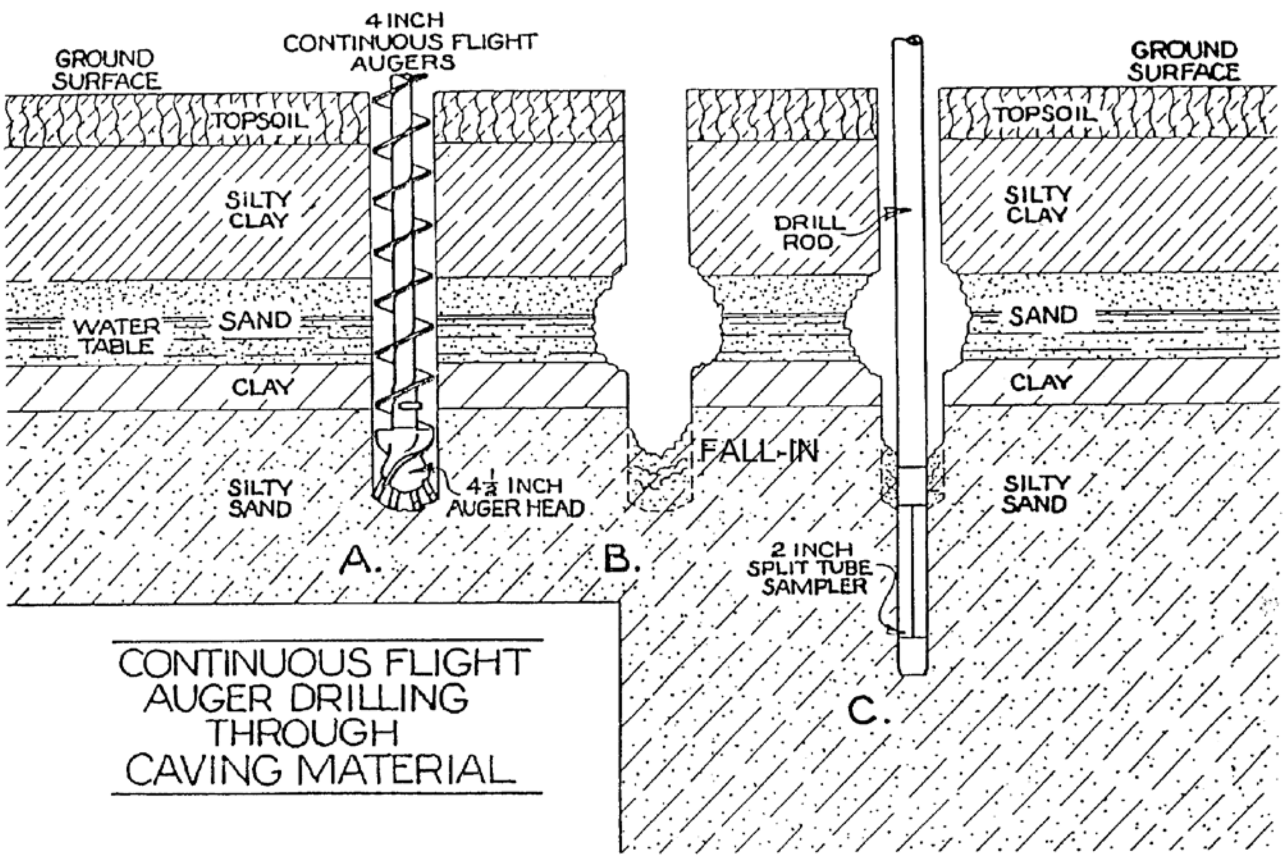
Q. How deep is the casing? How deep is the drill bit? Is the drill bit out from inside the bottom of the casing?
A.
Casing depth = depth of casing plus depth of casing shoe minus stick-up
= 6 x 5.0-ft + 1.0-ft – 1.1-ft = 29.9-ft below ground surface.
Drill bit depth = depth of drill rod plus depth of drill bit minus stick-up
= 7 x 5.0-ft + 0.7-ft – 5.9-ft = 29.8-ft below ground surface.

Therefore, the drill rod is still inside the bottom of the casing. The bit must be advanced at least 0.1-ft in order penetrate materials below the casing.

FALL-IN (CAVING)

When a boring is not supported by casing (pipe) or hollow auger, the boring sidewalls are susceptible to caving into the hole. The result is either a "bridge" above the base of the hole, or some depth of "slough" accumulated at the base of the hole. Sampling through an excessive thickness (say, 6-inches or more) of slough is typically unacceptable, producing error in the blow count and fewer undisturbed materials within the sampler.

To determine the amount of fall-in after a drill interval, compare the depth achieved during drilling (depth of bit) to the depth where sampling tools or a weighted measuring tape comes to rest in the boring. Caving which produces fall-in can be mitigated using casing and/or drill mud.



Solid-Flight Augers: General Unacceptable Utilization

LOGGING DRILLING INTERVALS

When logging the Drilling Interval, describe information such as the following:

1. Drill Depth Interval – Start/Finish Depths of the Drilling Interval
2. Drill Tools Used –Solid Flight Auger (SFA), Hollow Stem Auger (HSA), Rotary Bits (size, side or face discharge), Casing Installed (size), etc.
3. Classification ad Description of Soils Encountered – per Unified Soil Classification System (e.g., CL, SM, GP, etc.)
4. Depth(s) at Soil Layer Break(s)
5. Type of Soil Layer Break – Approximate, Conformable, Unconformable
6. Drill Penetration Rate – ft/min, etc. (rotary boring note circulation)
7. Drill "Action – Smooth, "choppy", or some combination thereof
8. Bulk Samples Taken – Sample Number, Interval, Description, etc.

One sample format and description of a drilling interval is as follows:

DEPTH	TOOL	BLOWS	DESCRIPTION
0.0-4.5'	4"SFA		Semicompact brown SILTY fine to medium SAND with fin GRAVEL, SM, wet, sub-rounded particles, smooth, steady and quick drilling, break at approximately 3-ft/ to stiff dark gray to black SILTY CLAY with PEAT stringers, CL, wet, slight organic odor, very slight dialtency, moderate dry strength, continued smooth, steady, and quick drilling

LOGGING SAMPLING INTERVALS

When logging the Sampling Interval, describe information such as the following:

1. Sample Depth Interval – Start/Finish Depths of the Sampling Interval
2. Sample Number – Sequential within a particular boring (e.g. sample no. 3 is (3))
3. Indication of Sampler Used – ID of Sampler (e.g., 1.4”, 2”).
4. Hammer Penetration Data –blows/0.5 ft penetration (Standard Penetration Test – SPT); for cobbles record blows/0.1 ft penetration.
5. Description and Classification of Soils Recovered in Sampler
6. Percent Sample Recovery – Recovered Length/Sampled Length –recovered 0.75 ft from 1.5 ft sampling length is 50% recovery
7. Number of Samples Retained for Laboratory Testing and/or Reference –e.g., two tubes retained is 2t
8. Depth at Soil Layer Break(s)
9. Type of Layer Break(s) – Approximate, Conformable, Unconformable

One sample format and description of a sampling interval is as follows:

DEPTH	TOOL	BLOWS	DESCRIPTION
4.5-6.0’	(1) 1.4” 75%, 1t CL		Stiff dark gray to black SILTY CLAY with PEAT stringers, CL, wet, slight organic odor, very slight dilatency, moderate dry strength, conformable contact break at 5.2-ft/ to semicompact brown SILTY fine to medium SAND with fine GRAVEL, SM, wet, sub-rounded particles

LOGGING GROUNDWATER

The depth to groundwater is typically measured where it is Encountered within a test boring, and then where it Stabilizes within the test boring. Stabilized groundwater level can be either the same, higher, or lower than the encountered level due to many soil and geologic factors. Groundwater is typically measured with an accuracy of 0.01-ft when suitable equipment is available, or the nearest 0.1-ft with a standard measuring tape or electronic depth sampler.

To log Groundwater within an Auger Test Boring, describe information such as the following:

- Date/Time/Depth where Groundwater is Encountered
- Date/Time/Depth of Groundwater at Periodic Intervals Until Groundwater "Stabilizes"

To log Groundwater in a Rotary Drilled Test Boring a bailer is typically used to remove drill fluid from the boring. Describe information such as the following:

- Date/Time/Depth of Groundwater/Drill Fluid Prior to Bailing
- Volume and Rate of Groundwater/Drill Fluid Bailed from the Boring
- Date/Time/Depth of Groundwater/Drill Fluid After Bailing
- Date/Time/Depth of Groundwater at Periodic Intervals Until Groundwater "Stabilizes"

PRINCIPLES OF LOGGING MONITORING WELLS AND PIEZOMETERS

A piezometer is constructed to obtain groundwater level measurements primarily for geotechnical purposes. A monitoring well is constructed to sample the groundwater and measure the groundwater level for environmental purposes.

Log The Soil Profile And Groundwater (per Geotechnical Test Borings)

Plus, where environmental conditions are known or suspected, record visual and olfactory evidence for presence or absence of contaminants.

Log The Monitoring Well Construction - - Information regarding each step of the monitor well construction should be described and recorded.

Pipe
- Pipe lengths/Diameters/"Schedules" (e.g. 15.3-ft/2"ID/Schedule 40)
- Manufacturer of materials used
- Screen size/Interval (e.g. 0.02-in machine-slotted pipe from 26.7-ft to 42.0-ft depth
- Blank interval (e.g. blank pipe from 0.0 to 27.6-ft depth
- Joint type (i.e. flush-mount, threaded, coupler, etc.)
- Caps (e.g. Bottom-caps and Top-caps)

Filter
- Depth interval where filter is placed (e.g. from 25.6-42.0 ft depth)
- Sand size/designation (e.g. "No. 3" sand)
- Manufacturer of materials used
- Quantity of filter material used in hole (e.g. cubic feet)
- Method of materials placement (i.e. tremie, pumped, poured, etc.)

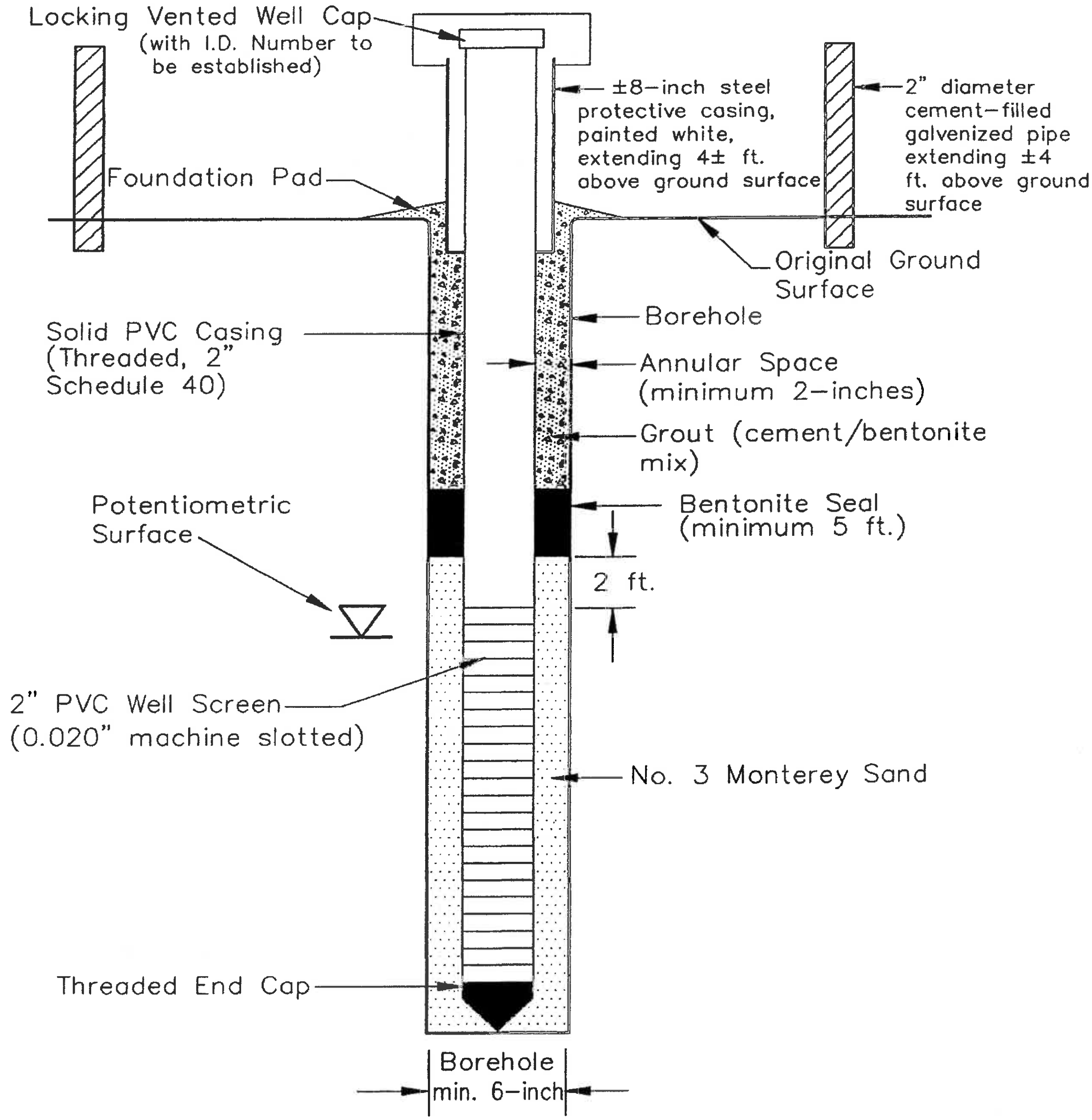
Seals
- Depth interval where seal is placed (e.g. from 20.6-25.6 ft depth)
- Size/type of seal materials used (e.g. ½-inch bentonite pellets)
- Manufacturer of materials used
- Quantity of seal material used in hole (e.g. cubic feet)
- Method of materials placement (i.e. tremie, pumped, poured, etc.)

Grout/Backfill
- Depth interval where grout/backfill is placed (e.g. from 0.0-20.6 ft depth)
- Materials used (i.e. cement, bentonite, powder, sand, water, etc.)
- Manufacturer of materials used
- Grout/backfill ("batch"- 2 ft^3 cement, 0.5 ft^3 bentonite powder & 30 gal. water)
- Quantity of grout used in fole (e.g. cubic feet)
- Method of material placement (i.e. tremie, pumped, poured, etc.)

Surface Completion
- Type/description of well box
- Manufacturer of materials used
- Size/construction of well pad (e.g. 2'x2'x4' concrete pad)
- Well box lock number

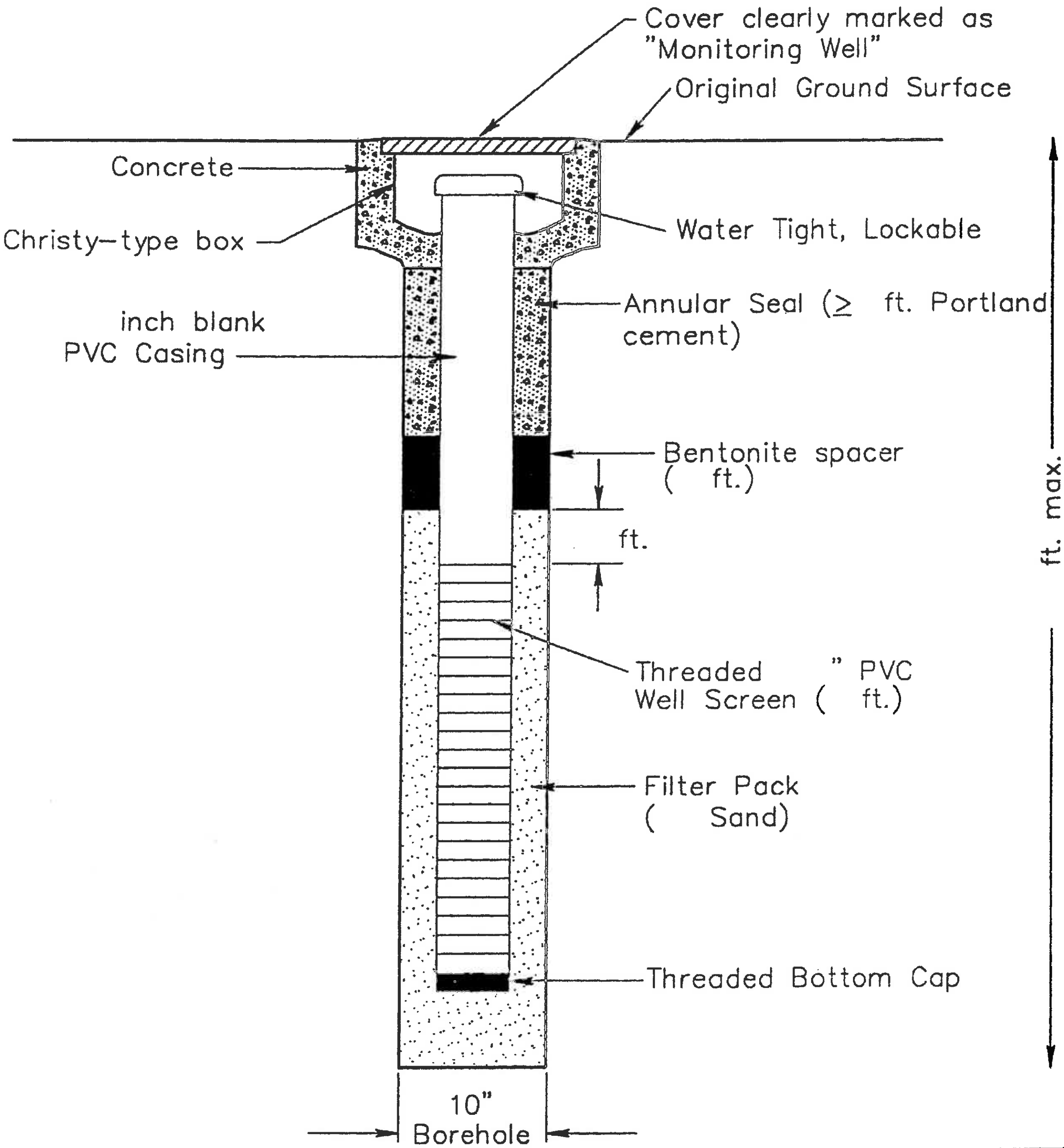
Miscellaneous
- Well number
- Well location
- Well elevation (i.e. elevations on pad, pipe, box, ground, etc.)

Graphic Log
- Sketch the monitor well construction to scale if possible (see Logs)



No Scale

WELL CONSTRUCTION DETAIL		
		Figure -



Not to Scale

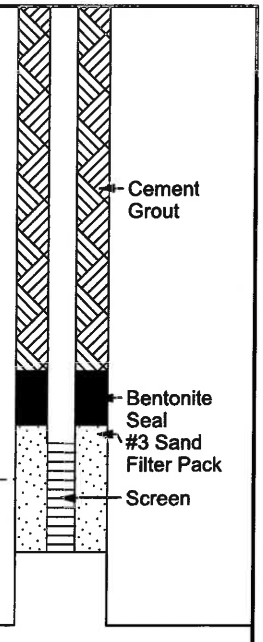
MONITORING WELL SCHEMATIC

Figure -

TYPE: 6" CP WIRELINE

SURFACE ELEVATION: EGS

UNCONFINED COMPRESSIVE STRENGT (tsf)	OTHER TESTS	DRY DENSITY (pcf)	Moisture (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (inches)	SAMPLE No.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION SYSTEM	DESCRIPTION
							0			Very soft, dark brown to black PEAT with fine SAND and SILT, wet
							5		∇ PT	
				Push	1.4	1				
							10			
				Push	1.4	2				
										Very soft, blue-grey to dark brown organic SILTY CLAY, wet
							15		CL	
				Push	1.4	3				
							20			
							25			
							30			
							35			
							40			



Bottom of hole at 17.0 feet.

Borehole very close (5-ft +/-) to boring no. 6 (10 new) assumed same groundwater table.

Borehole caved in to a depth of 15-ft.

"P" dentotes a push (the weight of the hammer pushed the sampler into the soil), no blow counts were recorded.

Covered with steel christy box extending approx. 3-ft above ground surface. 9/03/2008.

THE BORING LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOGGED BY: KMK | DATE: 09-03-2008

BORING LOG

Project No.

PIEZOMETER BORING/CONSTRUCTION LOG

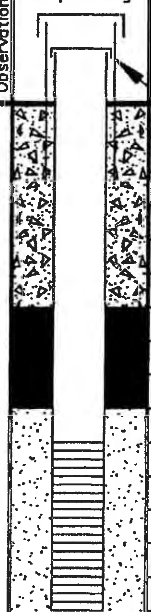
Job No.: 2P3/XXX/XXX

Project Name:

Client:

WELL CASING: 3/4" DIA. PVC		FROM	15	TO	0	ft.
TYPE OF PERFORATION: .020" Machine slotted		FROM	15	TO	10	ft.
SIZE AND TYPE OF FILTER: #3 Sand		FROM	15	TO	9	ft.
TYPE OF SEAL	NO. 1 1/4" Bentonite pellets	FROM	9	TO	6	ft.
	NO. 2 Cement/bentonite slurry	FROM	6	TO	0	ft.
	NO. 3	FROM		TO		ft.

Well No. P-1	Location: See Plan
Elevation: −5.79*	Reference: Per Survey
Drilling Equipment: Diedrich 120	
Drilling Method: 8-Inch Hollow Stem Auger	
Notes: * At Existing Ground Surface (E.G.S.)	



Depth (feet)	REMARKS (drill rate, fluid loss, odor, etc.)	SOIL TESTS	BLOWS/FOOT 350 ft lb.	SAMPLE SIZE (inches)	SAMPLE No.	UNIFIED SOIL CLASSIFICATION	Material Description
0–5	E.G.S. to top of well cover 2.1ft. Piezometer pipe 0.2ft. below top of well cover					CL / SM	Dense to compact black and brown CLAYEY SILT/ SILTY SAND with GRAVEL (fill)
5–9						CL / SP	Semicompact and stiff black and brown CLAYEY SAND/SANDY CLAY (fill)
9–14						Pt	Very soft dark brown and black PEAT and SILTY PEAT
14–17.5						Pt / SM	Very soft dark brown and black PEATY SILT and SILT with PEAT stringers and thin SANDY SILT layers

Depth scale: 5, 10, 15, 20, 25, 30, 35

THE MONITOR WELL LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

LOGGED BY: T.A.K.	CHECKED BY: G.D.A.	DATE: 9-20-94	FIGURE Page of

TYPE: 4-INCH AUGER/3.75-INCH Drag Bit

SURFACE ELEVATION: 10.2

UNCONFINED COMPRESSIVE STRENGT (tsf)	OTHER TESTS	DRY DENSITY (pcf)	Moisture (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (inches)	SAMPLE No.
	G			9	1.4	1
	G			9	1.4	2
	G			15	1.4	3
	G			22	1.4	4
	G			22	1.4	5
	G			21	1.4	6
	G			24	1.4	7
	G			28	1.4	8
	G			29	1.4	9
	G			28	1.4	10
	G			20	1.4	11
	G			7	1.4	12
				7	1.4	13
				6	1.4	14
				5	1.4	15

DEPTH IN FEET: 5, 10, 15, 20, 25, 30, 35, 40

UNIFIED SOIL CLASSIFICATION SYSTEM	Description
CL-ML	Loose, brown to dark brown CLAYEY SILT with SAND and mica, moist.
SP-SM	Semicompact to compact, brown to gray (turns blue-gray with depth), fine to coarse SAND with SILT and mica, wet.
CL-ML	Very soft to soft blue-gray mottled green and brown CLAY with varying amounts of SILT, rootlets present at 39.0-ft depth.

MATERIAL SYMBOL

--CONTINUED--

THE BORING LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOGGED BY: TJC

DATE: 10-13-2008

BORING LOG

Project No.

TYPE: 4-INCH AUGER/3.75-INCH Drag Bit SURFACE ELEVATION: 10.2

UNCONFINED COMPRESSIVE STRENGT (tsf)	OTHER TESTS	DRY DENSITY (pcf)	Moisture (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (inches)	SAMPLE No.	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION SYSTEM	
				4	1.4	16				
										Bottom of hole at 41.0 feet.
										Borehole elevation reported by AGI.
							45			Borehole backfilled with cement grout at completion of drilling. 10/13/2008.
							50			
							55			
							60			
							65			
							70			
							75			
							80			

THE BORING LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOGGED BY: TJC DATE: 10-13-2008

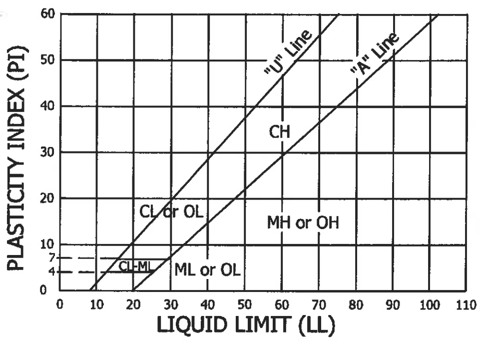
BORING LOG

Project No.

UNIFIED SOIL CLASSIFICATION SUMMARY

(ASTM D 2489)

Pt	OH	CH	MH	OL	CL	ML	SC	SM	SP	SW	GC	GM	GP	GW
Highly organic soils	Silts and clays Liquid limit 50 or more			Silts and clays Liquid limit less than 50			Sands with fines >12% fines		Clean sands < 5% fines		Gravels with fines > 12% fines		Clean gravels < 5% fines	
							Sands-50% or more of coarse fraction is smaller than No 4 Sieve				Gravels-more than 50% of coarse fraction is larger than No 4 sieve			
Fine grained soils (50% or more is smaller than No 200 sieve)							Coarse grained soils (More than 50% is larger than No 200 sieve)							



LABORATORY CLASSIFICATION CRITERIA

GW and SW - Cu > 4 for GW and 6 FOR SW; 1 < Cc < 3

GP and SP-Clean gravel or sand not meeting requirements for GW and SW.

GM and SM-Atterberg limits of fines below "A" line or P.I. less than 4.

GC and SC-Atterberg limits of fines above "A" line with P.I. greater than 7.

Fines (silt or clay)	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		
Sieve sizes	200	40	10	4	3/4"	3"	10"

Classification of earth materials shown on the test boring logs is based on field observation and should not be construed to imply laboratory analysis unless so stated.

MATERIAL SYMBOLS

- Gravel
- Sand
- Silt
- Clay
- Sandy clay or clayey sand
- Sandy silt or silty sand
- Silty clay or clayey silt
- Peat and/or organic matter
- Fill material
- Igneous rock
- Sedimentary rock
- Metamorphic rock

CONSISTENCY CLASSIFICATION FOR SOILS

Standard Penetration "N"-Value*	Granular	Cohesive
0-5	Very loose	Very soft
6-10	Loose	Soft
11-20	Semicompact	Stiff
21-35	Compact	Very stiff
36-70	Dense	Hard
> 70	Very dense	Very hard

* According to the Standard Penetration Test (ASTM D 1586)

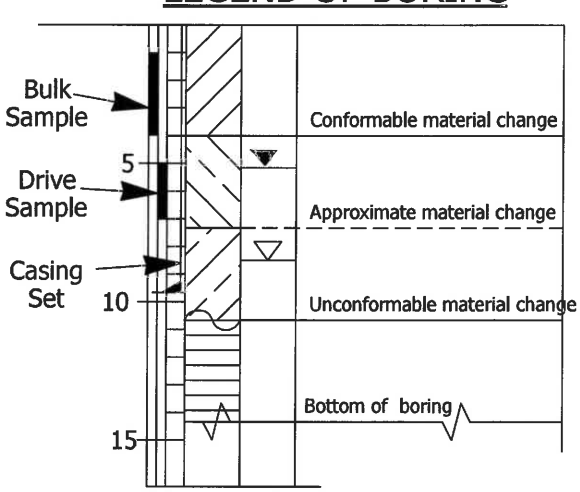
Blow count of 50/0.5 indicates 50 blows for 1/2 foot.

Where standard penetration test has not been performed, consistencies shown (in parenthesis) on logs are estimated.

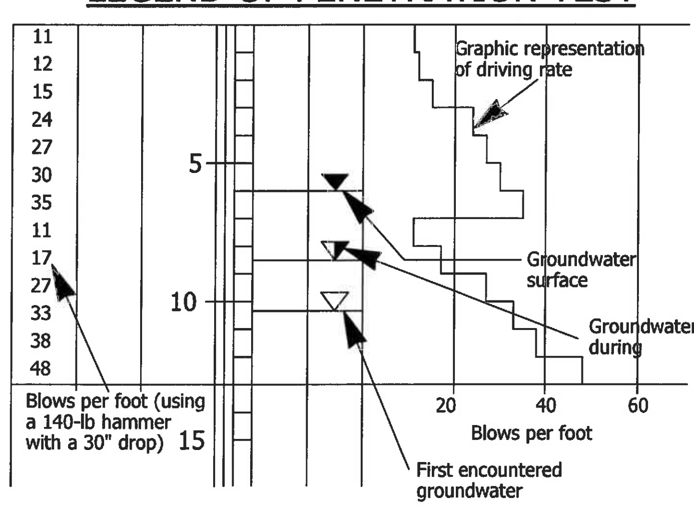
KEY TO "OTHER TESTS" LABORATORY

A - Atterberg Limits
C - Consolidation
CR - Corrosivity
E - Expansion Index
G - Gradation
H - Hydrometer
M - Maximum Dry Density
P - Permeability
R - Resistance Value
S - Direct Shear
SE - Sand Equivalent
SG - Specific Gravity
T - Triaxial Shear

LEGEND OF BORING



LEGEND OF PENETRATION TEST



BORING LEGEND	Project No.
Client	*Proj #*
Proj Name Loc, CA	*Date*
	FIGURE *

American Society for Testing and Materials (ASTM) Designations for Test Methods

ASTM Designation: D 1452 – 07a
Standard Practice for Soil Investigation and Sampling by Auger Borings

ASTM Designation: D 1586 – 99
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils

ASTM Designation: D 1587 – 00
Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes

ASTM Designation: D 2113 – 06
Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation

ASTM Designation: D 2487 – 06
Standard Practice for Classification of Soils for Engineering Purposed (Unified Soil Classification System)

ASTM Designation: D 2488 – 06
Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)

ASTM Designation: D 3550 – 01 (Reapproved 2007)
Standard Practice for Thick Wall, Ring-Lined, Split-Barrel, Drive Sampling of Soils

For further information about ASTM, its function and its purpose, visit their website at http://www.astm.org

Sampling Techniques

<u>STANDARD PENETRATION TEST (SPT) – SUMMARY</u>

(Refer to ASTM D 1586)

The "Standard Penetration Test" is a field test used to describe soil consistency. The purpose of the test, outside of retrieving a soil samples, is to investigate soil resistance to sampler penetration. The product of the test is a number which describes how many hammer blows were required to advance the sampler over a certain interval (1-ft, as described below). This "blow-count" is known as "N". The N value obtained in the field may need to be modified for hammer type, sampler type, boring size, overburden pressure, etc. for use in design. The Standard Penetration Test is used world-wide so that applicable soil test and research on subjects such as liquefaction potential, bearing capacity, and settlement may be shared by all geotechnical engineers, geophysicists, and geologists.

The principle components of the Standard Penetration Test are as follows:

- Use the Standard Penetration Sampler (1 3/8-inch I.D., 2.0-inch O.D.)
- Hammer weight is 140-lb
- Three 6-inch intervals are marked on the drill rod –with reference to a fixed point such as ground surface (i.e. total sampler penetration in each sampling interval is 18-inches)
- The number of hammer blows required to penetrate <u>each six inch interval</u> is logged as the sampler is driven into the ground
- The Standard Penetration County ("N") is the sum of the <u>last two six inch intervals</u> (i.e. for blow counts recorded as 3/6/7, "N" = 13)

NOTES on the STANDARD PENETRATION TEST

Origins of the Standard Penetration Test

Around 1902 Colonel Charles R. Gow, owner of the Gow Construction Co. in Boston, began making exploratory borings using 1-inch diameter drive samplers (Fig. 1). Up until that time, contractors used wash borings with cuttings, similar to the methods presently used in advancing water wells. During the late 1920s and early 1930s, the procedure was standardized by Harry Mohr, one of Gow's engineers, then with Raymond Concrete Pile Co. (H.A. Mohr, 1940, *Exploration of Soil Conditions and Sampling Operations:* Bull 269, Graduate School of Eng'g, Harvard University). Mohr developed a slightly larger diameter split-spoon drive sampler and recorded the number of blow counts per foot of penetration on an 18-inch deep sample round, using a 140-lb hammer dropping 30 inches, pushing a 2-inch outside diameter sampler, while recovering a 1-3/8 inch diameter sample, as shown in Figs. 2 and 3. The value recorded for the first round of advance is usually discarded because of fall-in and contamination in the borehole. The second pair of numbers are then combined and reported as a single value for the last 12 inches (1 foot). This value is reported as the SPT blowcount value, commonly termed "N".

Not everyone used Gow's sampler, which originated in the Boston area, but Karl Terzaghi liked it. Terzaghi and Arthur Casagrande vigorously sponsored adoption of the split spoon sampling procedure through the auspices of ASCE's Committee on Sampling and Testing of the Soil Mechanics and Foundations Division of ASCE, formed in 1938. The work of this committee was carried out at Harvard by Juul Hvorslev, and pretty much standardized by 1940, when Hvorslev wrote *"The Present Status of the Art of Obtaining Undisturbed Samples of Soils"*, included as an 88-page appendix to the Purdue Conference on Soil Mechanics and Its Applications.

Terzaghi's concept of using "standard" blow counts to estimate soil properties was not realized until 1947, when he sat down with Harry Mohr and developed correlations between allowable bearing pressure and [SPT] blowcounts in sands, while completing his draft of *Soil Mechanics in Engineering Practice.* Later that year Terzaghi christened the 2-inch Gow sampler the "Standard Penetration Test", in a presentation titled "*Recent trends in subsoil exploration*", which he delivered to the 7th Conference on Soil Mechanics and Foundation Engineering at the University of Texas. The first published SPT correlations appeared in Fig. 177 on p. 423 of *Soil Mechanics in Engineering Practice (First Ed.)* by Terzaghi and Peck, published in 1948.

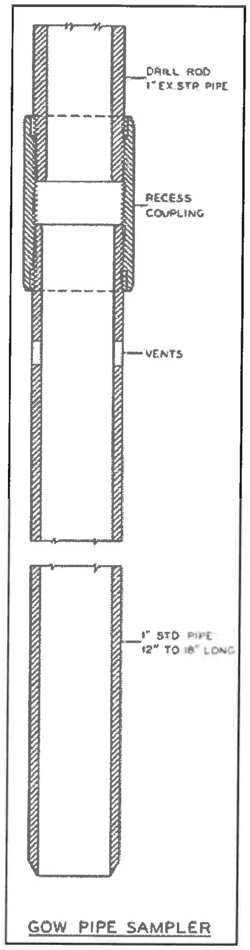
The "standard drive sampler" test was subsequently adopted by ASCE and The Corps of Engineers in Hvorslev's *"Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes"*, which appeared in November 1949 (reprinted by The Engineering Foundation in 1962 and 1965). Sprague and Henwood began producing the Mohr 2-inch diameter split spoon sampler in the early 1950s and it became a nation-wide standard in 1958 when the apparatus and procedures were officially adopted by ASTM as Test Method D1586 (and last revised in 1984). The ASTM sanctioned sampler and its dimensions are shown in Fig. 3.

Baseline References on the SPT procedure

Many of the SPT correlations have been explored, and there exist no small number of problems, requiring considerable judgment. Most of these problems are discussed in the following articles:

1) G.F.A. Fletcher, 1965, *Standard Penetration Test: Its Uses and Abuses*: Journal Soil Mechanics & Foundations. Div., ASCE, v. 91:SM4, p. 67-75.

2) Ireland, Moretto and Vargas, 1970, *The Dynamic Penetration Test: A Standard That is not Standardized*: Geotechnique, v. 20:2, p. 185-192;



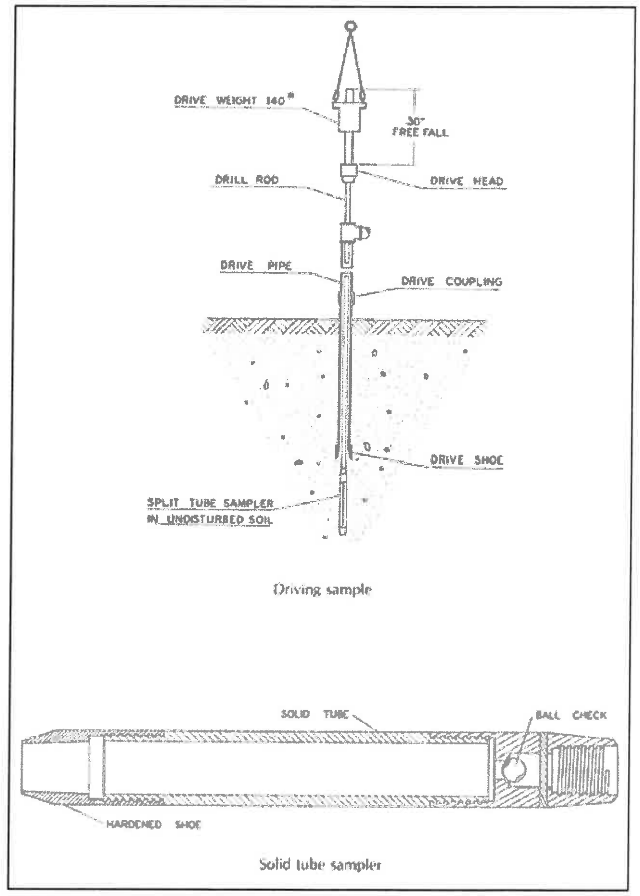
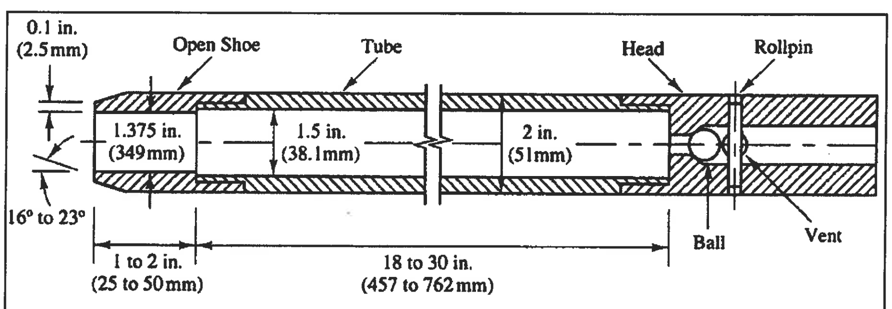


Figure 1 (above left) – *The original Gow Pipe Sampler utilized 1-inch diameter drill rod and 1-inch diameter pipe with a beveled cutting tip. It was introduced around 1902.*

Figure 2 (above right) – *Components of SPT spilt spoon sampler, as developed by Harry Mohr in the early 1930s, after Gow Construction had been absorbed by the Raymond Concrete Pile Company.*



The SPT sampler (Adapted from ASTM D1586; Copyright ASTM, used with permission)

Figure 3 – *Standard dimensions for the SPT sampler, as given in ASTM D1586.*

3) V.F.B. de Mello, 1971, *The Standard Penetration Test*: Proceedings of the 4th Panamerican Conference on Soil Mechanics and Foundation Engineering: San Juan, PR, v.1:1-86; and

4) Yves Lacroix and Harry Horn, 1973, *Direct Determination and Indirect Evaluation of Relative Density and Its Use on Earthwork Construction Projects:* in *Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils*: ASTM Special Technical Publication 523, p. 251-280.

In 1986 a series of new correlations and corrections were introduced, which are in current usage. These include:

5) C.O. Riggs, 1986, North American Standard Penetration Test practice: An essay: in Use of Insitu Tests in Geotechnical Engineering, ASCE *Geotechnical Special Publication No. 6*;

6) A.W. Skempton, 1986, Standard Penetration Test Procedures and the Effects in Sands of Overburden Pressure, Relative Density, Particle Size, Aging and Overconsolidation: *Geotechnique*, v. 36:3, p. 425-447; and

7) S.S.C. Liao and R.V. Whitman, 1986, Overburden Correction Factors for SPT in Sand: *Journal of Geotechnical Engineering*, A.S.C.E., v. 112:3, p. 373-377.

In 1990, Clayton presented an expanded listing of SPT hammer efficiencies (E_m) in:

8) C.R.I. Clayton, 1990, SPT Energy Transmission: Theory, Measurement, and Significance: *Ground Engineering*, v. 23:10, p. 35-43.

For evaluation of liquefaction potential, raw SPT blowcounts must be corrected to $(N_1)_{60}$ values, as described in the following sections.

Burmister's (1948) input energy correction

Despite all the encouragement to adopt Terzaghi's SPT test, most people went on using whatever devices they had previously, until more and more of Casagrande's students infiltrated the ranks of foundation engineering. In the New York area the favored device was the 3.625-inch diameter Moran & Proctor, or M & P Sampler, which had been developed by Carlton Proctor for the firm's exploration of the San Francisco Bay Bridge project (C. S. Proctor, 1936, The Foundations of the San Francisco-Oakland Bay Bridge: *Proceedings of the Int'l Conference on Soil Mechanics and Foundation Engineering*, Harvard Univ., v.3, p. 183-193). The M & P sampler allowed recovery of a much larger 3-inch diameter samples, using 5000 in-lbs. per blow in lieu of the SPT's 4,200 in-lbs. The M & P sampler is shown in Fig. 4. Moran & Proctor engaged Professor Don Burmister of Columbia University to develop a scheme for correlating M & P sampler blow counts with those of Mohr's SPT sampler, commonly employed in New England and elsewhere (after 1948). Burmister assumed that SPT blowcounts relate energy input versus the area of the sampler barrel and sample. He reasoned that simple correlations could be made between the various size samplers by ignoring the increase in skin friction area that accompanies larger diameter samplers and increasing skin friction with depth. Despite the simplistic physics, these rough correlations have proven valuable in practice. Burmister suggested a simple input energy correction for the ratio of driving weight (hammer energy) versus sample diameter, published as: D.M. Burmister, 1948, The importance and practical use of relative density in soil mechanics: *Proceedings of ASTM*, v. 48:1249. Burmister's relationship only considered energy input (weight of hammer multiplied by drop height), size of the recovered sample (D_i) and sample barrel diameter (D_o).

This could be rewritten to provide input energy and diameter correction for other tests to correlate with the SPT (ASTM D-1586):

$$N^* = N_R \frac{(W\ \text{lbs})(H\ \text{in})}{(140\ \text{lb})(30\ \text{in})} \left[\frac{(2.0\ \text{in})^2 - (1.375\ \text{in})^2}{(D_o)^2 - (D_i)^2} \right]$$

Where W is the hammer weight, H is the height of the drop, D_o is the outside diameter of the sample barrel, D_i is the diameter of the drive sample, N_R is the raw blow count, and N* is the blowcount reported as the equivalent SPT value. The Burmister energy correction takes the raw "SPT" blow count value and multiplies it by an appropriate fraction, derived from the relationship above. The corrected blowcount value is usually denoted by an asterisk (*) on the boring log, with a note explaining that the blow counts have been adjusted.

If we apply Burmister's simple equation to the Modified California sample barrel, with an outside diameter of 3.0 inches and a sample diameter of 2.4 inches, the calculated correction would be 0.65. This means the equivalent SPT N values would be about 65% of those recorded with the Modified California apparatus. Most workers cite Burmister's 1948 "correction" for adjusted blow counts recorded with larger diameter drive samplers, or for lower energy hammers (such as the 70-lb hammer used with Mobile Drilling's Minuteman portable drilling rig).

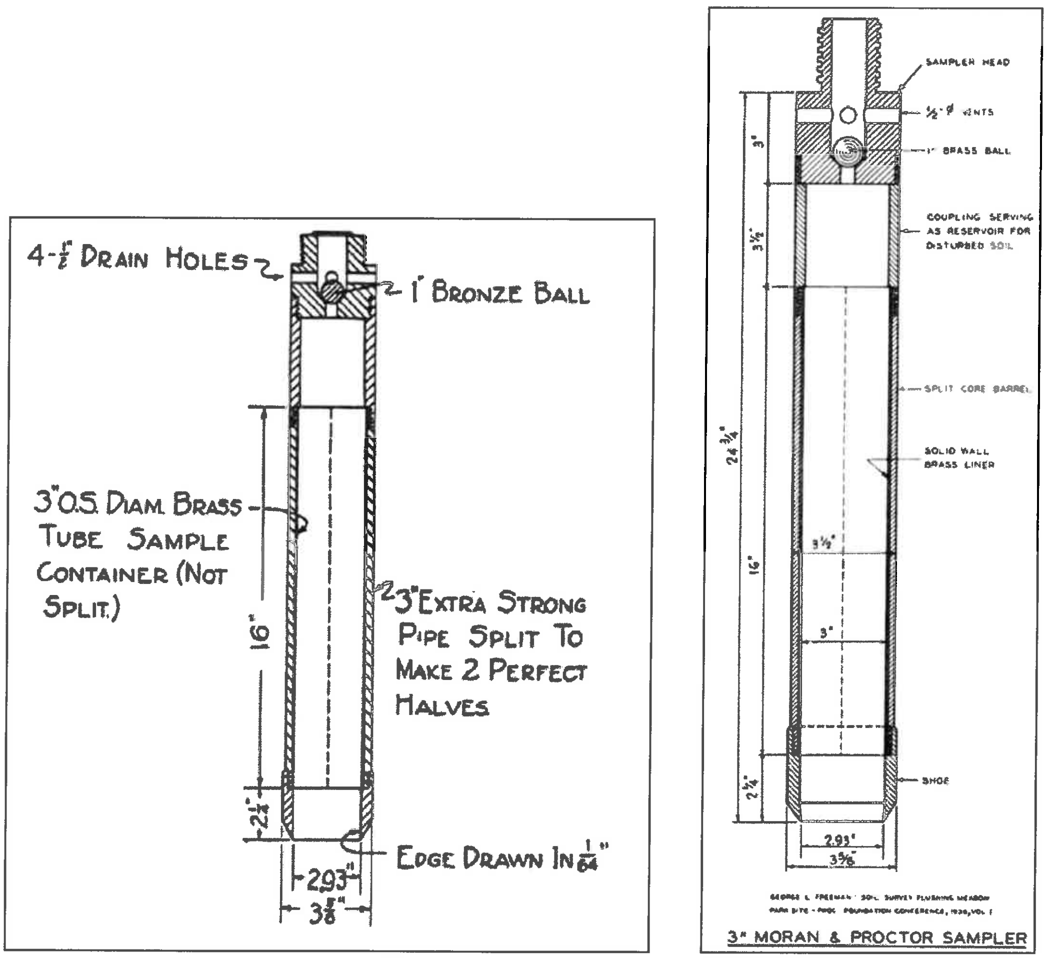


Figure 4 – *The M & P drive sampler was developed by Carlton Proctor of Moran & Proctor in the early 1930s. It was the preferred drive sampler in the New York City area because it recovers a less disturbed sample than the smaller diameter SPT.*

Lacroix and Horn (1973) correction

In 1973 Yves Lacroix and Harry Horn of Woodward-Clyde wrote an article titled "Direct Determination and Indirect Evaluation of Relative Density and Its Use on Earthwork Construction Projects", published in

Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils: *ASTM Special Technical Publication 523*, p. 251-280. This ASTM conference had been held in Los Angeles in June 1972, and the correction proposed by Lacroix and Horn was subsequently adopted by many geotechnical practioners <u>when working close to geologic contacts with stiffer materials</u>.

Lacroix and Horn (1973) proposed that nonstandard penetration resistance, N_l, could be correlated with Standard Penetration Resistance, N, for drive samples or a solid conical point, such as a static cone, which incorporated consideration of driving energy and distance of penetration. They reasoned that the energy required to drive the sampler or cone a given distance or "depth" (L) was directly proportional to the square of the outside diameter (D) and the distance of penetration, and inversely proportional to the energy per blow (Weight of hammer multiplied by the height of drop, WH):

$$N = N_l\,(2\text{ in}/D_l)^2 \text{ x } 12\text{ in}/L_l \text{ x } W_l/140\text{ lb x } H_l/30\text{ in} = \frac{2\,N_lW_lH_l}{175\,D^2L_l}$$

When this correction is applied to the Modified California sampler (D_o = 3.0 in., and D_i = 2.4 in), the predicted correction is reduced, to 0.44 N_l, where N_l is the raw blowcount recorded during driving of the larger diameter Modified California sampler. The depth of penetration (L_l) in the denominator is the distance the sampler was advanced during the sample round. In most instances, this is given as the last 12 inches in a 18 inch sampling round (so, L_l = 12 inches). The Lacroix and Horn correction gives a more conservative estimate of the SPT blowcounts than that derived by Burmister's (1948) energy correlation (discussed previously). The LaCroix and Horn correction appears most valid when sampling is undertaken within 5 to 8 sample barrel diameters (15" to 24") of a geologic contact with a stiffer horizon (such silty or clayey materials above cemented sands or gravels). The stiffness of the base layer causes the blow counts to become elevated because the tip of the sampler senses the ground beneath its cutting tip.

Side-by-Side Correlations

Between 1991-95 the author performed a series of side-by-side sampling tests using the SPT and Modified California samplers in Tertiary-age sedimentary materials along the California coast to evaluate what manner of correction seemed most appropriate under general conditions of use. This study revealed that the best fit correction is close to: SPT N = 0.50 to 0.67 N_R, where N_R is the raw recorded blowcount on the larger sampler (the average correction was about 0.56). The Lacroix and Horn correction would appear most applicable to those situations where the sampler is being driven through relatively low stiffness materials in close proximity to a much stiffer horizons (like cemented sands or gravels) within 5 to 8 barrel diameters (15" to 24") beneath the sampler tip.

Standardized SPT corrections

SPT data can be corrected for a number of site specific factors to improve its repeatability. Burmister's 1948 energy correction assumed that the hammer percussion system was 100% efficient (a 140-lb hammer dropping 30 inches = 4,200 ft-lbs raw input energy). In A.W. Skempton, 1986, *Standard Penetration Test procedures and the Effects in Sands of Overburden Pressure, Relative Density, Particle Size, Aging and Overconsolidation*: Geotechnique, v. 36:3, p. 425-447, the procedures for determining a standardized blowcount were presented, which allow for hammers of varying efficiency to be accounted for. This corrected blowcount is referred to as "N_{60} ", because the original SPT (Mohr) hammer has about 60% efficiency, and this is the "standard" to which other blowcount values are compared. N_{60} is given as:

$$N_{60} = \frac{E_m\,C_B\,C_S\,C_R\,N}{0.60}$$

where N_{60} is the SPT N-value corrected for field procedures and apparatus; E_m is the hammer efficiency; C_B is the borehole diameter correction; C_S is the sample barrel correction; C_R is the rod length correction; and N is the raw SPT N-value recorded in the field. Skempton (1986) provides charts for estimating the appropriate values of C_B, C_S and C_R. Clayton provides an expanded listing of SPT hammer efficiencies

(E_m) in C.R.I. Clayton, 1990, SPT Energy Transmission: Theory, Measurement, and Significance: *Ground Engineering*, v. 23:10, p. 35-43.

In sands the Standard Penetration resistance, N, has been found to be influenced by the effective overburden pressure. Gibbs and Holtz (1957) presented method for determination of SPT N values corrected for a "standard effective overburden" pressure, N', so that various resistance values could be compared. N' was defined as $N' = C_N N_F$; where N_F is the unfactored SPT blowcount value, taken in the ground; and C_N is a correction factor. The standard effective overburden pressure was given as an effective stress, σ', (overburden confinement). The correction factor C_N is taken as the reciprocal of the square root of σ'.

The Gibbs and Holtz overburden correction was found lacking when applied to situations where samples were taken from near the bottom of uniform soil deposits, which exhibit higher blowcounts, because the sampler senses the stiffer material lying below. In 1986, Liao and Whitman presented the currently held overburden correction, termed $(N_1)_{60}$ (S.S.C. Liao and R.V. Whitman, 1986, Overburden Correction Factors for SPT in Sand: *Journal of Geotechnical Engineering*, A.S.C.E., v. 112:3, p. 373-377). The $(N_1)_{60}$ blowcount is given as:

$$(N_1)_{60} = N_{60} \sqrt{\frac{2000\text{ psf}}{\sigma_z'}}$$

where σ_z' is vertical effective stress where the sample was recovered.

Over the past decade, published correlations usually relate corrected Standard Penetration Test resistance $(N_1)_{60}$ with other parameters, such as relative density and angle of internal friction. These are also useful in assessments of liquefaction potential.

Drive sample disturbance

Thin wall samplers are defined as those with a wall thickness less than 2-1/2% of the diameter, such as Shelby Tubes. Drive samples less than 6.35 cm (2.5 inches) in diameter should be regarded as "disturbed" and their reported moisture and bulk density values judged accordingly. 6.35 cm (2.5 in) drive samples can also be disturbed, especially when taken in the rooted zone of natural slopes, commonly in the upper 2 meters (6.56 ft). This is because the rooted zone is typically of lower density, due to root action and creep (Burmister, 1948, op cit.). When advancing drive samples within the upper 2 m of a native slope, it is advisable to mark the Kelly bar with crayon and compare the distance advanced during the sample round with the actual thickness of the recovered soil taken from the sample barrel sleeves. A careful comparison usually shows that the sample has been densified during the sampling process, often leading to erroneous conclusions about soil strength in these upper zones.

Sample disturbance has also been described by the Area Ratio, originally described by Mohr in 1936 (Harry M. Mohr, 1936, Exploration of Soil Conditions and Sampling Operations: *International Conference of Soil Mechanics and Foundation Engineering*, Harvard Univ., v. 3:24 and a short while later by Hvorslev (H.J. Hvorslev, 1940, The Present Status of the Art of Obtaining Undisturbed Samples of Soils: *Harvard Univ. Soil Mechanics Series 14*). It was eventually standardized in M. J. Hvorslev (1949, op cit). The Area Ratio is given as:

$$A_r(\%) = \frac{(D_o)^2 - (D_i)^2}{(D_i)^2} \times 100$$

Shelby Tube Sampler

A drive sample can be considered undisturbed if the area ratio is less than or equal to 10%. In 1936 Harry Mohr developed the "Shelby Tubing" or "Thin-Wall Sampler" in response to a request made by Prof. Arthur Casagrande at Harvard. Casagrande wanted a less disturbed sample within the standard 2-1/2-inch casing size then employed in most exploratory borings around Boston. The term "Shelby Tubing" was a trade name for hard-drawn, seamless steel tubing, manufactured by the National Tube Company of Lorain, Ohio, which is available nation-wide (National Tubing was owned by US Steel). Shelby Tubes have an inside diameter of 2.0 inches and use 16 gage (0.0578 inches) to 18 gage (0.0451 inches) wall thickness tubes advancing a 36-inch section of tubing ahead of the sampler, recovering a 33-inch long sample (see Fig. 5).

The area ratio for a SPT test is 110%, while that for a thin-wall Shelby Tube is about 13.7%. Despite this, soft soils recovered in Shelby Tubes are generally assumed to be more-or-less undisturbed, while all split-spoon samplers should be regarded as disturbed samples. The dimensions of a typical Shelby Tube sampler are shown in Figure 5. Some caution should been advised in performing strength and compressibility tests, such as consolidated triaxial compression, and unconfined compression. The impact of drive sampling disturbance on laboratory indices was recognized early on, and summarized by Phil Rutledge in his early research with Arthur Casagrande at Harvard (P.C. Rutledge, 1944, Relation of Undisturbed Sampling to Laboratory Testing: *Transactions ASCE*, v. 109, p. 1155-1183). Soil behavior in lab test is greatly influenced by sampling disturbance, as illustrated in Figure 6. Nevertheless, disturbed samples may be adequate for indices tests, such as Atterberg Limits and grain size distribution necessary for proper classification of the soil type.

A great many piston and drive samplers of larger than 4-inches diameter were developed by various workers beginning in the late 1930s, in the attempt to recover less disturbed samples of compressible materials, such as the Boston Blue Clay. These alternative samplers are summarized in: M. Juul Hvorslev, 1949, *Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes*: USACOE Waterways Experiment Station, Vicksburg [reprinted by the Engineering Foundation in 1962 and 1965], 521 p.

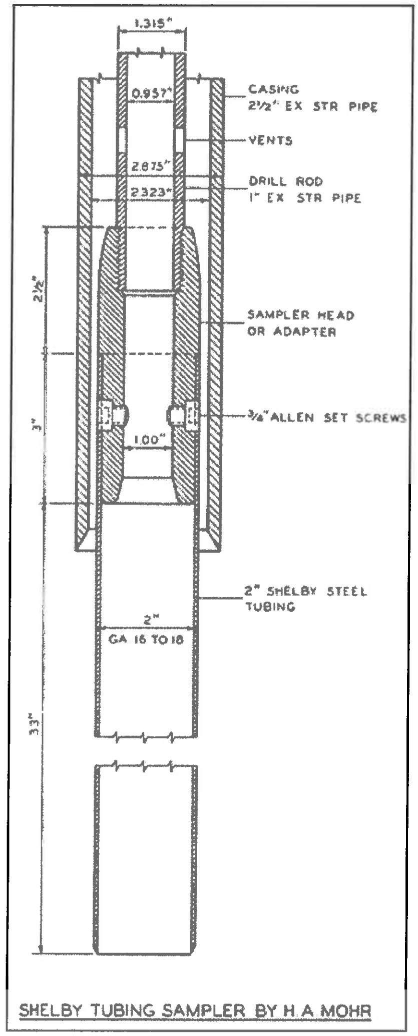


Figure 5 – *Original concept for Shelby Tube thin-wall piston sampler, as developed by Harry Mohr in 1936.*

<u>Hammer Energy Measurements</u>

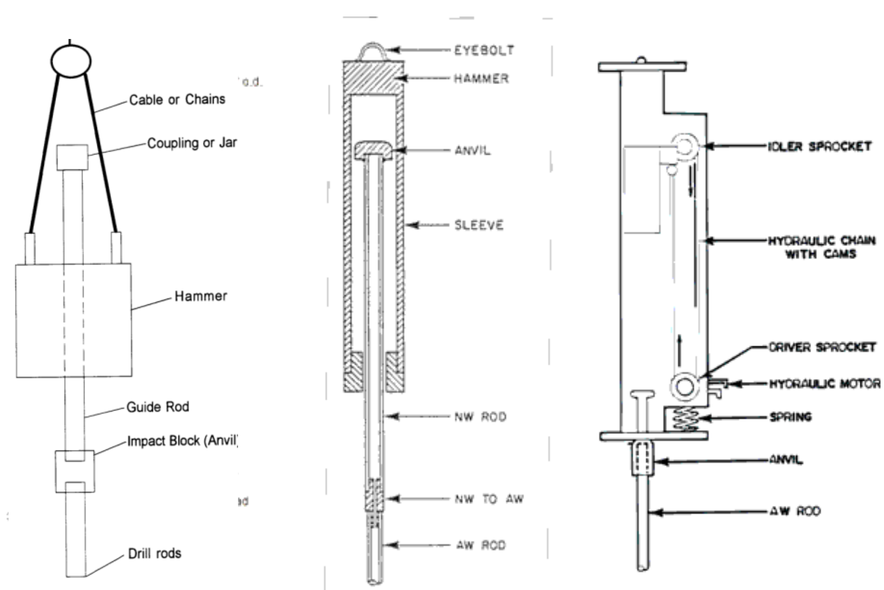
Hammer Energy Measurements

<u>Introduction</u>

Geotechnical engineers in the United States commonly use the Standard Penetration Test (SPT) N-value for correlations with friction angle (ϕ) and in analysis of foundations, retaining walls, settlement, and liquefaction. ASTM D-1586 explains the details of the SPT method, but the primary portion of the method is driving a 1-3/8″ ID, 2″ OD split spoon sampler with a 140-pound hammer falling 30 inches. Hammer blows over three 6-inch increments are recorded and the last two sets of blows are added to arrive at a number of blows per foot. Many equations used in soil mechanics that are based on SPT N-values were derived when a cathead and rope mechanism was the primary system to raise and drop the hammer.

Ideally, the hammer would deliver 350 ft-lbs of energy (140# weight dropping 30″ or 2.5′) to the anvil. However, since there is a great amount of friction in the cathead and rope system, a significant amount of energy is lost before the hammer impacts the anvil. The cathead and rope system has been documented to have efficiencies on the order of 36 to 82% (of the 350 ft-lbs). The variation in efficiency is also contributed to variability in drill rigs, operators, and other factors. It is generally accepted that N-values are based on 60% energy, as H.B. Seed suggested in 1985. We now consider $N_{60} = (ER/60\%) \times N$, where ER is the hammer efficiency and N is the uncorrected blowcount.

Today, different hammer mechanisms, such as the automatic hammer which has less energy loss, and different sized samplers (2″ ID, 2.5″ ID) are used. Combinations of different hammers and samplers produce a different energy relative to the cathead and rope and SPT sampler. Therefore, the need to measure the actual energy the delivered by the hammer system has become increasingly important in geotechnical analyses. Typical hammers are shown below.



Donut Hammer *Safety Hammer* *Automatic Hammer*

Schematic of Various SPT Hammer Arrangements

History

In the 1960s and 1970s, researchers and practitioners started noticing the wide variation in N-value results. Early studies to quantify the hammer energy included 1) calculating force using load cells and 2) measuring the speed of the hammer drop and integrating force-time relationships. These approaches were acceptable, but the technology available made the test very time consuming and expensive to perform. This led to the instrumented rod systems used today, which offers more efficient and less expensive data gathering.

Instrumented Rod

When the hammer strikes the anvil on the top of the rod string, a wave is sent through the rod to the sampler and back up. Force and velocity measurements are taken immediately below the hammer using an instrumented section of drill rod (see below). Depending on the type of drill rod used for the borehole, the instrumented rod is available in different diameter and wall thickness.

The instrumented rod has two full bridge foil resistance strain gage circuits (end of the blue wire) and two piezoresistive accelerometers (silver blocks at left) mounted approximately in the center of each rod. Analog signals from the strain gages and accelerometers are conditioned, digitized, stored, and processed with the data recording equipment. Selected output from the recording equipment for each impact includes the maximum calculated rod top force, maximum rod top velocity, energy transfer calculated by two methods, and the hammer operating rate.

Data Collection

Results of hammer energy are more reliable if the data is collected at depths greater than about 25 feet, if the blows in the data group are between 5 and 50, and at least 5 sets of data is collecting in each boring. Data sets that don't meet these criteria can still be collected; however, they will likely not be as reliable.

Many things in the field can lead to unreliable data, such as loose rod connections, a instrumented section that is different from the rod string, or loose accelerometers. Loose connections in the rod string can modify the wave travelling through the rod. If the rod string is a different size (diameter and wall thickness) from the instrumented rod, the wave could also be modified. A loose accelerometer will not collect the velocity data correctly, possibly not at all. The operator should check for loose rod connections and accelerometers, and verify the rods are similar.

The acceleration measurements are often demanding in the SPT environment, because of high frequency and high acceleration motion components. An experienced operator, therefore, has to evaluate the quality of this data before final conclusions are drawn from the numerical results calculated by the SPT Analyzer. Once the data for each set of blows is obtained, it is verified and then stored in the SPT Analyzer and finally downloaded onto a PC once back in the office.

Data Reduction Methods

Originally, ASTM D4633 was created to standardize hammer energy measurement and use a "force squared" method, which integrates the square of the force over time. The latest edition of ASTM D4633 (05) cites the EMX method, which integrates the force and velocity products over time. The force squared method is considered to be theoretically incorrect and has been shown to be not as precise. Therefore, most practitioners agree that the EMX method is more reliable:

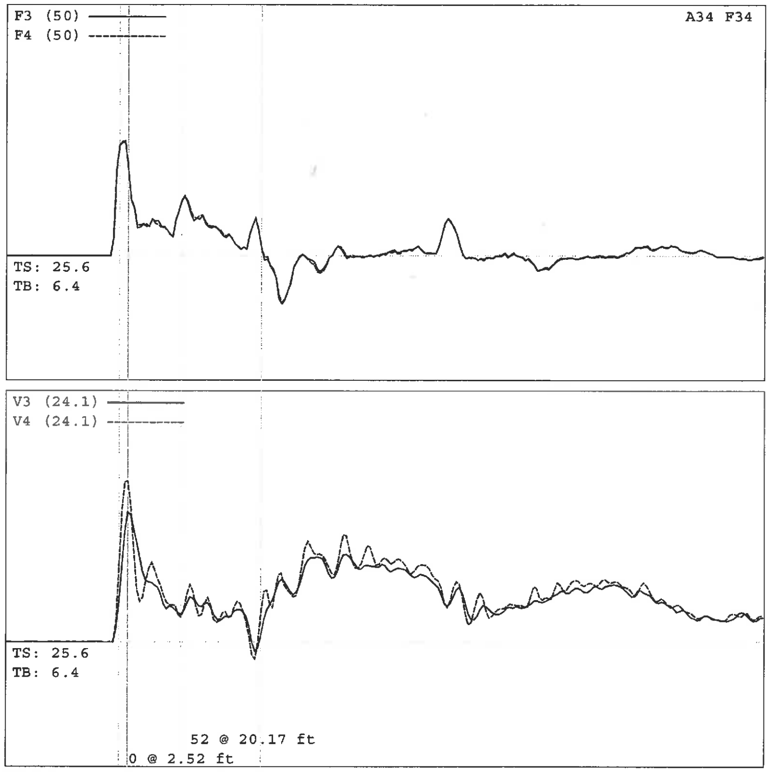
$$EMX = \int_a^b F(t)\ v(t)dt$$

"F" and "v" represent force and velocity. The time "a" corresponds to the start of the record which is when the energy transfer begins and "b" is the time at which energy transferred to the rod reaches a maximum value which is also the time of maximum pile top displacement and zero velocity.

Inside the energy measurement software, the individual data points for force and strain are reduced to force and velocity curves for each blow. The force and velocity curves for each blow should be reviewed and adjusted if the coupling curves are not proportional.

Taber — Pile Driving Analyzer ®

CHEVERON PIER — B-1



Project Information

PROJECT: DEEP FOUNDATION
PILE NAME: B-1
DESCR: SMT;ROPE CATHEAD RED;
OPERATOR: TIM DARCY
FILE: B-1small
8/13/2008 1:26:28 PM
Blow Number 4

Pile Properties

LE	38.00	ft
AR	1.16	in^2
EM	30000	ksi
SP	0.492	k/ft3
WS	16807.7	f/s
EA/C	2.1	ksec/ft
2L/C	4.52	ms
JC	0.00	[]
LP	38.00	ft

Quantity Results

FMX	23	kips
VMX	14.0	f/s
EMX	0.216	k-ft
ETR	61.8	(%)
BPM	29.2	bpm
RMX		kips
QNV	0.00	[]
QNV	0.00	[]
QNV	0.00	[]

Sensors

F3: [F1] 218.51 (1)
F4: [F2] 219.42 (1)
A3: [A1] 335 mv/5000g's (1)
A4: [A2] 340 mv/5000g's (1)
CLIP: OK
F3/F4: OK 1.02
V3/V4: OK 0.81

Version 2008.098.031

Once the force and velocity data is verified, the integration is performed to obtain final energy transferred for each blow. The efficiency, or energy transfer ratio (ETR), can be obtained by dividing the EMX by 350 ft-lbs.

The energy for each set of blows is averaged and the average represents the energy for that set. Similarly, the average of all the sets of blows within a boring represents the average energy for that boring.

Presenting Data

Once the data is summarized and statistical values are generated, it can be presented in summary form to the design engineer.

Summary of Field Results
SPT Energy Measurements

Date	10/2/08
Drill Rig	CME-55 Track
Hammer Type	Automatic
SPT Analyzer Operator	Tim d'Arcy
Taber Project No.	2D2/308/148

Sampler Depth (ft)	Number of Blows Analyzed	Average Energy Transfer* ft-lbs	Average Transfer Efficiency* %	Hammer Operating Rate bpm
3.5-5	32	329	94	37
10-11.5	36	340	97	36
15-16.5	32	335	96	36
20-21.5	27	355	101	37
25-26.5	54	351	100	39
30-31.5	111	350	100	36
35-36.5	92	345	99	37
40-41.5	104	354	101	35
45-46.5	105	356	102	39
	Ave	346	99	37
	Max	356	102	39
	Min	329	94	35
	Std Dev	10	3	1

Note:
Transfer Efficiency is based on 350 ft-lbs, 140# hammer with a 30-inch drop.

In-Situ Testing - CPT

HISTORY THE CONE PENETRATION TEST

It is said that early man pushed sticks into the earth to determine if a chosen location was stable enough to support his dwelling.

- 1931 - first true static cone penetrometer was developed whereby contact between the push rods and soil was avoided (Netherlands)
- 1947 - mantel cone developed whereby tip was protected by a jacket (Netherlands)
- 1964 - Fugro, Netherlands had first commercial use of the electric penetrometer, tip only
- 1968 - an electric penetrometer was developed in Australia that incorporated a friction sleeve to measure frictional resistance
- 1969 - addition of friction sleeve to the mechanical penetrometer to enable measurement of lateral friction.
- 1975 - introduction of pore pressure measurement at the tip of the cone penetrometer
- 1975 – ASTM D 3441 was developed to standardize the mechanical CPT apparatus and procedure.
- mid 1970s - introduction of the electronic cone which amplified signals in the cone and significantly reduced the noise and thermocouple effects on data quality.
- 1988 - the seismic cone was introduced
- 1995 – ASTM D5778 was developed to standardize the electronic CPT apparatus and procedure

Since 1988 other devices have been incorporated into cone penetrometers (e.g. resistivity modules, lateral stress cell, vibratory cone, vision cone).

COMPONENTS OF A MODERN CONE PENETRATION TESTING SYSTEM

- Rig
  - Hydraulic rams mounted in:
    - Load frame
    - Trailer
    - Light truck 7 ton±
    - Heavy truck 20 ton±
    - Crawler

- Data Acquisition System
  - Computer with A/D circuit
  - Computer (no A/D required) for digital cone

- Rods
  - Usually 1 meter long, 10 cm^2 cross section
  - High strength but malleable and tough

- Cable
  - 10-12 strand cables are common
  - Should be somewhat tough and flexible

- Cone Penetrometer
  - 10 ton or 15 ton for general profiling
  - 10 ton is standard cone on which most correlations are based
  - 2.5 ton-5 ton cone for detailed testing of soft material

CHARACTERISTICS OF MECHANICAL, ELECTRICAL, ELECTRONIC AND DIGITAL CONES

- Mechanical Cone
  - Friction and tip measurements conducted at different stages of test.
  - Data recorded by hand.
  - Data processing quite laborious

- Electric Cone
  - Equipment is less expensive than electronic or digital cones.
  - Signal is transmitted through cable to computer at the microvolt, μV, level.
  - Measurements, especially friction measurements, can be significantly affected by common environmental noise (pore lines, microwave transmissions) and resistance losses.

- Electronic_Cone
  - Equipment is more expensive than electronic cones.
  - Signal is amplified from the microvolts, μV, to the volt, V, before being transmitted through the cable.
  - Signal significantly less affected by common environmental noise.

- Digital Cone
  - More expensive than electronic cone equipment.
  - Data is digitized before entering cable.
  - Data is unchanged by noise or thermal effects in the cable.

PRINCIPAL COMPONENTS OF A CONE PENETROMETER

- Tip
  - $q_c - q_t$
  - most fundamental cone measurement
  - must make adjustments in q_c to account for pore pressure effects
  - Standard 10 ton cone
    - $10 cm^2$ (3.5 cm Ø or 1.26")
    - 60° apex angle

- Friction Sleeve
  - $f_s - f_t$
  - if not equal end area sleeve adjustments must be made for pore pressure effects
  - $f_c = f_t$ for equal end area cone
  - f_t very difficult measurement to make accurately
    - one order of magnitude difference between force on tip and force on friction sleeve
    - Subtraction cones incorporate 2 strain elements. One is subjected to tip force only, one is subjected to combined tip and friction forces. Tip stress is subtracted from combined tip and friction stress to get friction stress. Small error in tip measurement can cause rather large error in friction determination.
    - Errors also possible in the subtraction channel of cone.
    - Electrical and mechanical cross talk is possible with a poorly designed cone.
    - Dirt in joints (friction sleeve not moving freely).
    - Damage (friction sleeve not moving freely)

- Inclinometer
  - Used to prevent equipment damage, by controlling the radius of curvature of the CPT rods.
  - Provides a means of correcting depth. For most work inclination are kept within practical limits and depth correction not necessary.

- Pore Pressure Element
  - u_1 – Element on the tip or face of cone.
    - Excess pore pressures induced by compression of the soil structure dominates.
    - Excursions in values are relatively large.
    - Best position to define stratigraphic changes.

  - u_2 – Element behind the tip.
    - Soil shear during penetration designates the response.
    - Best position to correct tip and sleeve for pore pressure effects.
    - Negative pore pressures can be observed in stiff, overconsolidated clays

  - u_3 – Element located anywhere behind the friction sleeve.
    - Pore pressures not very different from u_2 position.

- Thermistor
  - Measures (dynamic) cone temperature during testing.
  - Can measure ambient soil and groundwater temperatures if probe advance is stopped and temperature allowed to equilibrate.

- Geophone
  - Determine shear wave velocity profiles (requires wave source, generated from ground surface).
  - Like down hole seismic from bore hole.
  - To minimize any systematic errors, all subsequent (deeper) shear wave arrival times are referenced to the arrival times from the shallowest test.

- Resistivity Module
  - determine profile of electrical conductance/resistance in soil column

CPT DATA INTERPRETATION & APPLICATIONS

- Stratagraphic Interpretation
  - CPT is better suited for layer definition than borings.
  - Excellent tool to assess variations in material type and stiffness in a soil profile.
  - CPTs should be performed first. The strategically plan boring and sampling program based on CPT results.
  - Tip resistance and friction ratio are the primary measurements used to evaluate material type.
  - Tip resistance and pore pressure parameter measurements is an additional method of classifying soils below the water table.
  - Pore pressure (dynamic) measurements can help indicate the presence and thickness of very thin layers that tip resistance and friction may not detect.
    - Influence zone 1-10 diameters in front of the cone tip.
    - Friction sleeve averages unit friction over its length (5.26”).

- Pile Capacity
  - Pile Capacity determination is one of the earliest applications of cone penetration testing.
  - For many years pile design using CPT data has been generally successful.
  - Since the estimation of pile capacity is complicated by the many varieties of pile types and installation procedures, pile design using CPT data is highly empirical.
  - Many methods exist for evaluating pile capacity using CPT data.

- Shallow Foundation Design
  - Direct methods based on observed field experience have been developed for design of shallow foundations on sand. These methods when applied in similar situations can produce reliable results.
  - Direct methods have a particular advantage in estimating settlement of footings on granular soils where use of parameters like relative density can be misleading.
  - Bearing capacity of narrow footing on sand can be estimated using bearing capacity calculation based on friction angle, Φ, obtained from CPT data interpretations.
  - Stability of shallow foundations on clay is assessed from bearing capacity calculations using undrained shear strength, Su, obtained from CPT data interpretations.
  - Compressibility of clay is not reliably estimated from CPT data.

- Rate of Consolidation and Relative Permeability of Clays
  - Rate of consolidation and relative permeability of clays can be estimated from pore pressure dissipation curves generated with the piezo-cone. These estimates for any particular clay should be adjusted using local experience.

- Phreatic Surface
  - Phreatic surface determination from 100% pore pressure dissipation curves

- Hydraulic Gradients
  - Hydraulic gradients determined from 100% pore pressure dissipation curves conducted at various depths in a soil profile.

Horizontal/Radial Coefficient of Permeability

- K_n or K_r for wick/sand drain design from pore pressure dissipation curves

- Relative Blow Count
  - Estimates of N_{60} for input into blow count base design material from CPT data interpretations.

- Liquefaction Potential Assessment
  - Liquefaction potential estimates based on q_t and f_{sr} as proposed by Seed, et al., 2003 (recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework) and Idriss & Boulanger 2004 (Semi-Empirical Procedures for Evaluation Liquefaction Potential during Earthquakes).
  - Liquefaction potential estimates based on V_s profiles from seismic CPT data interpretation, as proposed by, Seed et al., 2003 and Andrus and Stokoe, 2003.

- Compaction Control
  - CPT is commonly used for quality control of deep compaction projects.
  - The CPT has been found to be one of the best methods to monitor and document the effect of deep compaction due to the continuous, reliable and repeatable nature of the data.

- G_{max} determination
  - G_{max} ($G_{max} = \rho V_s^2$) and V_s are required properties in earthquake site response and soil structure interaction analysis. V_s can be assessed from some seismic CPT results.

- Resistivity
  - Refine stratigraphic interpretation.
  - Determine the limits of contaminant plus if contaminant changes the electrical properties of the soil.
  - Has been used for many years to estimate in situ porosity and density of soils.

- An important input parameter for evaluation of corrosive potential of soil.

- Temperature
  - Estimate the depth of water table in some soil types.
  - Correct CPT data for temperature effects (research).
  - Determine ambient groundwater/soil temperatures.
  - Track contaminants that generate heat through biological/chemical activity.
  - Locate frozen soil zones.

Electronic Cone Penetrometer Test (CPT)

Cone penetrometer test (CPT) provides rapid and continuous (in situ) soil profile. CPT measures cone tip resistance, sleeve friction, and friction ratio and has the ability to make dynamic pore pressure measurements. With the addition of a seismic cone and geophones, CPT borings can measure compression and shear waves. ASTM D 5778 can be referenced for standard procedures for the electronic CPT.

A few applications:

- Commonly used to verify soil improvement techniques such as soil-mixing, deep dynamic compaction, stone columns, etc.
- The combination of CPT soundings and drilled borings with SPT samples is the best evaluation method for liquefaction studies.
- Seismic studies using seismic cones to evaluate foundation and structure behavior to ground shaking loads.

Method:

The cone tip is attached to a string of steel rods and pushed into the ground at a rate of 2 cm/sec. Continuous measurements are taken from transducers located in the cone and sent to a computer through wires located inside the steel rods.

Advantages:

- CPT gives rapid and continuous strength profile of a soil deposit. Considered to be the best technique for delineation of stratigraphy.
- CPT is much less operator dependent than other in situ tests and the test sequence is simple. As a result, the data from a CPT are reproducible. (Possible exception to this is pore pressure measurements.)
- The soil parameters are measured in place under the actual in situ stress conditions.
- CPT is very well suited to the design of vertically loaded piles because of the close analogy of loading.
- CPT has been used for a long time and a number of design rules are well documented.
- Dimensions of the penetrometer tip have been standardized (ASTM).
- CPT is fast and cost effective compared to drilling and laboratory testing.

Disadvantages:

- The penetration depth is limited in the stronger soils. It cannot be used reliably in dense gravel, cobbles, boulders, and rock. Cemented soils may be interpreted as gravel.
- CPT methods cannot provide some of the material characteristics which can only be obtained by use of other in situ tests or borings, sampling, and laboratory testing.
- Performing a CPT requires skilled operators with a knowledge of electronics and hydraulics, in a case of equipment failure.
- Methods used to obtain the soil parameters are mainly based on correlation instead of theory.

Electronic or Electric Cone?

Electronic Cone has <u>internal electronic power amplification</u> and <u>regulation</u> mounted directly in the cone. The amplifier minimizes the effect of cable resistance on the measurements. Also, cone circuitry is such that crosstalk errors between the tip and friction channels are minimized or eliminated. Electronic CPT results are relatively operator independent.

Electric Cone is older technology where cone measurements are subject to noise and resistance losses as they travel in the cables from the cone to the computer thus affecting the data. The operator must be a highly skilled technician with a good understanding of an electric system which incorporates the cable as part of the bridge, Proper impedance matching and bridge balancing is crucial to obtain good CPT results.

Does it make a difference? ***Yes!***

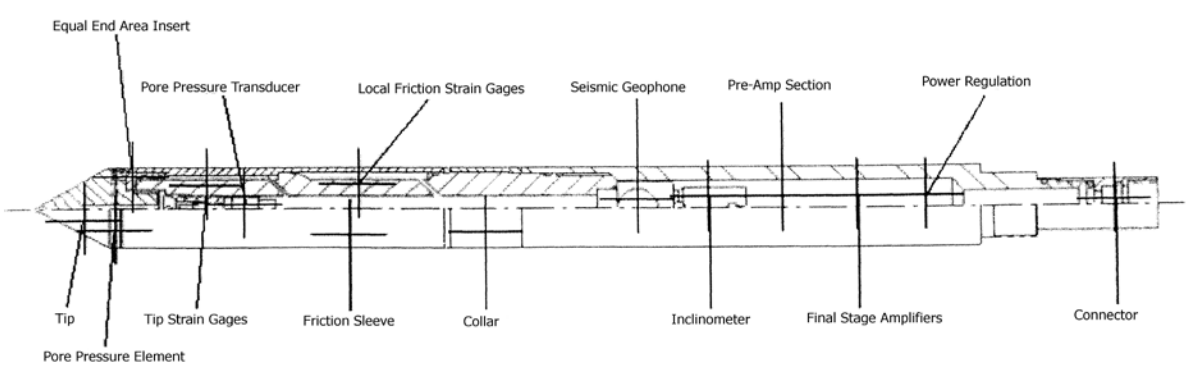
Cone measurements are small (microvolts) and in a frequency range easily influenced by noise common and present in most areas (e.g., power lines, microwave transmissions, etc.) where in situ testing takes place.

An electronic cone circuitry allows <u>greater sensitivity</u> and accuracy during measurements. This is especially important in soft soils where resulting data from -

Electric cone - may show a clay
Electronic cone (more accurate) - may show a potentially liquefiable silt

Bottom Line

An Electronic cone is more costly but you receive higher quality data
Electic cone may be cheaper but you typically get unreliable or questionable data



Ref. Bowles, Foundation Analysis & Design (modified) - McGraw Hill

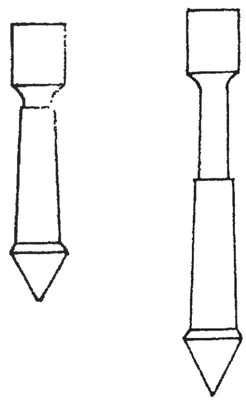


FIG. 1.1. Dutch Mantle Cone

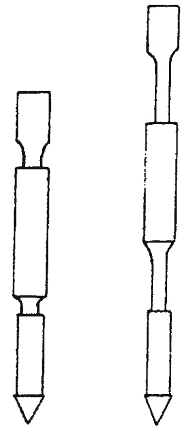


FIG. 1.4. Mantle Mechanical Cone Fitted with a Side Friction Sleeve of the Begemann Type

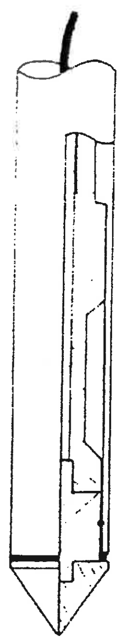


FIG. 1.2. Electric Penetrometer Tip (q_c)

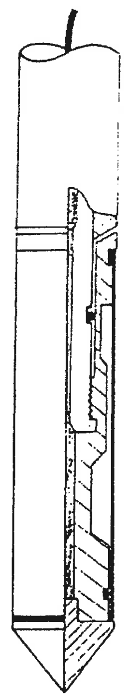


FIG. 1.3. Electric Penetrometer Tip ($q_c + f_s$)

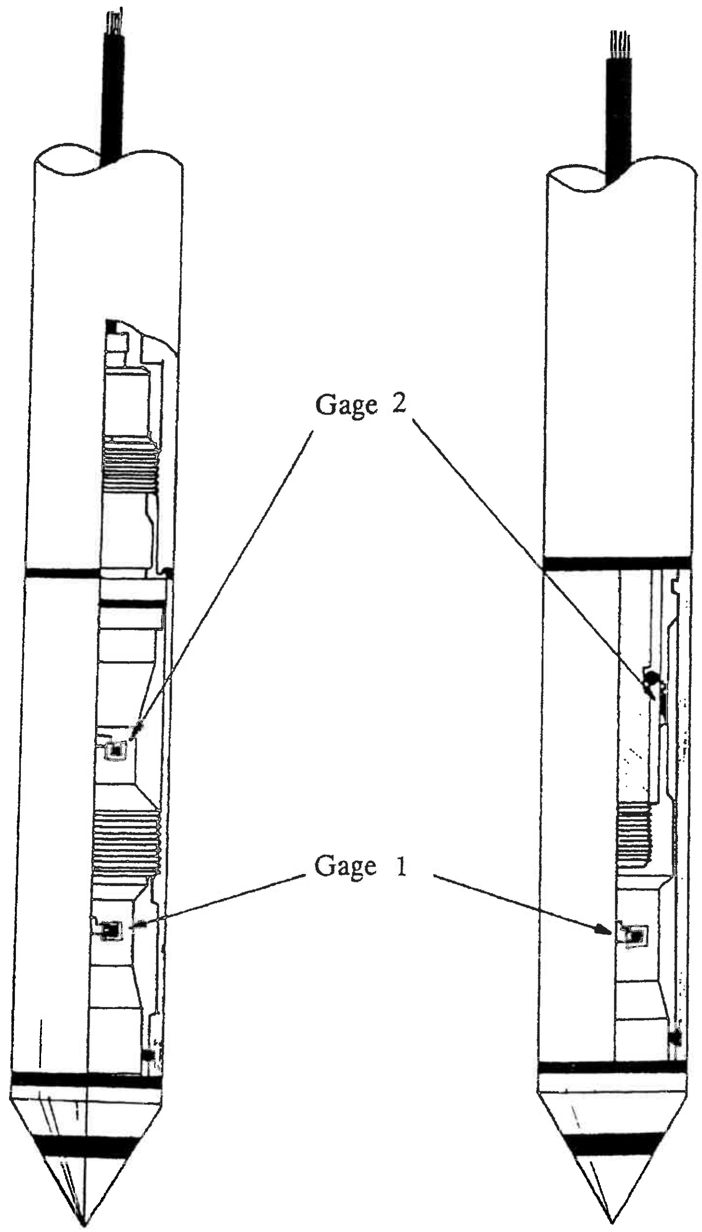


FIG. 2.7. Subtracting Penetrometer

FIG. 2.8. Tension Penetrometer

Electronic Cone Penetrometer Test

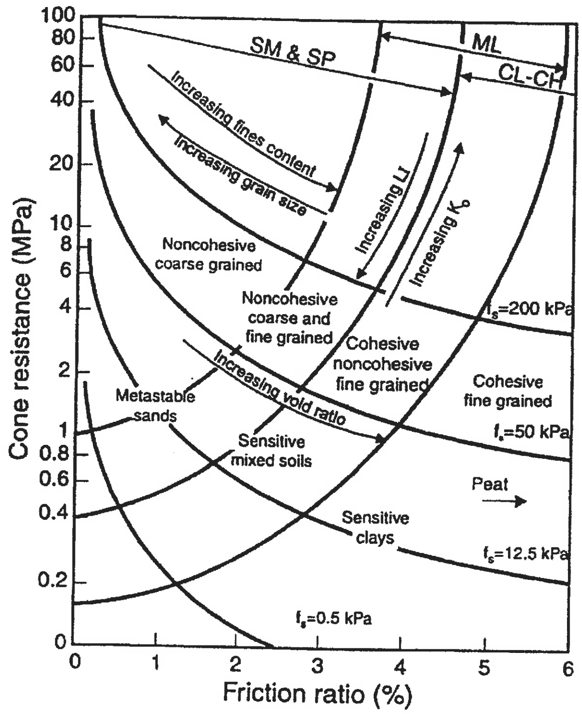


Figure 5.6 CPT soil behaviour type classification chart by Douglas and Olsen (1981).

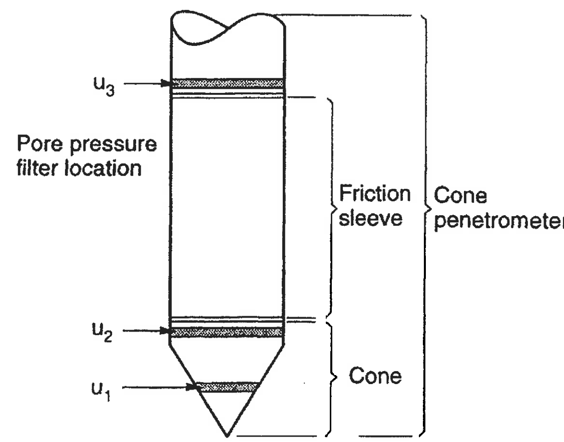
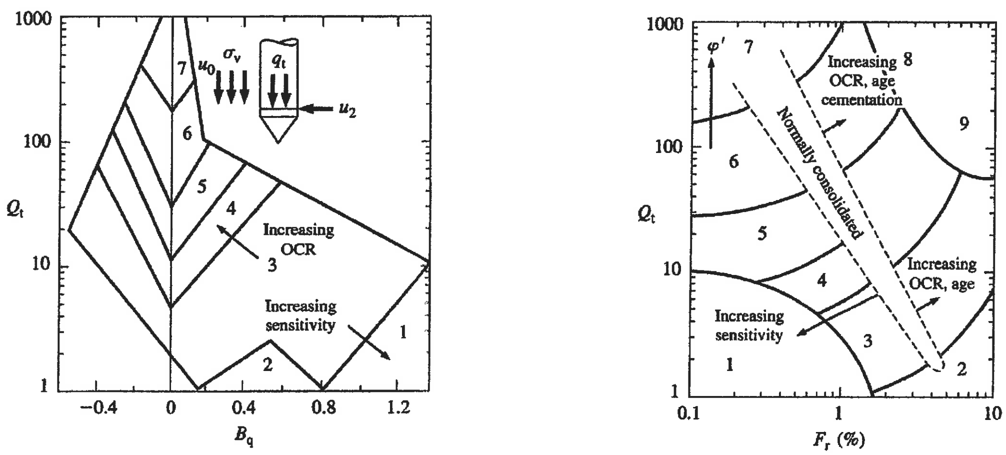


Figure 1.1 Terminology for cone penetrometers.



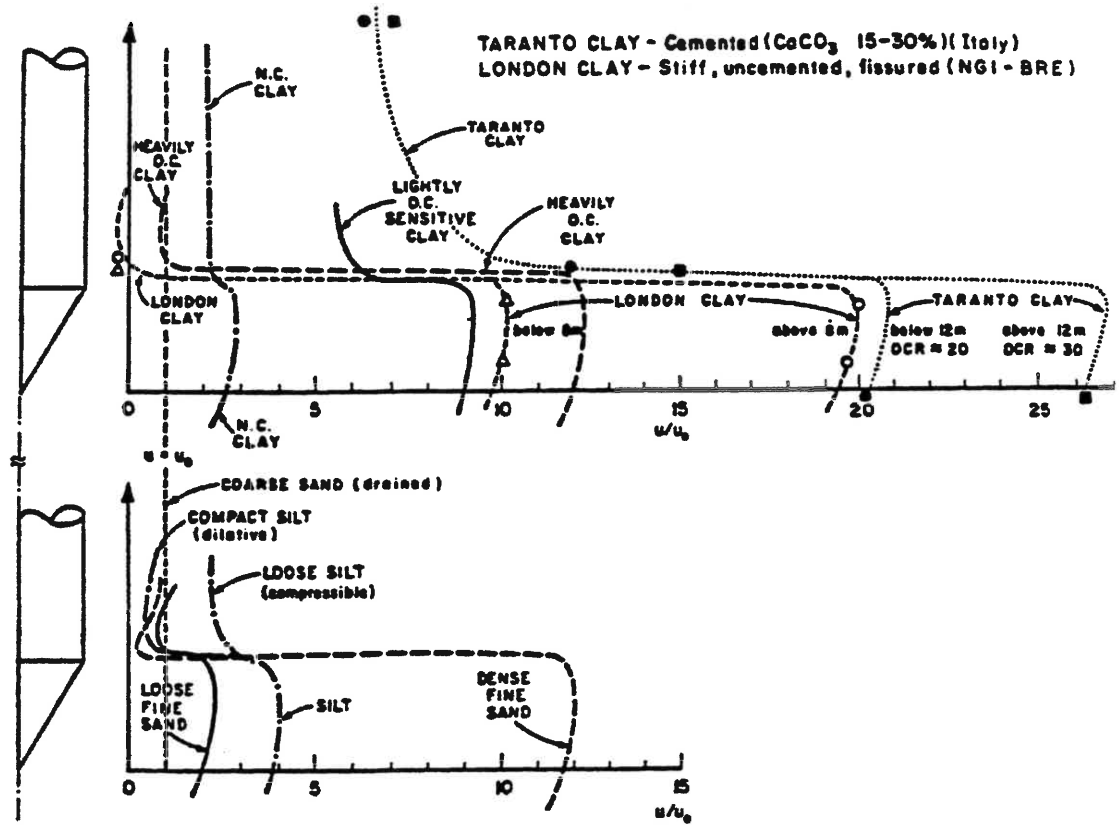
$$Q_t = \frac{q_t - \sigma_v}{\sigma'_v} \qquad B_q = \frac{u_2 - u_0}{q_t - \sigma_v} \qquad F_r = \frac{f_s}{q_t - \sigma_v} \times 100\%$$

Zone	Soil behavior type	Zone	Soil behavior type	Zone	Soil behavior type
1	Sensitive, fine grained	4	Silt mixtures; clayey silt to silty clay	7	Gravelly sand to sand
2	Organic soils; peats	5	Sand mixtures; silty sand to sand silty	8	Very stiff sand to clayey sand
3	Clays; clay to silty clay	6	Sands; clean sands to silty sands	9	Very stiff fine grained

Figure 5.7 Proposed soil behaviour type classification system from CPTU data (after Robertson *et al.*, 1986).

Ref. Lunne, T., Robertson, P.K. & Powell, J.J.M., Cone Penetration Testing – Blackie Academic & Professional

Ref. Salgado R., "The Engineering of Foundations" pg 308



Conceptual Pore Pressure Distribution in Saturated Soil During CPT Based on Field Measurements (After Robertson et al, 1986)

Seismic Cone

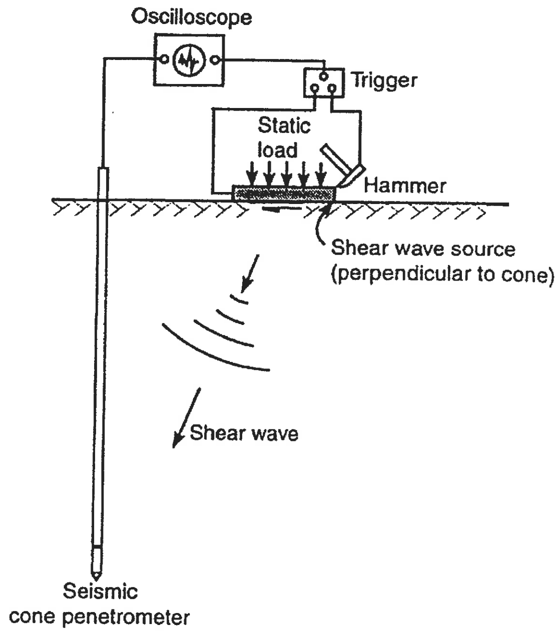


Figure 7.13 Principles of the seismic cone survey technique (Campanella *et al.*, 1986).

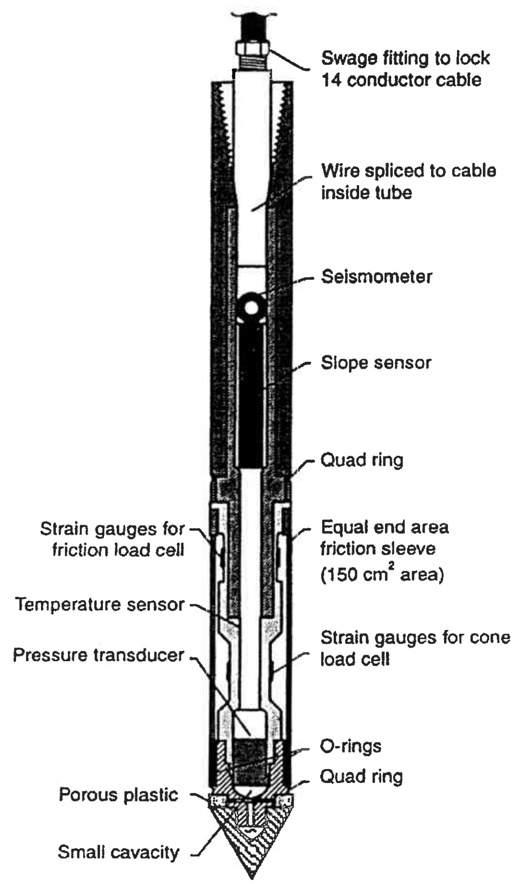


Figure 7.12 The UBC seismic cone (Campanella *et al.*, 1986).

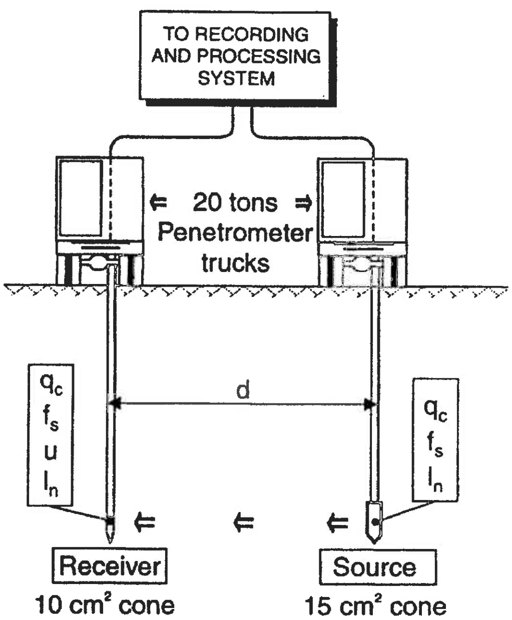
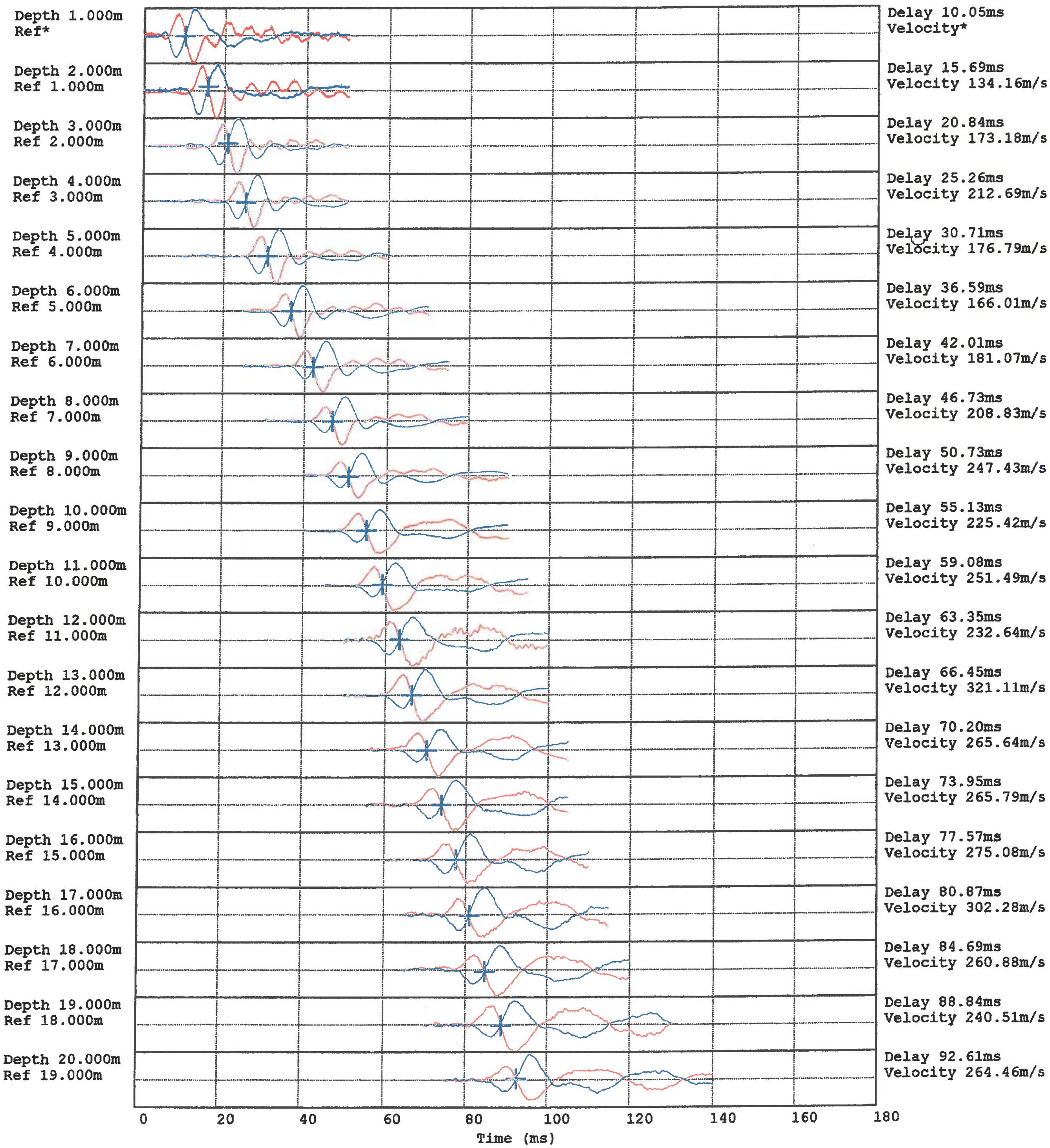


Figure 7.15 Crosshole seismic piezocone tests run by Baldi *et al.* (1988).

Ref. Lunne, T., Robertson, P.K. & Powell, J.J.M., Cone Penetration Testing - Blackie Academic & Professional

Taber In-Situ Testing



Hammer to Rod String Distance 1.25(m)
* = Not Determined

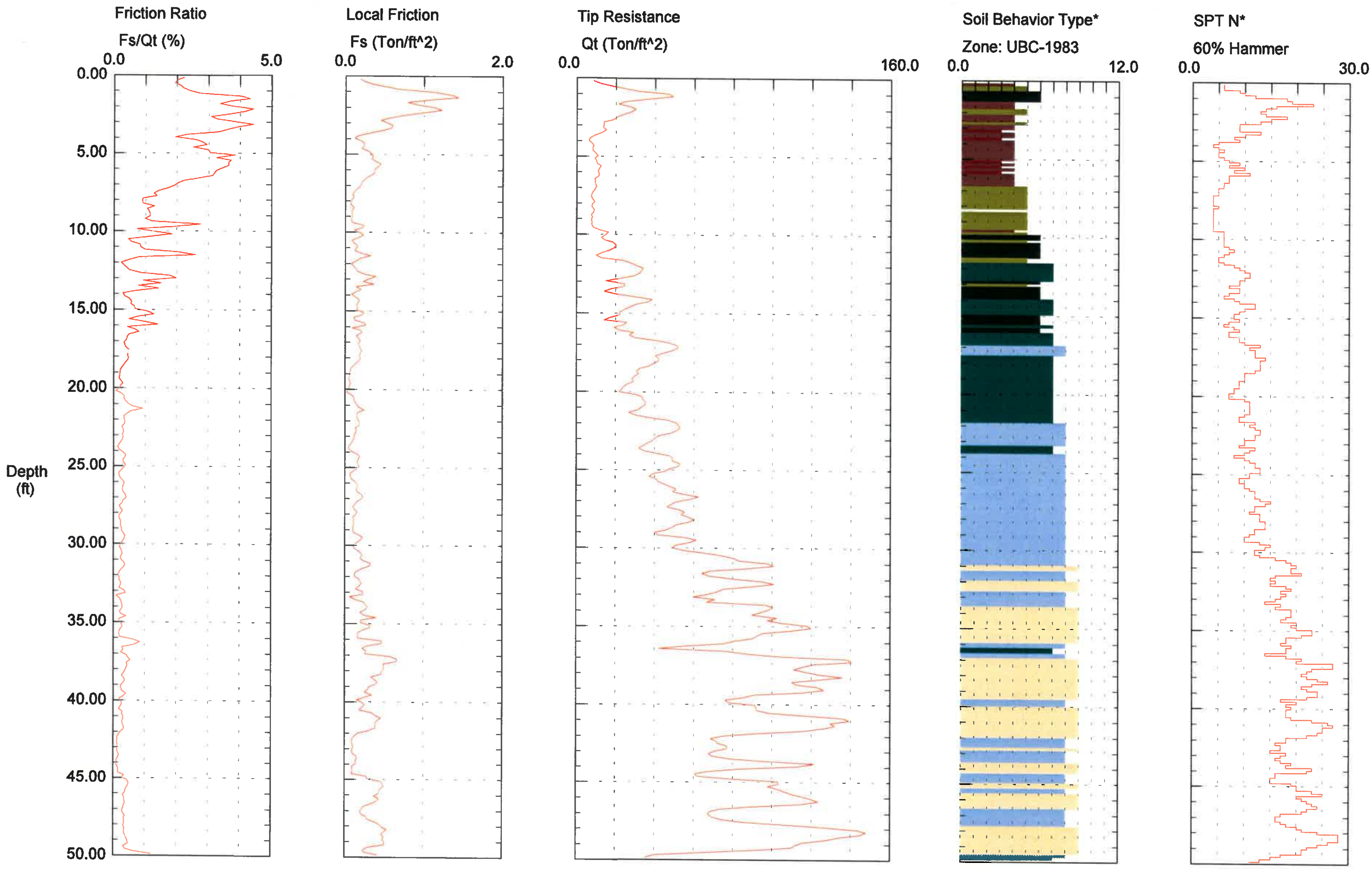
CPT-1

*Soil behavior type and SPT based on data from UBC-1983

Taber In-Situ Testing

Operator:
Sounding:
Cone Used: HO736TC

CPT Date/Time:
Location: CPT-2
Job Number:



Maximum Depth = 50.36 feet

Depth Increment = 0.16 feet

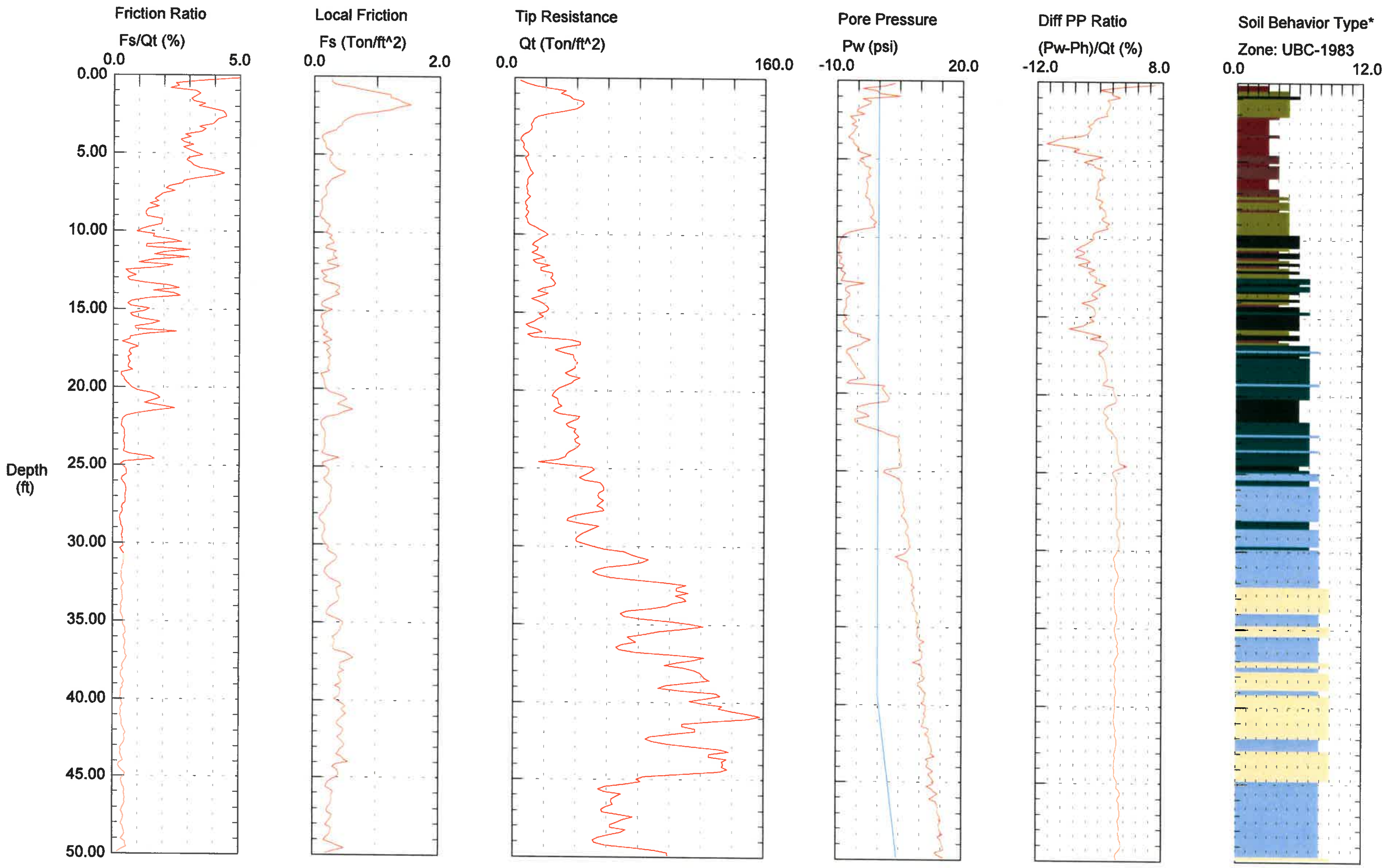
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

Taber In-Situ Testing

Operator:
Sounding:
Cone Used: HO736TC

CPT Date/Time:
Location: CPT-1
Job Number: .



Maximum Depth = 50.52 feet

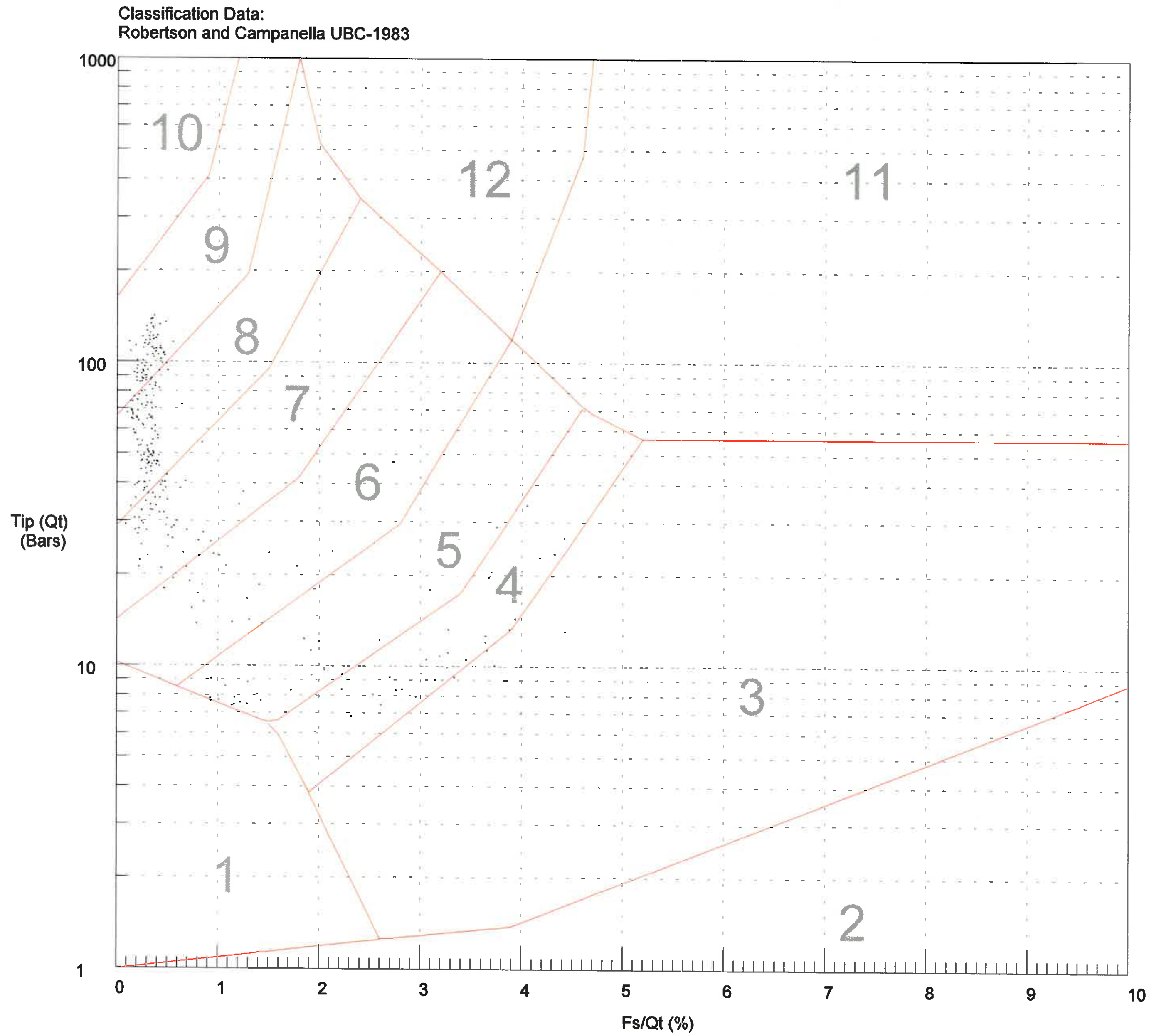
Depth Increment = 0.16 feet

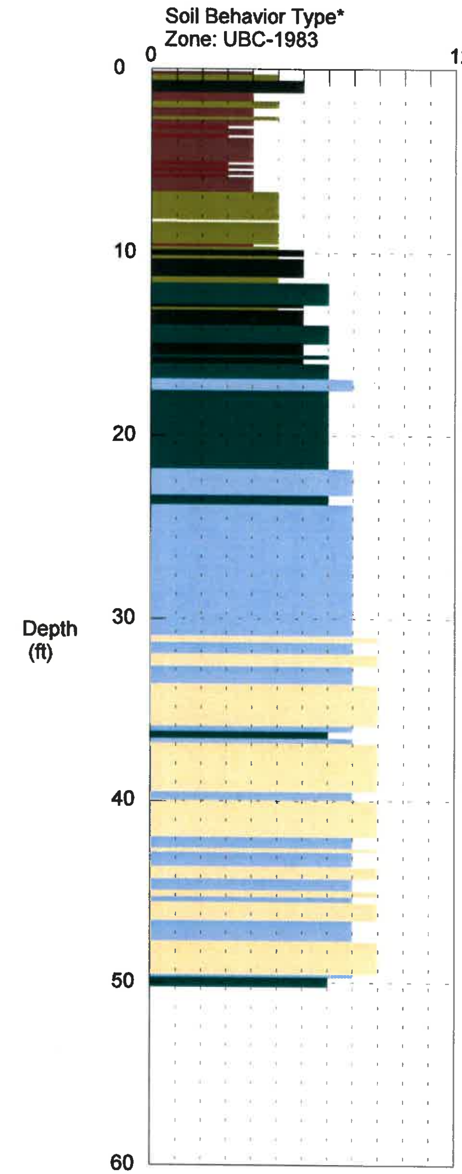
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Taber In-Situ Testing

Operator:
Sounding:
Cone Used: HO736TC

CPT Date/Time:
Location: CPT-2
Job Number:





1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

In-Situ Testing - DMT

FLAT DILATOMETER TEST (DMT)

Flat dilatometer test (DMT) originally developed by Marchetti and introduced in 1980, provides rapid and continuous (in situ) soil profile. DMT measures horizontal stress index, material index, and dilatometer modulus. Interpretation of these results provide a variety of soil parameters including undrained shear strength/friction angle, preconsolidation pressure, in situ lateral/vertical stress ratio, overconsolidation ratio, vertical effective stress and 1-dimensional consolidation modulus. The DMT procedure has been standardized in ASTM D 6635.

Best applications:

- M and C_u profiles
- Estimating settlements
- Monitoring soil improvement
- Recognizing soil type
- Distinguishing freely-draining layers from non f.d.
- Locating active/quiescent slip surfaces in clay slopes

Useful information also on:

- OCR and K_o in clay
- Coefficient of consolidation
- P-y curves for laterally loaded piles
- Sand liquefiability
- Friction angle in sand
- (K_o in sand + OCR)

Method:

Similar to CPT and uses the same rig, the Marchetti dilatometer plate, attached to a series of rods is pushed into the ground to desired depth. The operator then uses gas pressure to expand the flexible steel membrane on the plate horizontally into the soil. Two pressures are recorded; the pressure on the membrane before expansion (membrane lift-off pressure) and the pressure required to expand the membrane 1.1mm into the soil. The membrane is then deflated and the pressure behind the membrane when it reseats onto the probe is sometimes recorded and can be related to excess pore pressure (Schmertmann, 1986).

Advantages:

- Best applications are the design of horizontally loaded piles, and settlement of shallow foundations because DMT provides a modulus.
- DMT equipment is mechanically very simple, easy to repair, and fully portable.

"Practical" : Quick, simple, economical, reproducible,
Variety of insertion equipment

A. DMT is fast, economical, and easy to perform. Typical test sequence requires about 1-2 minutes! Eighty to 120 feet of DMT per day is usual.
B. DMT data is highly reproducible and test results are much less operator dependent than with some other in situ tests.
C. Soil parameters are measured in situ, which reduces the delay and costs of laboratory testing.

"Conceptual"

- DMT a two-parameter test, one related to stress history (dominating soil behavior).
- Flat blade ≈ avoids arching typical of cylindr. Probes
- M and K_D highly sensitive even to slight modifications
- Well balanced wealth of data, besides mechanical, determines U_o in sand and various flow properties.

SETTLEMENT CALCULATIONS

- Lower insertion distortion
- The modulus obtained by expanding a membrane (a mini load test) more closely correlated to in-situ soil modulus than a penetration resistance.

M_{dmt} is the only modulus taking routinely into account possible high σ_h (also felt) that reduce considerably soil compressibility (Massarsch, Delhi 1994).

Disadvantages:

- The plate penetration disturbs the soils.
- DMT cannot be used in cobbles, boulders and rocks. Penetration depth is limited in the strongest soils where the blade and membrane can be damaged.
- Performing a DMT requires skilled operator.
- Methods used to obtain the soil parameters are mainly based on correlations (direct methods) instead of theory. Therefore, correlations can be considered reliable only under conditions similar to those from which the correlations were derived. Under differing conditions correlations should be verified on the basis of local experience.

Overview

- The flat dilatometer is a blade shaped probe having a thin circular steel membrane mounted flush on one face.
- Results of DMT tests are mostly used to obtain information on soil stratigraphy, in situ state of stress, deformation properties, shear strength, homogeneity, voids or cavities and depth to firm layers.
- The test consists on inserting vertically into the soil a blade-shaped steel probe with a thin expandable circular steel membrane mounted flush on one face and determining, at selected depths or in a semi-continuous manner, the contact pressure exerted by the soil against the membrane when the membrane is flush with the blade and subsequently the pressure exerted when the central displacement of the membrane reaches 1.10 mm.
- The DMT test is most appropriate in clays, silts and sands where all particles are small compared to the size of the membrane.
- A standard test procedure has been adopted by ASTM (Designation: D 6635-01)

Equipment :

The main parts of a dilatometer (see illustration on page 82-83):

- Dilatometer blade or dilatometer probe: The blade-shaped steel probe that is inserted into the soil to run a DMT test.
- Membrane: The circular steel membrane that is mounted flush on one face of the blade and is expanded when applying a gas pressure at its back.
- Switch mechanism: The apparatus housed inside the blade, behind the membrane, which activates and disconnects an electric circuit which in turn respectively sets off and on an audio and/or visual signal when the membrane expands and reaches two preset deflections equal to 0.05 mm (lift-off) and 1.10mm respectively.
- Pneumatic-electric biaxial cable: The cable that connects the control unit to the blade, delivers gas pressure at the back of the membrane, and provides electric continuity between the control unit and the switch mechanism.

- Control and calibration unit: A set of suitable devices capable of supplying gas pressure to the back of the membrane and measuring the pressure when the switch mechanism activates and disconnects the electric contact behind the membrane.
- Ground cable: A cable connecting the control unit to the ground.
- Pressure source: A pressurized gas tank filled with any dry nonflammable and non-corrosive gas.

Testing procedure:

Overview of the DMT:

- Membrane calibration: The procedure to determine the membrane calibration pressures equal to the suction and the pressure that must be applied in air to the back of the membrane to retract its center to 0.05 mm or to expand it 1.10 mm respectively.
- Dilatometer profiling: The execution of a sequence of dilatometer tests from the same station at ground lever along a vertical direction at closely spaced intervals with depth increments ranging between 150 mm (6 inches) and 300 mm (12 inches).

DMT Parameters

The various DMT parameters are defined as follows:

- A-Pressure: The pressure that must be applied to the back of the membrane to expand its center 0.05 mm (lift-off) in soil;
- B-Pressure: The pressure that must be applied to the back of the membrane to expand its center 1.10 mm in soil;
- ΔA-membrane-calibration-pressure: The suction, recorded as a positive value, that must be applied to the back of the membrane to retract its center to the 0.05 mm deflection in air.
- ΔB-membrane-calibration-pressure: The pressure that must be applied to the back of the membrane to expand its center to the 1.10 mm deflection of air.
- ΔAavg and ΔBavg: The averaged values of the membrane calibration pressures obtained from the respective values of ΔA and ΔB measured before and after each dilatometer profiling or a single dilatometer test;
- Zm-pressure: Any gage pressure deviation from zero when venting the blade to atmospheric pressure.
- P_0: the soil pressure against the membrane when it is flush with the blade (e.g. at 0.05 mm expansion), also termed contract pressure.
- P_1: the soil pressure against the membrane when its center is expanded 1.10 mm.
- U_0: the in situ pore water pressure prior to blade insertion at the elevation of the center of the membrane.
- σ_0: The in situ effective vertical stress prior to blade insertion at the elevation of the center of the membrane
- I_{DMT}, dilatometer material index: An index related to the type of soil.
- K_{DMT}, dilatometer horizontal stress index: An index related to the in situ horizontal stress.
- E_{DMT}, dilatometer modulus: A parameter related from theory to the modulus of elasticity of soil.

Insertion Apparatus

The equipment for inserting the dilatometer blade into the soil is comprised of:

- A thrust machine to insert and advance the dilatometer blade into the soil.
    - Diving should be avoided except when advancing the blade through stiff or strongly cemented layers, which cannot be penetrated by static push.
    - The thrust machine should be capable of advancing the blade vertically with no significant horizontal or torsional forces.
- Push rods with suitable adapters to connect the blade.
- Hollow slotted adapters for lateral exit of the pneumatic-electrical cable

Membrane Calibration

- The membrane is calibrated to measure the values of the ΔA-suction and ΔB-pressure with the dilatometer equipment assembled and ready for testing immediately before inserting the blade into the soil.
- Apply suction to the back of the membrane to measure stiffness of the membrane.
- Apply pressure behind the membrane to determine the elasticity of the membrane.
- Especially when testing soft soils, the membrane calibration should be performed several times to assure that the values of ΔA and ΔB fall within prescribed limits and are consistently repeatable.

Measurements used to convert dilatometer data to soil properties for engineering design.

Field Data:

(a) test readings A and B corrected using equipment calibrations:
$p_0 = f(A)$, $p_1 = f'(B)$.

(b) est'd. (or C-reading) in-situ pre water: u (see Note 10) pressure.

(c) est'd. in-situ effective vertical stress: $\sigma_v' = (\sigma_v - u_0)$

(d) est'd. DMT bearing capacity, q_D. Obtained from thrust = P data:

DMT Indices:

Material Index (a normalized modulus)

$$I_D = f(A,B,u) = (p_1 - p_0)/(p_0 - u_0)$$

Horizontal Stresses Index (a normalized lateral stress)

$$K_D = f(A,u,\sigma_v') = (p_0 - u_0)/\sigma_v'$$

Dilatometer Modulus (theoretical elastic modulus)

$$E_D = f(A,B) = 34.7(p_1 - p_0)$$

Interpreted Soil Engineering Properties:

Soil Type

$I_D = f(p_0,p_1,u_0) = f(A,B,u)$	empirical
Lateral Stress (drained)	
K_0 (sand) $= f(K_D, \Phi') = f(A,\sigma_v',u_0,q_D)$	semi-empirical
K_0 (clay) $= f(K_D) = f(A,\sigma_v'u)$	empirical
Strength	
Φ' (sand) $= f(K_0,\sigma_v'P) = f(A,\sigma_v'u_0,q_D)$	theoretical
S_u (OC clay) $= f(K_D, \sigma_v') = f(A, \sigma_v'u)$	empirical
S_u (NC clay) $= f(p_0) = f(A, u_0)$	empirical
Compressibility (drained)	
$M = (1/m_v) = f(E_D, I_D) = f(A,B,u_0)$	semi-empirical
p_c' (sand) $= f(K_D, \Phi') = f(A,\sigma_v'u_0,q_D)$	semi-empirical
p_c' (clay) $= f(K_D) = f(A,\sigma_v'u_0)$	empirical
Modulus (drained, v = Poisson's ratio)	
E_{25} (sand) $= f(E_D) = f(A,B)$	semi-empirical
E (clay) $= f(M,v) = f(A,B,u_0,v)$	semi-empirical

Suitability of DMT in Different Types of Soil (From Schmertmann 1988)

Suitability ranking: 0 = do not use DMT; 1 = sometimes use DMT; 2 = good; 3 = best application

Note: Hammer-driving alters the DMT results and decreases the accuracy of correlations.

Soil Type	Suitability for Different Soil Conditions: Weak, Loose *, $N_{spt} < 5$***, $q_c < 15$**** — Fills Dumped, Pumped	Weak, Loose — Natural	Medium, $N_{spt} = 25$***, $q_c = 75$**** — Fills Light compxn	Medium — Natural	Stiff, Dense **, $N_{spt} > 40$***, $q_c > 150$**** — Fills Heavy compxn	Stiff, Dense — Natural
Clays	3	3	2	2	2	2
Silts	2	2	2	2	1	1
Sands	3	3	2	2	1	1
Gravel, lg, shell and concretions	1	1	0	0	0	0
Cobbles	0	0	0	0	0	0
Rocks (weathered	0	1	0	0	0	0
CL + SI + SD	3	3	3	2	2	2
CL + SI + SD + Shell	2	2	2	2	0	0
CL + SI + SD + Rock	1	1	1**	1**	0	0
Sand + Gravel	2	2	2**	1**	0	0
Organic CL + SD	3	3	2	2	1	1
Residual w/o rock	3	3	2	2	1	1
Residual w/ rock	1	1	0	1**	0	0
Cemented sand	-	1	-	1**	-	0
Tallus with rock	-	1	-	1**	-	0
Glacial till	0	1	0	0	0	0
Varved Clays	3	2	2	2	1	1
Loess	3	2	2	2	-	-
Peats	3*	2*	2	2	-	-
Slimes, tailings	3*	-	2	-	-	-

* Sensitive testing in very weak soils.

** High risk of damage; use high strength blade and membrane

*** N_{spt} = Standard Penetration Test blow count: Blows/ft.

**** q_c = Cone Penetrometer point resistance: bars.

TABER IN-SITU TESTING

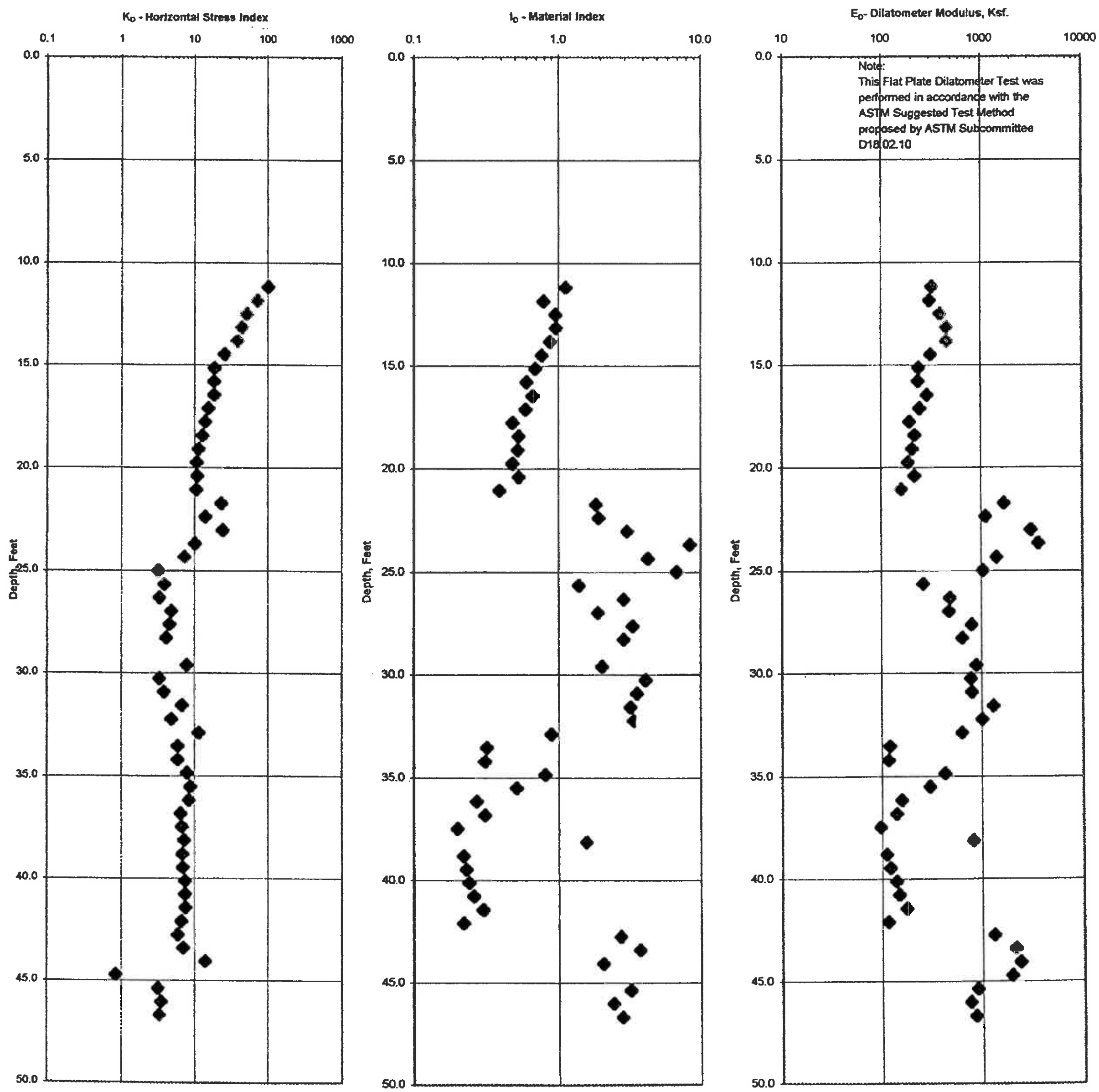
DILATOMETER TEST RESULTS

Job Name:
Job No.:
Operator:
DMT Date:
Location:

K_D - Horizontal Stress Index
$(p_0 - u_0)/\sigma_v'$

I_D - Material Index
$(p_1 - p_0)/(p_0 - u_0)$

Ed - Dilatometer Modulus



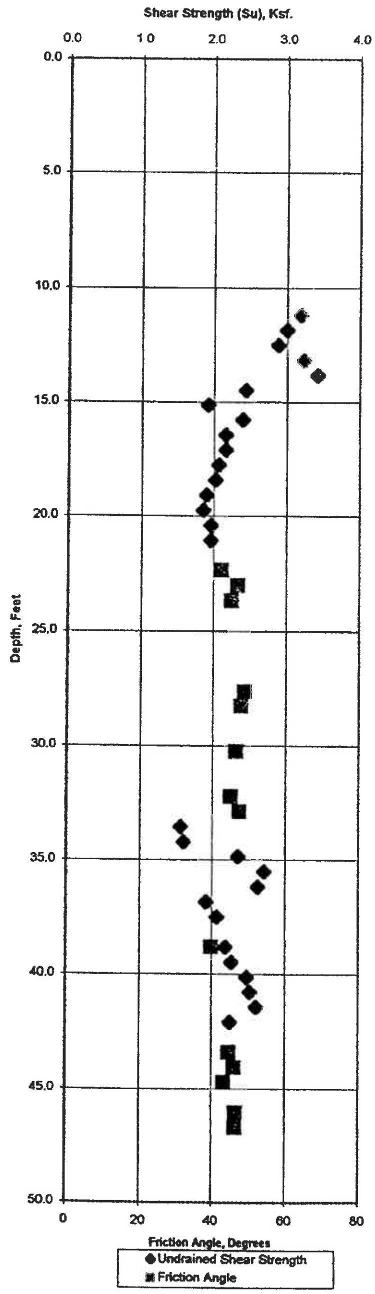
TABER IN-SITU TESTING

DILATOMETER TEST INTERPRETATION

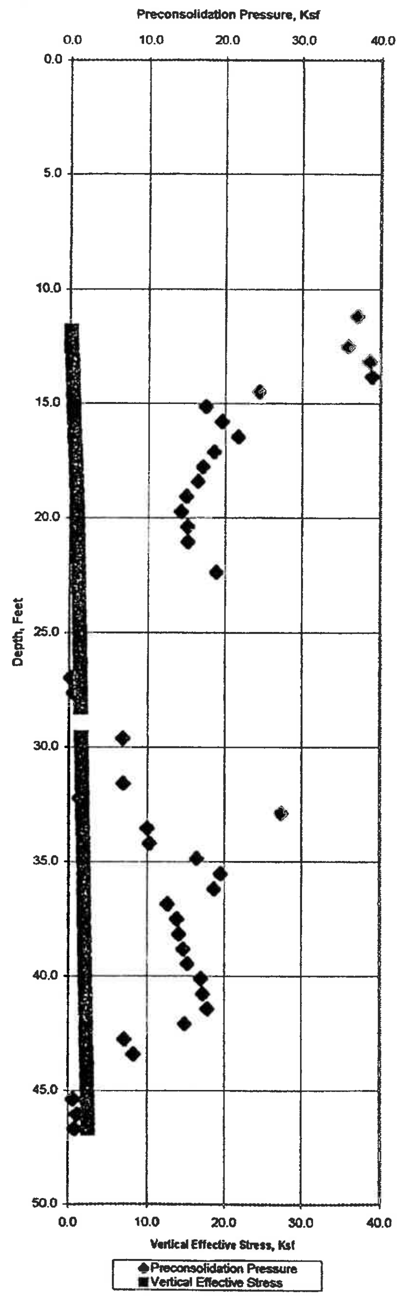
Job Name:
Operator:
Location:

Job No.:
DMT Date:

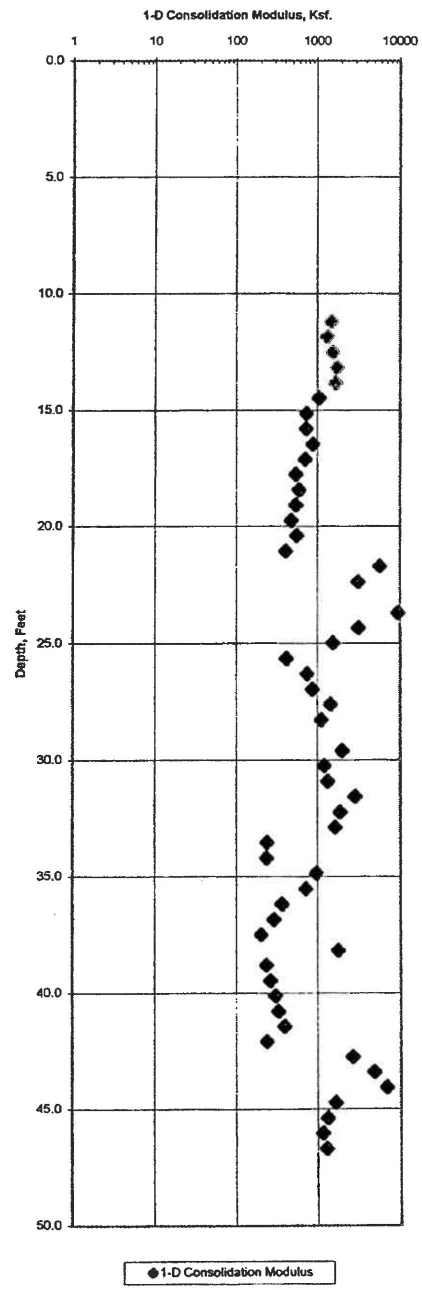
Undraine Shear Strength (S_U) &Friction Angle (Phi)



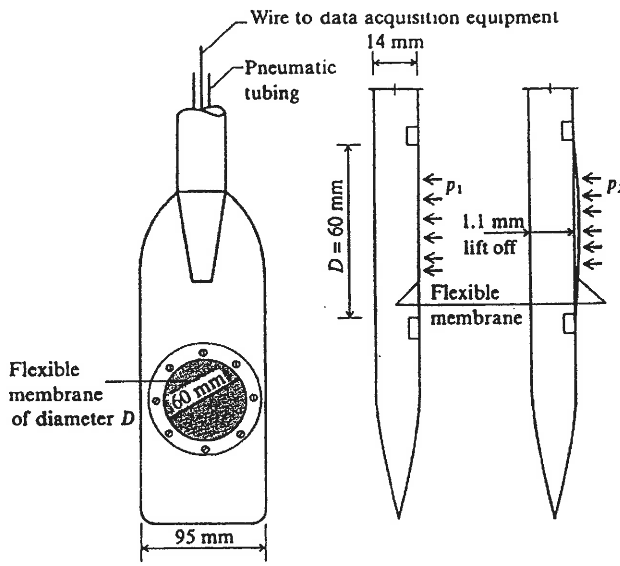
Preconsolidation Pressure (P_C) & Vertical Effective Stress (σ_v')



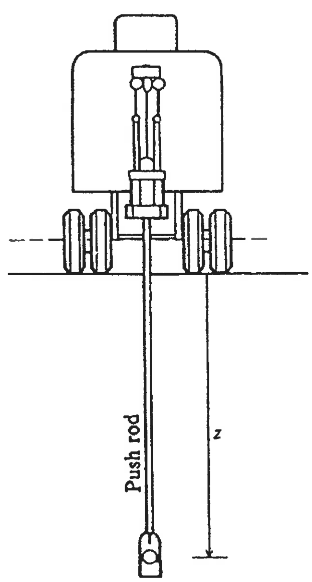
1-D Consolidation Modulus, M



Flat Plate Dilatometer Test

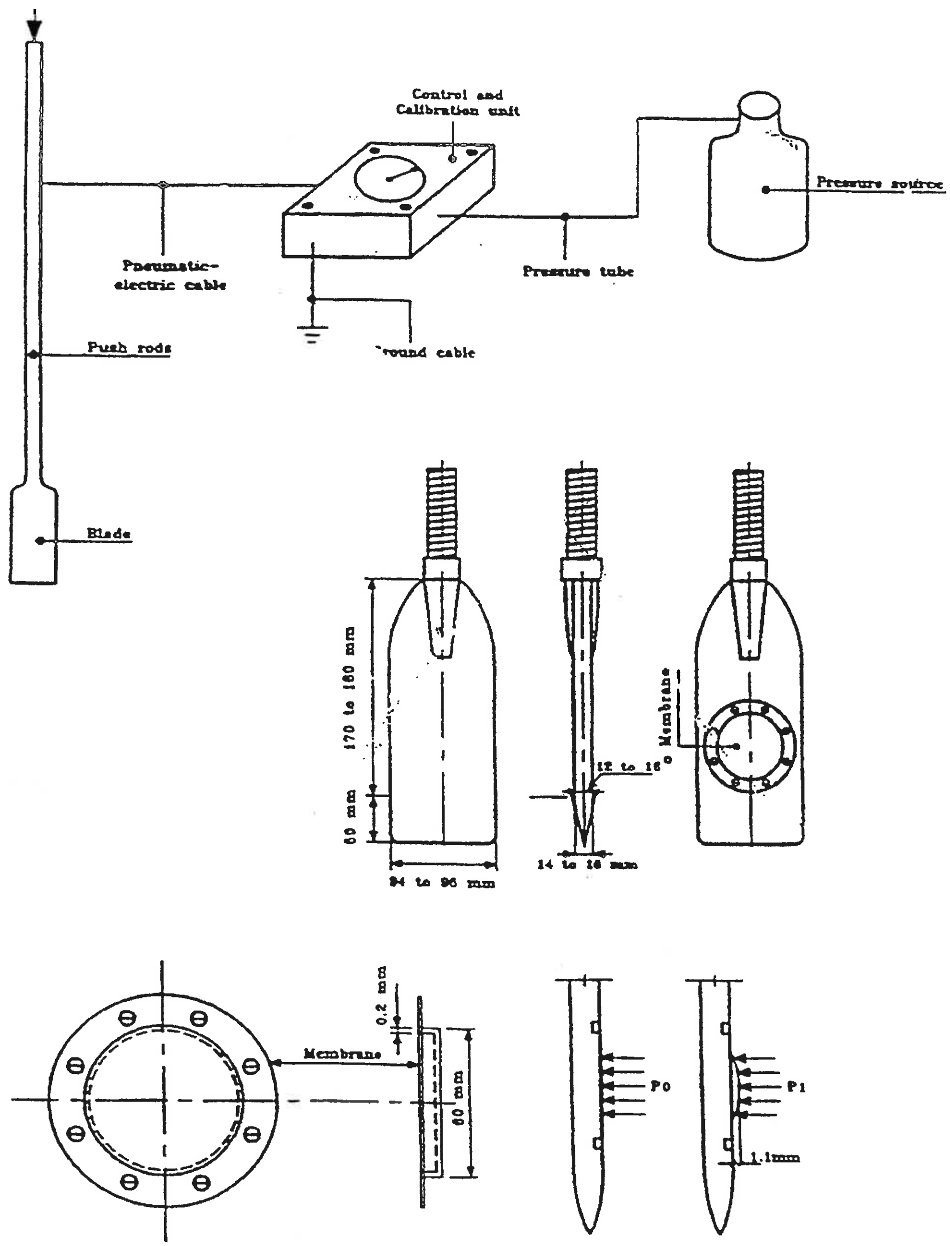


(*a*) Marchetti dilatometer [*After Marchetti (1980)*].



(*b*) The dilatometer pushed to depth z for test.

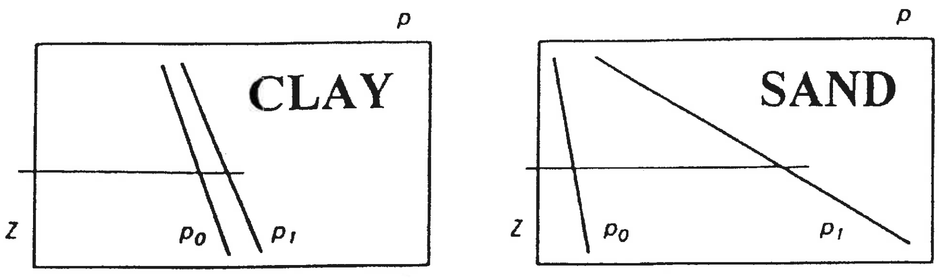
Figure 3-28 The flat dilatometer test (DMT).



Dilatometer equipment and definition of calculated in situ soil pressure

I_d - Material index (soil type)

Whoever does DMT 1st time notes :



∴ natural (apart theory) define Id as a ratio expressing vicinity

$$I_d = (P_1 - P_o)/(P_o - U_o)$$

Experience has shown

- Id v. sensitive, 0.1 to 10 (2 log cycles)

0.1 — 0.6	0.6 — 1.8	1.8 — 10
CLAY	**SILT**	**SAND**

- Like FR in CPT **but** : amplified, highly reproducible
- Not primary scope, but a nice extra - generally reliable
- I_D not result of sieve analysis, but from mechanical response (≈ *rigidity index*)
- Eg clay + sand described by I_D as *silt* ⇒ behaves mechanically as... (incorrect for grain size, + relevant mechanical behavior)
- If interest in permeability, (besides I_D) other index U_D

Flat Dilatometer Test (DMT)

The following is an example of correlations that may be used to determine the value of the one-dimensional tangent modulus $E_{oed} = d_{\sigma}/d_{e}$ from the results of the DMT tests:

$E_{oed} = R_M \, E_{DMT}$

In which RM is estimated either on the basis of local experience or using the following relationships:

- if $I_{DMT} \leq 0.6$: $R_M = 0.14 = 2.36 \log K_{DMT}$

- if $I_{DMT} \geq 3.0$: $R_M = 0.5 = 2 \log K_{DMT}$

- if $0.6 < I_{DMT} < 3.0$: $R_M = R_{MO} + (2.5 - R_{MO}) \log K_{DMT}$,
  in which $R_{MO} = 0.14 + 0.15\,(I_{DMT} - 0.6)$

- if $K_{DMT} > 10$: $R_M = 0.32 + 2.18 \log K_{DMT}$

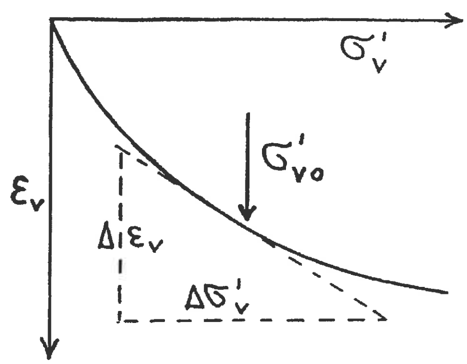
when values of RM < 0.85 are obtained in the above relationships, RM is taken equal to 0.85.

Reference:

Marchetti, S. (1980)
In situ test by flat dilatometer
Journal of the Geotechnical Engineering Division, Proc. ASCE, Vol. 106, N. GT3, pp 299-321.

1. Definition of M (no ambiguity)

$M = Eoed = 1/m_V = \Delta\sigma'_v/\Delta\varepsilon_v$ (at σ'_{vo}).



Vert. drained confined tangent (at σ'_{vo}) modulus

Name : Eoed OK, traditionally measured by oedometer. *Improper* if not by oed (correlations). Hence "M", but same.

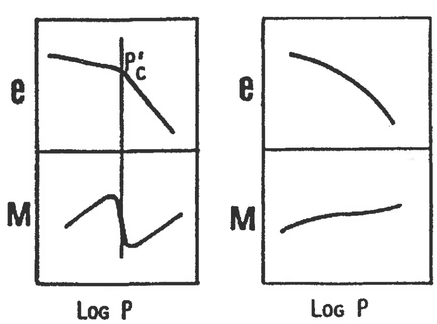
Usual range M_{DMT} **4 - 4000 bar**

2. M for what settlement (initial, 1ry, 2ry) ?

M is just for **primary**. Correlations : calibrating vs Eoed (1-D). M_{DMT} **treated as if obtained by oedometer**.

Use same methods used with oedometer type calculations, including, if applicable, usual corrections (depth, shape, rigidity, possibly Skempton-Bjerrum).

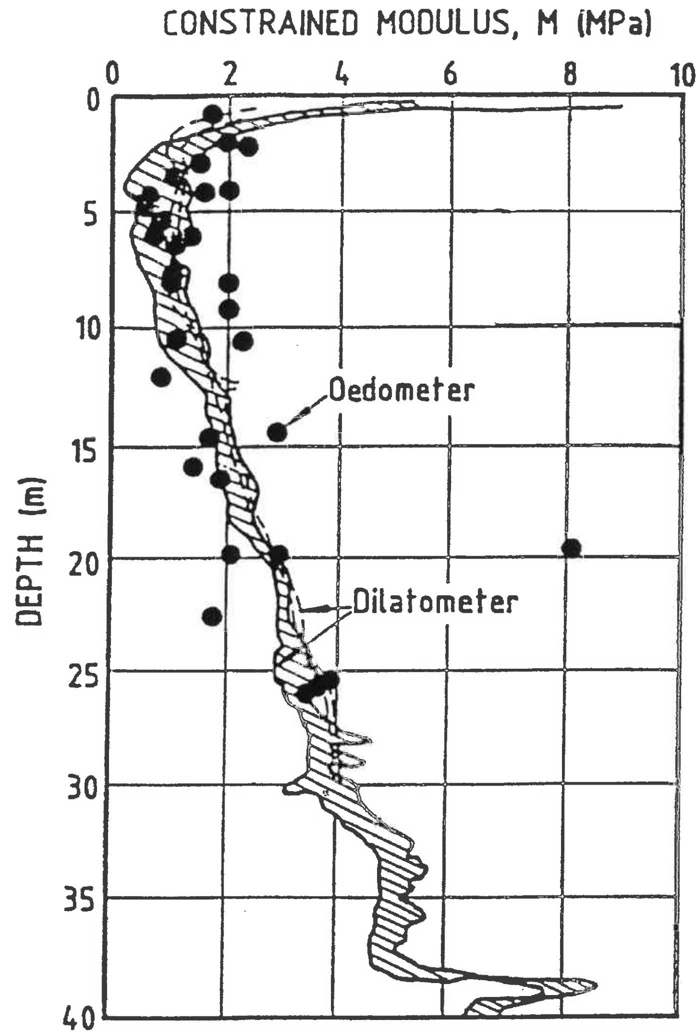
3. Use M constant if $\Delta\sigma'_v$ large ?



If $\Delta\sigma'_v$ large : will σ'_v exceed p_c?

Many structured NC clays (eg some Canadian) sharp break in e-log p curve ⇒ marked drop in M at p_c. There M_{DMT} may be few times too high.

But in many common clays, (in most sand?) M accross p_c mild fluctuation, hence M=const. ≈ OK



(a) Soft plastic clay (OCR=1-2)
(Onsøy site)

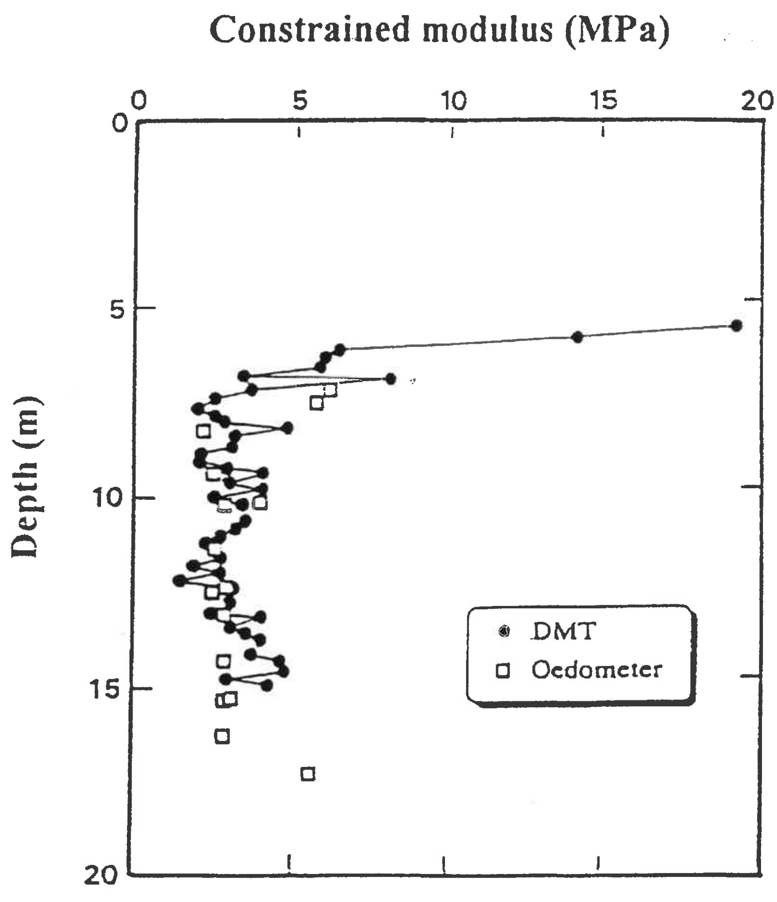
OFFSHORE SITE EXPLORATION TECHNIQUES
Comparisons of constrained modulus obtained from dilatometer test and reference constrained modulus.
Norwegian Geotechnical Institute

CONSTRAINED MODULI FROM OEDOMETER AND FROM DMT

Iwasaki, Tsuchiya, Sakai, Yamamoto (1991)

Geotechnical Research Center
Kiso-Jiban Consultants Company, Tokyo

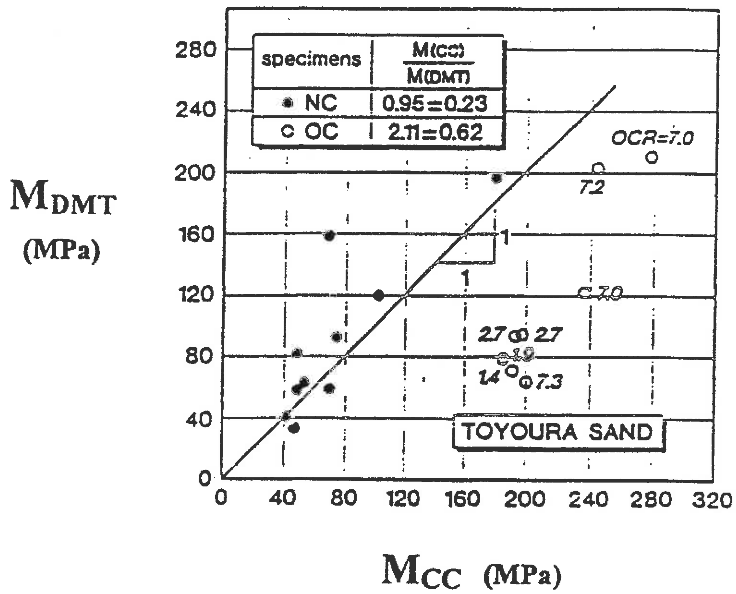
TOKYO BAY COHESIVE ALLUVIAL DEPOSITS



Iwasaki K., Tsuchiya H., Sakai Y., Yamamoto Y., (1991) "Applicability of the Marchetti Dilatometer Test to Soft Ground in Japan". GEOCOAST '91. Sept. 1991. Yokohama 1/6

M_{CC}/M_{DMT} from Calibration Chamber

Bellotti, Fretti, Jamiolkowski, Tanizawa - Delhi 94, Toyura sand



from these (18 tests) & other CC ⇒ essentially:

NC sands	$M_{DMT} \approx M_{cc}$
OC sands	M_{DMT} underestimates M_{cc} by factor ≈ 2 ($M_{cc,oc}$ = truth ?, since such overestimation not noted in real jobs).

However, assuming CC = truth:

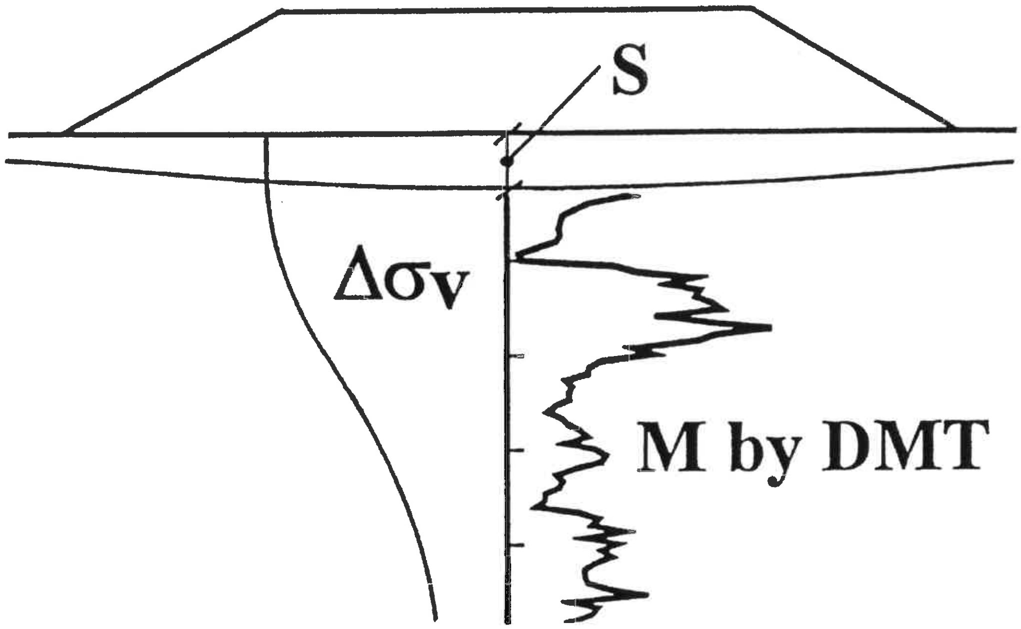
-M_{DMT} can underestimate M by a factor 2

-$\alpha \cdot Q_c$ can underestimate M by a factor 4

∴ <u>according to CC</u> <u>accuracy gain by DMT ≈ 2</u>

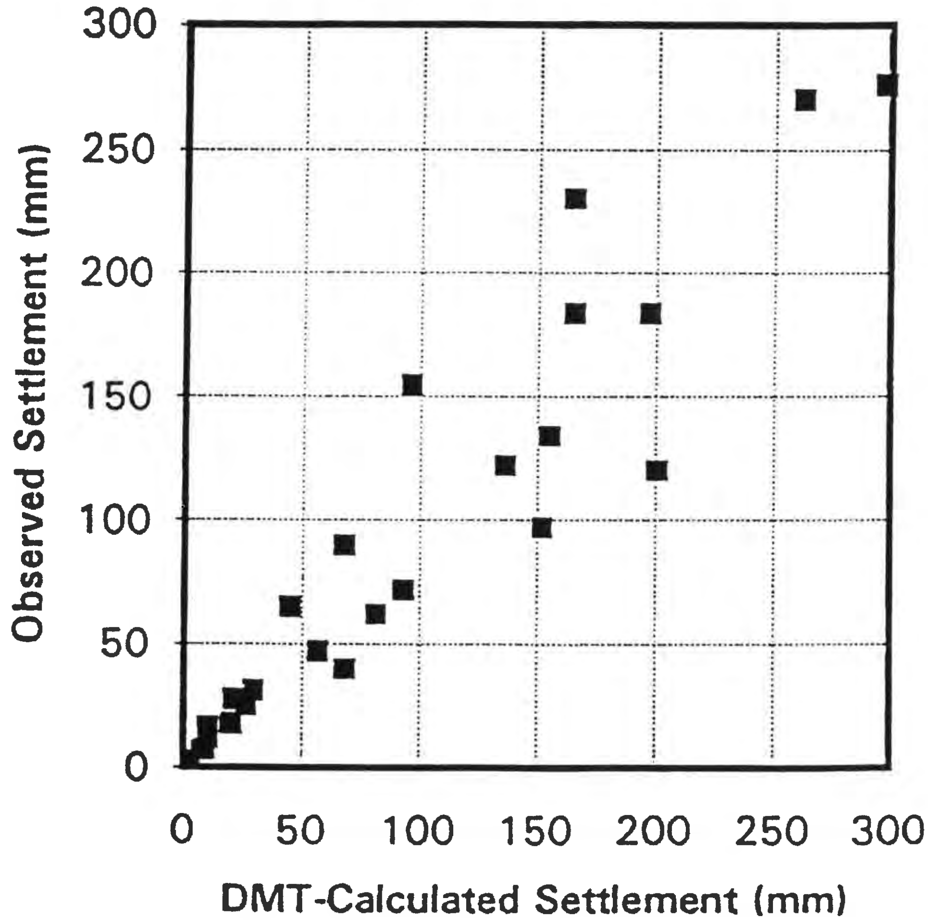
Note : since high OCR means high K_d, coincidence could be obtained easily by increasing the slope R_M vs K_d(thanks to availability of <u>two</u> parameters). But not advisable unless confirmed by real life observations.

SETTLEMENT CALCULATION



$$S = \sum \frac{\Delta \sigma_v}{M} \cdot \Delta Z$$

COMPARISON OBSERVED vs DMT-CALCULATED SETTLEMENT (HAYES 1990)



COMPARISON of DMT-CALCULATED and OBSERVED SETTLEMENTS SCHMERTMANN, 1986

"Dilatometer to compute Foundation Settlement" Proc. In Situ '86 ASCE Spec. Conf. Virginia Tech, Blacksburg.

No. 5.2.	Location	Structure	Compress. soil	Settlement (mm) DMT	**	Meas.	ratio DMT/Meas.
1	Tampa	bridge pier	NOC Clay	* 25	b,d	15	1.67
2	Jacksonvll.	Power Plant	compacted sand	* 15	b,o	14	1.07 (ave. 3)
3	Lynn Haven	factory	peaty sd.	188	a	185	1.02
4	British Columbia	test embankment	peat org. sd.	2030	a	2850	0.71
5a	Fredricton	surcharge	sand	* 11	a	15	0.73
b	"	3' plate	sand	* 22	a	28	0.79
c	"	building	quick cl. silt	* 78	a	35	2.23
6a	Ontario	road embankment	peat	*300	a,o	275	1.09
b	"	building	" peat	*262	a,o	270	0.97
7	Miami	4' plate	peat	93	b	71	1.31
8a	Peter-borough	Apt. bldg	sd.& si.	* 58	a,o	48	1.21
b	"	Factory	"	* 20	a,o	17	1.18
9	"	water tank	si. clay	* 30	b,o	31	0.97
10a	Linkoping	2x3 m plate	si. sand	* 9	a,o	6.7	1.34
b	"	1.1x1.3m plate	si. sand	* 4	a,o	3	1.33
11	Sunne	house	silt & sand	* 10	b,o	8	1.25

16 CASES HISTORY. AVERAGE CALCULATED/OBSERVED = 1.18

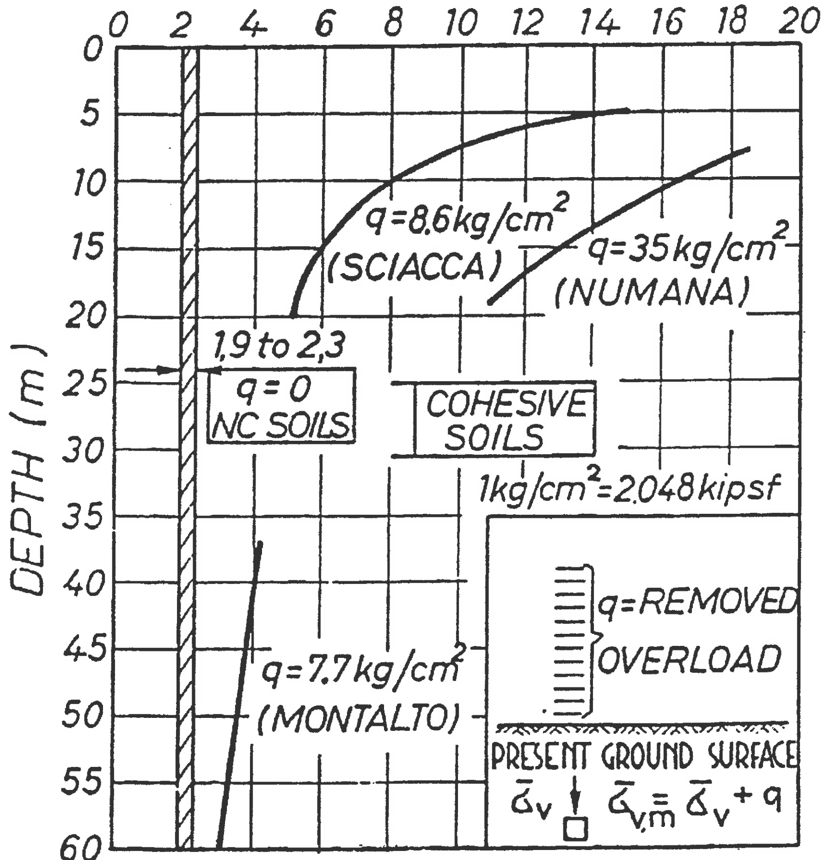
Similar agreement by others, e.g.:

Lacasse, S. & Lunne, T. 1986. Dilatometer Tests in Sand. Proc. In Situ '86 ASCE Spec. Conf. Virginia Tech, Blacksburg.

Sallfors G. (1988) "Validity of compression modulus determined by dilatometer tests", Proc. of two-day seminar at NGI on calibration of in situ tests.

K_D

K_D $\left(\approx \text{amplified } K_o\right)$

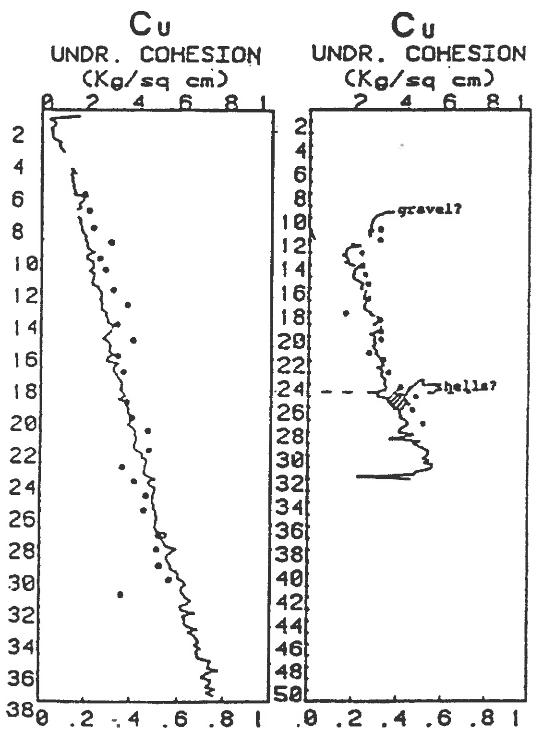


K_D SIMILAR TO OCR (z)

$$\frac{C_u}{p} \cong \left(\frac{C_u}{p}\right)_{NC} \cdot OCR^{\sim 0.8}$$

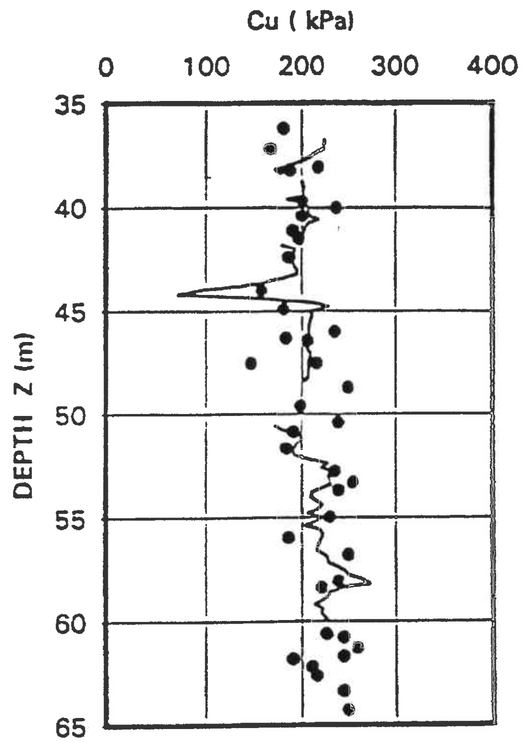
Cu predicted by DMT vs Cu determined FV / UU

NC



Cu predicted by DMT vs Cu
determined by Field Vane
(Skeena Ontario Canada, Soft Clay)

OC



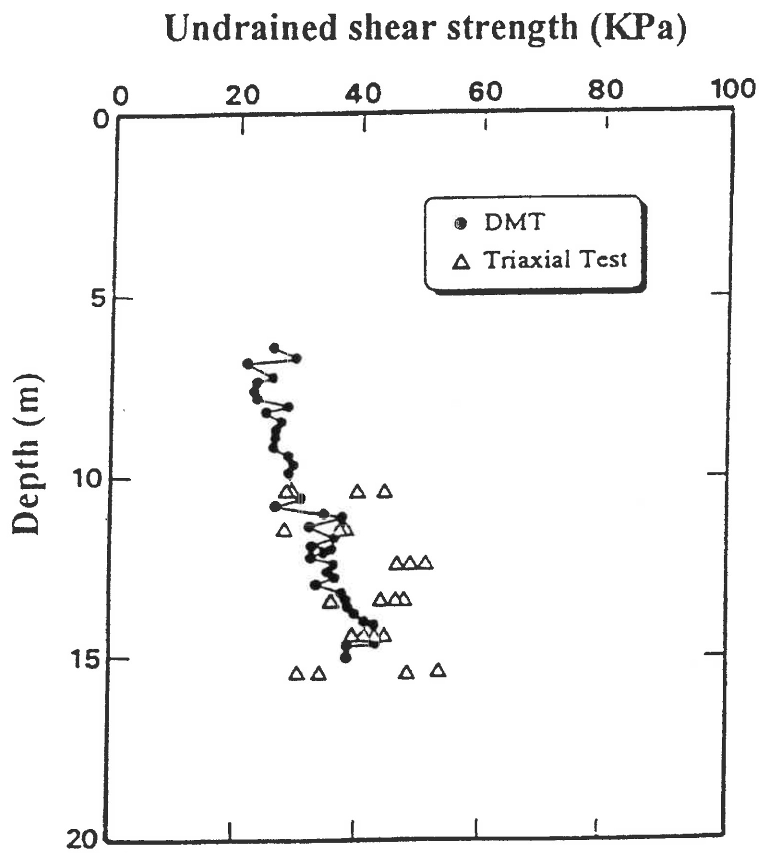
Cu predicted by DMT vs Cu
determined by lab TRX UU
(Montalto di Castro, Italy)

UNDRAINED SHEAR STRENGTH FROM UU TRIAXIAL AND FROM DMT

Iwasaki, Tsuchiya, Sakai, Yamamoto (1991)

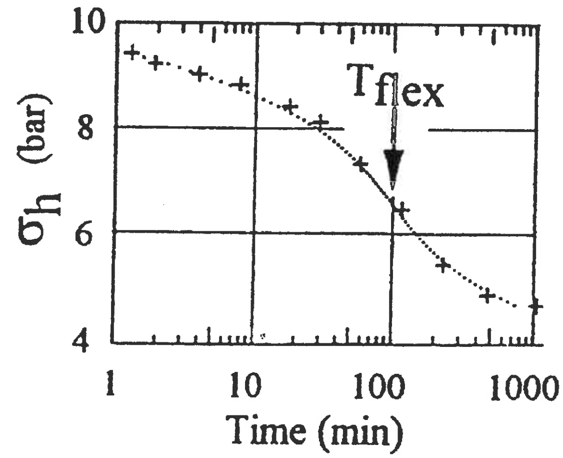
Geotechnical Research Center
Kiso-Jiban Consultants Company, Tokyo

TOKYO BAY COHESIVE ALLUVIAL DEPOSITS



Iwasaki, K Tsuchiya H., Sakai Y., Yamamoto Y. (1991) "Applicability of the Marchetti Dilatometer Test to Soft Ground in Japan", GEOCOAST '91, Sept. 1991, Yokohama 1/6

COEFFIC. CONSOLIDATION/ PERMEABILITY



$$C_h \cong \frac{7cm^2}{T_{flex}} \qquad k = \frac{C \cdot \gamma_w}{M}$$

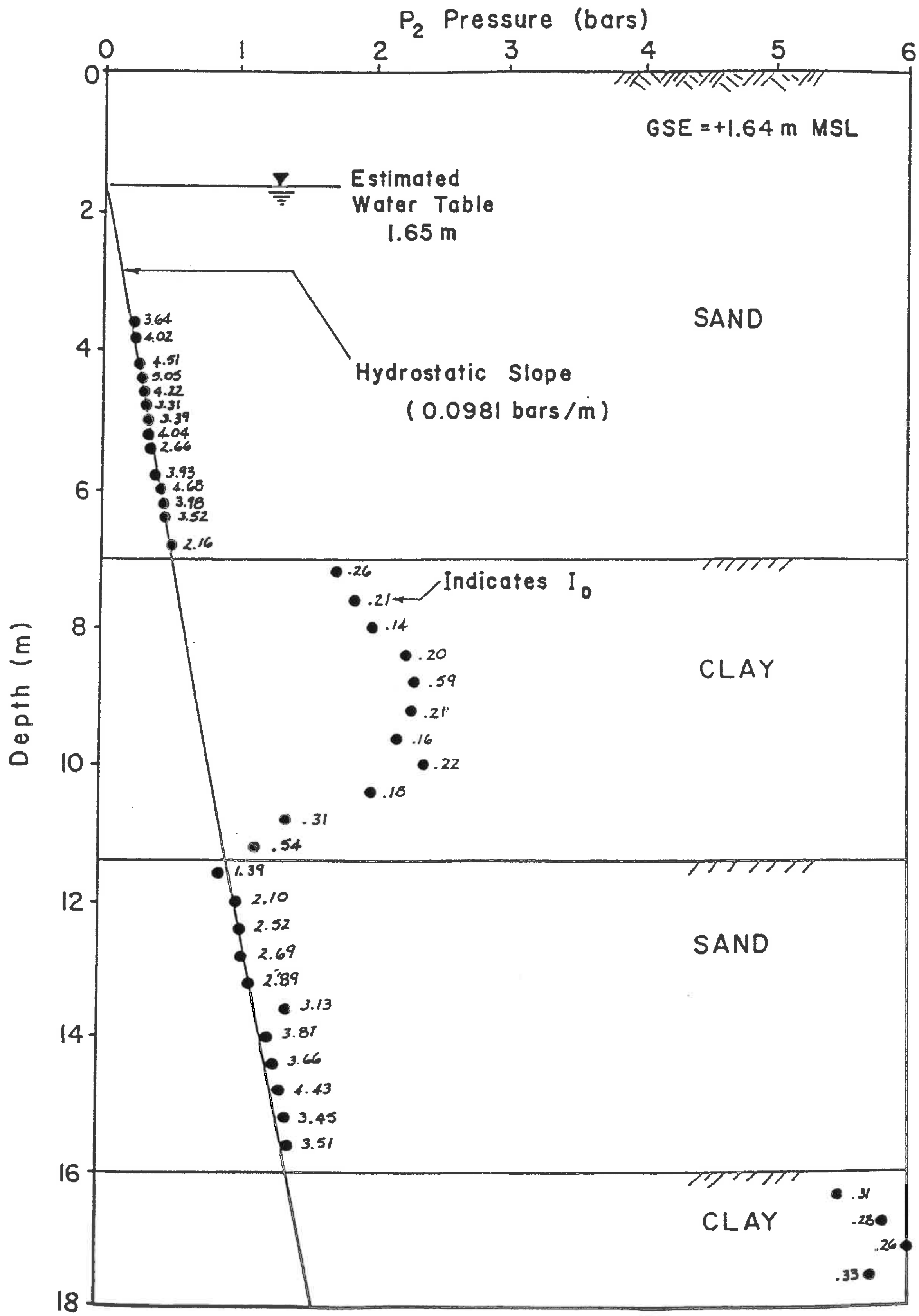
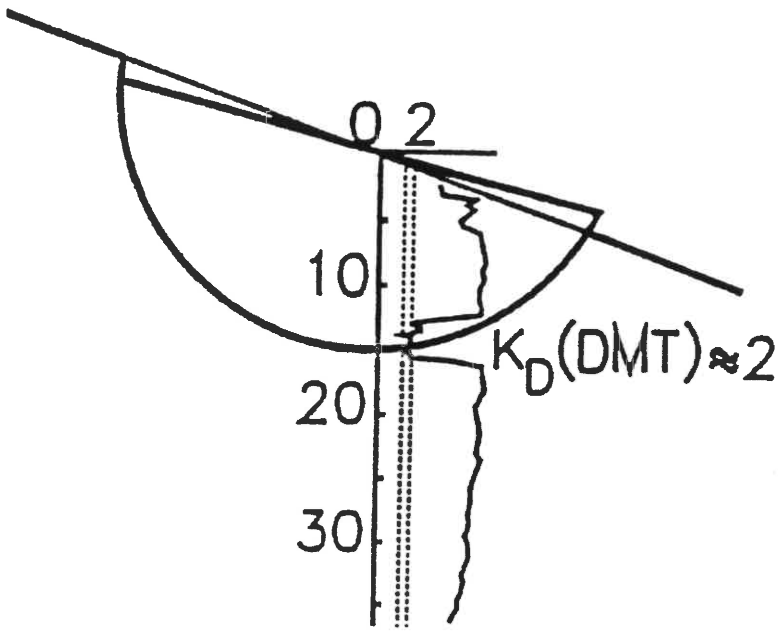
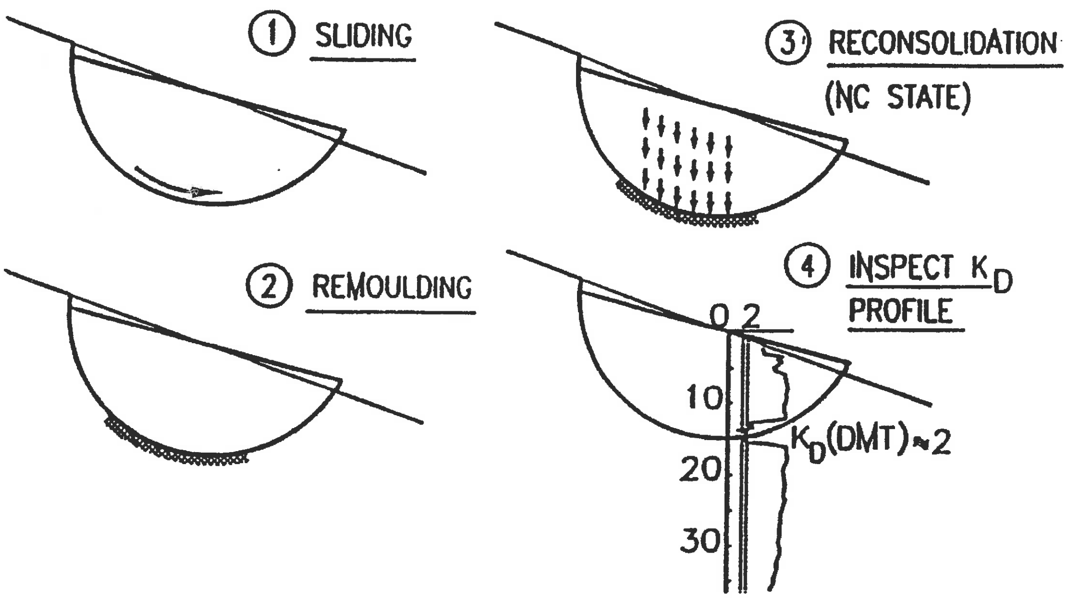


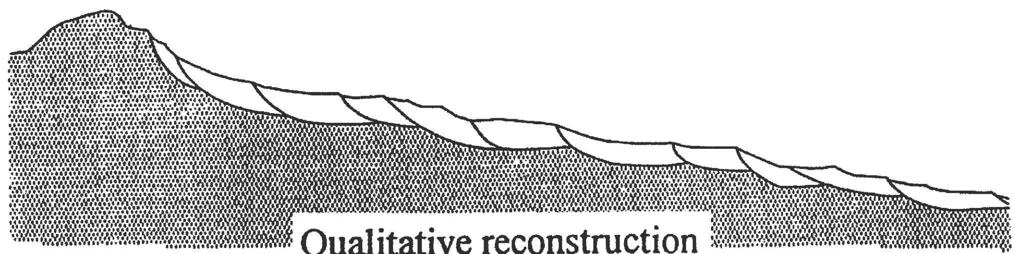
FIGURE 4.8 P_2 vs. DEPTH RESULTS FROM CAUSEWAY AT CHOCTAWHATCHEE BAY, FL (GPE Inc. Demonstration Sounding 1987)

DETECTING SLIP SURFACES IN OC CLAY SLOPES

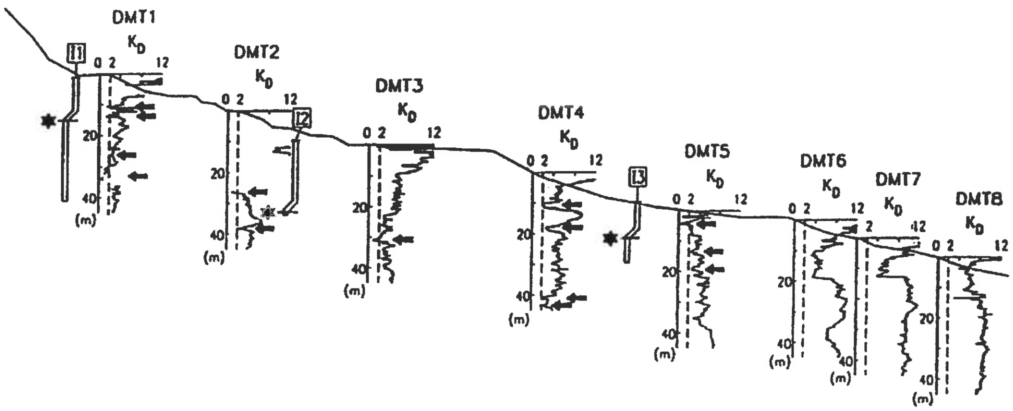




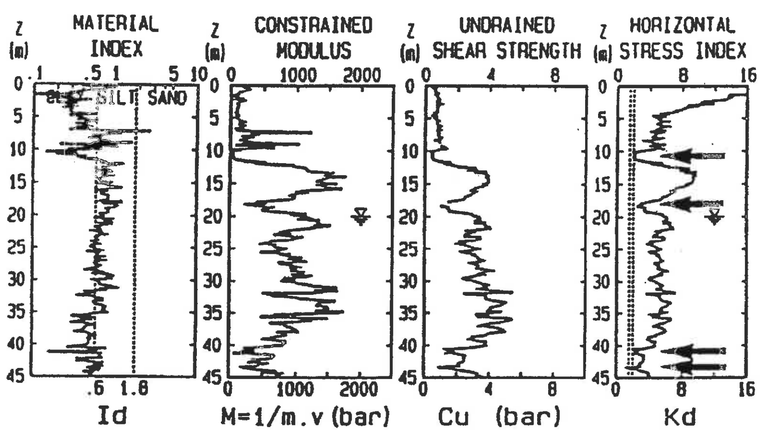
TODI HILL (UMBRIA)



Qualitative reconstruction
(Tonnetti, 1978)



DMT4

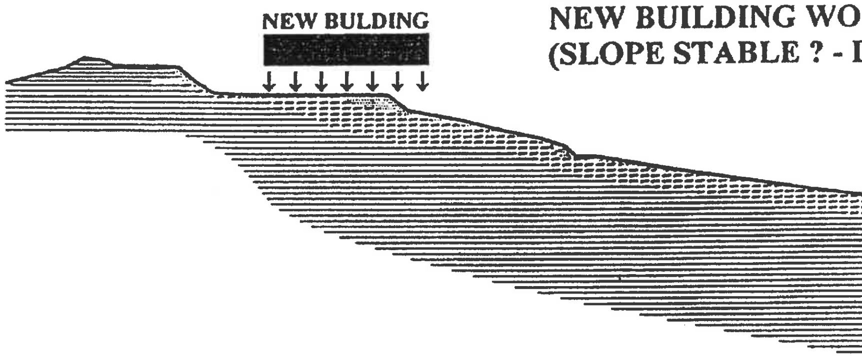


- Specific value Kd ≈ 2
- Can detect quiescent (inclinometers can't - must move)

IDENTIFICATION OF SLIP SURFACES (active /quiescent)

Case history : Chieti hills (Central Italy), 1996.

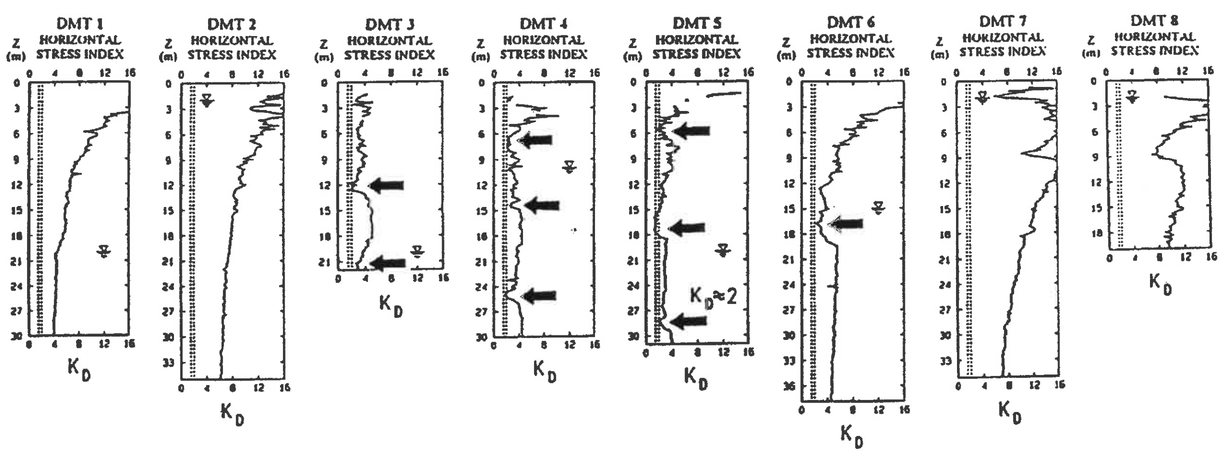
THE PROBLEM :
NEW BUILDING WOULD BE SAFE ?
(SLOPE STABLE ? - DIAGNOSIS)



TOP HILL

ZONE "PLAGUED" BY $K_D \approx 2$ LAYERS
($K_D \approx 2 \Rightarrow$ "BLEEDING WOUNDS")
past / present shear planes

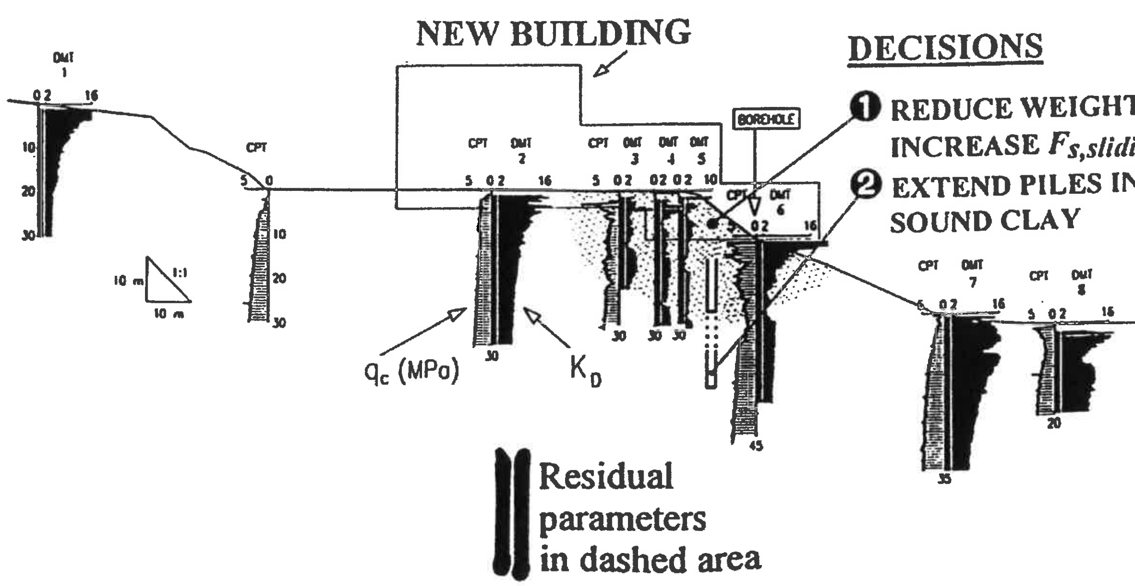
DOWN HILL



DECISIONS

❶ **REDUCE WEIGHT to INCREASE $F_{s,sliding}$**

❷ **EXTEND PILES IN SOUND CLAY**



EVALUATION OF UNDRAINED SHEAR STRENGTH OF COHESIVE SOILS USING A FLAT DILATOMETER

TAKESHI KAMEI[i] and KIMITOSHI IWASAKI[ii]

SOILS AND FOUNDATIONS Vol. 35, No. 2, 111-116, June 1995
Japanese Society of Soil Mechanics and Foundation Engineering

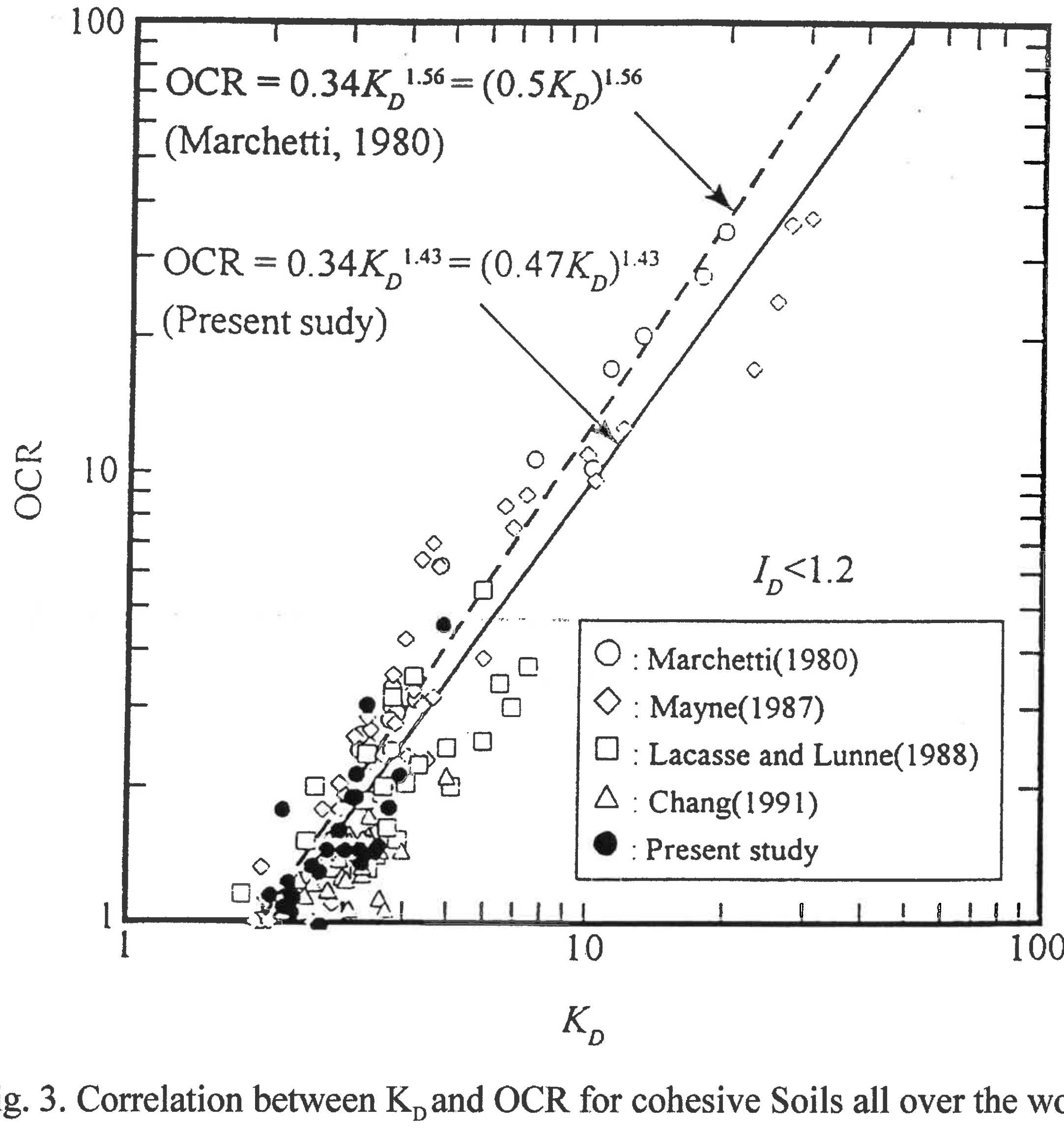
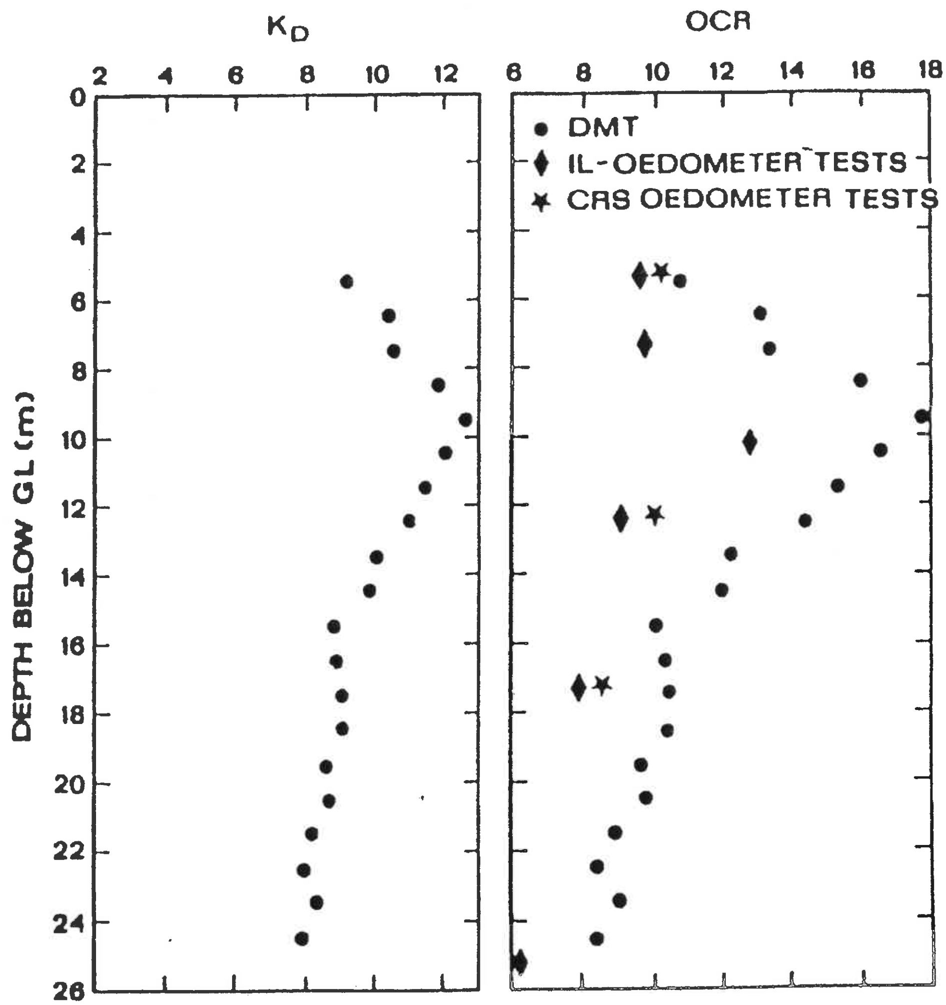
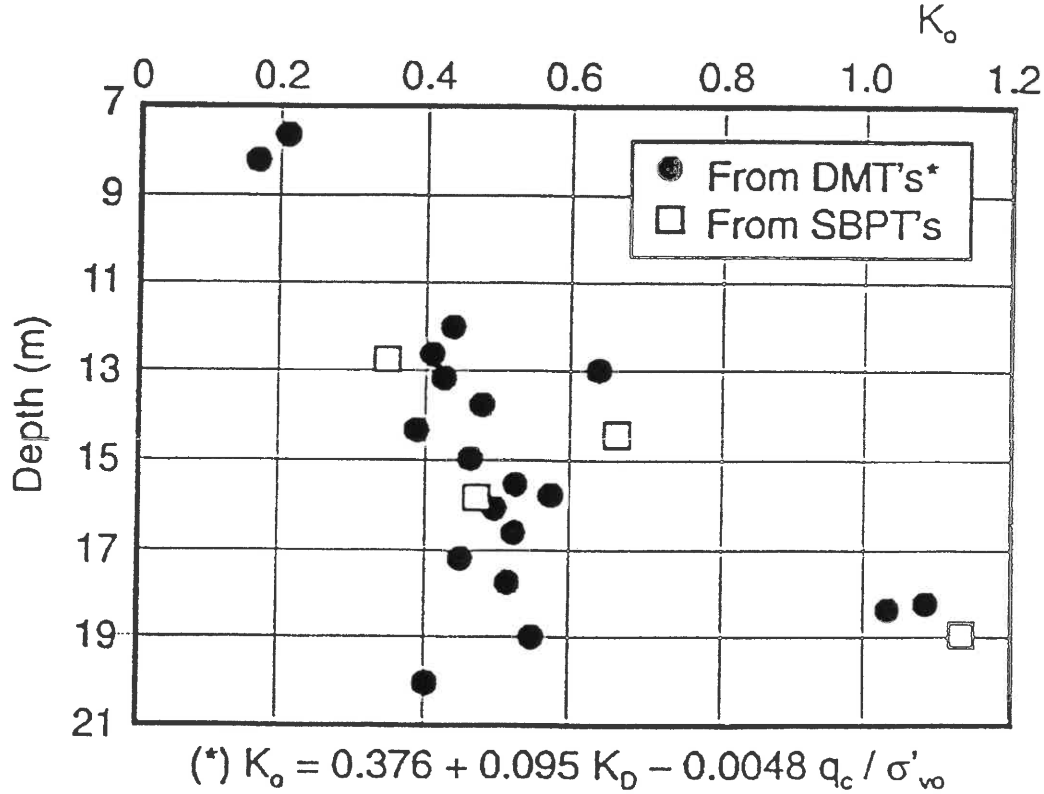


Fig. 3. Correlation between K_D and OCR for cohesive Soils all over the world



OCRDMT VS OCRoED (AUGUSTA CLAY, SICILY)

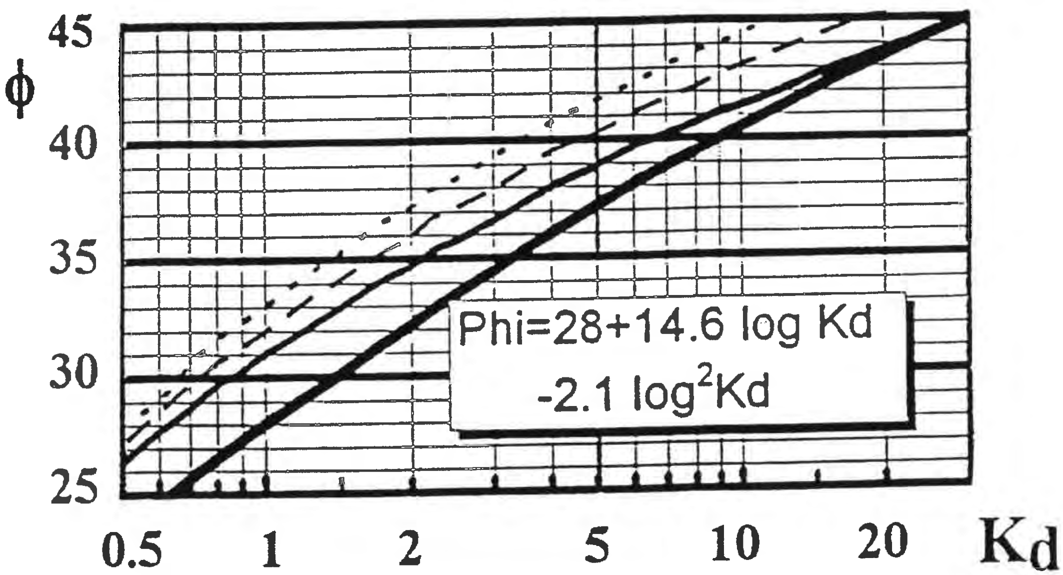
Jamiolkowsky, 1988, ISOPT I.



K_0 from DMT's and SBPT's in natural Ticino sand (Jamiolkowski 1995)

OPERATIVE LOWER BOUND ϕ

(using only K_D)



Use lower curve (or equation) Marchetti, 1987 (uncemented sands).

In-Situ Testing - FVST

FIELD VANE SHEAR TEST

- The field vane shear test (FVST) is the most widely used method for the in situ determination of undrained strength, S_u, of soft, normally consolidated or slightly over-consolidated clays.

- Originally used in Sweden in 1919.

- Employed extensively on a worldwide basis since the late 1940s.

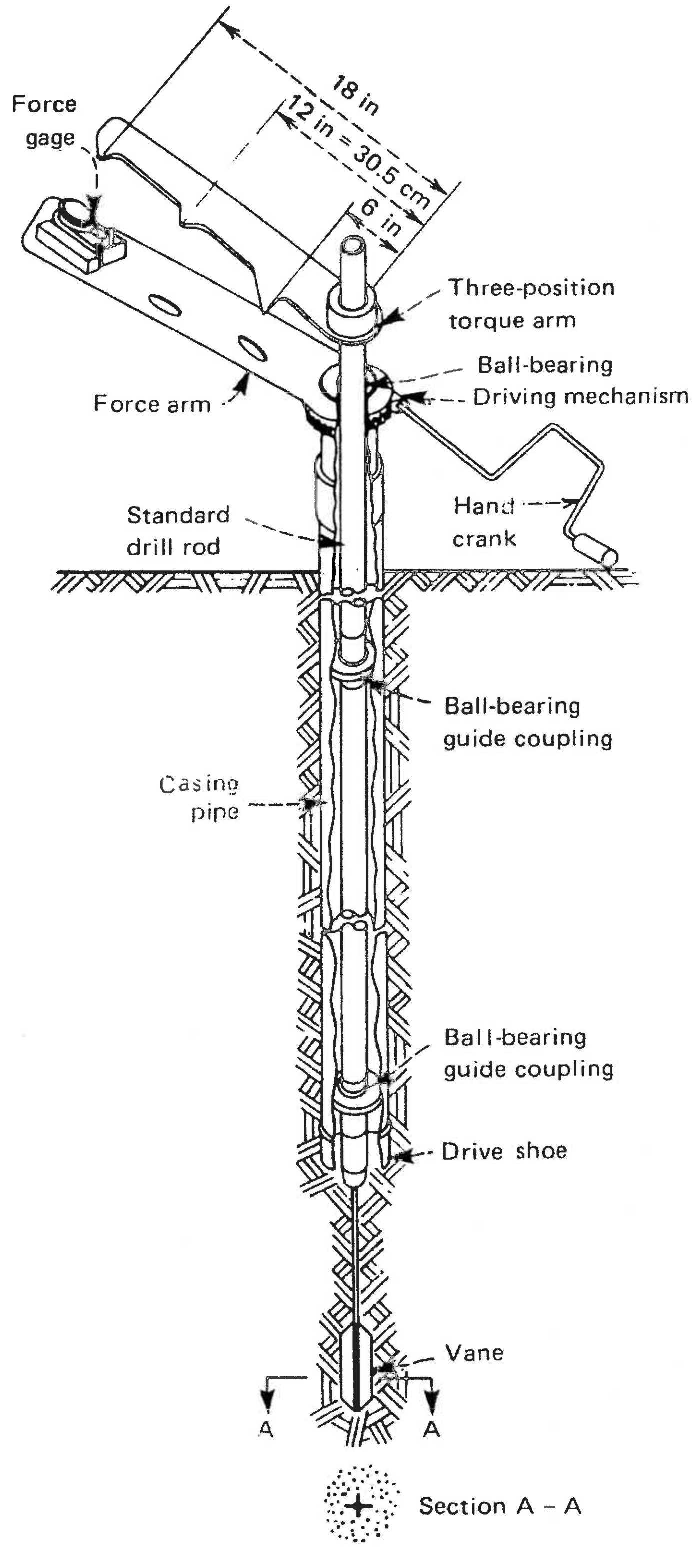
- Generally considered to give the most reliable values for S_u and S_r.

- However, FVST applies shear directionally and the values are affected by soil anisotropy.

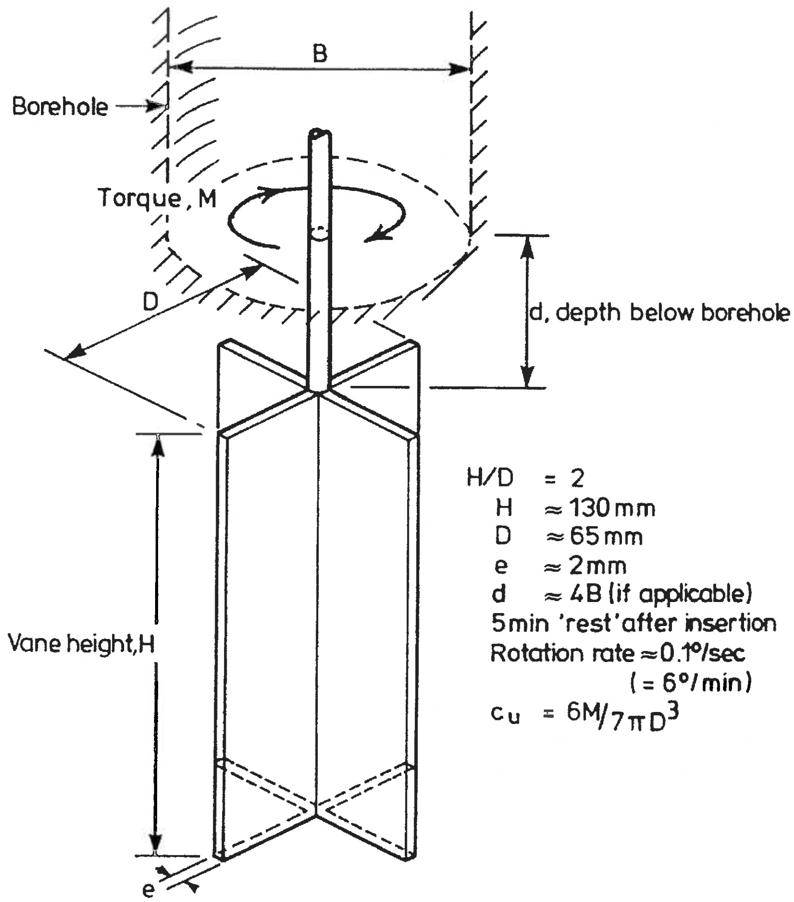
- Bjerrum et al. (1972) found that S_u measured by the field vane relate to Su measured in triaxial tests as follows:
  - S_u (compression) ≈ 1.5 S_u (vane)
  - S_u (extension) ≈ 0.5 S_u (vane)
  - S_u (extension) ≈ 1.0 S_u (vane) in highly plastic clays

- Field vane loading time rate is very rapid compared with actual field loading and during construction values are generally lower. Bjerrum et al. (1972) proposed a loading time rate correction (see figure that follows).

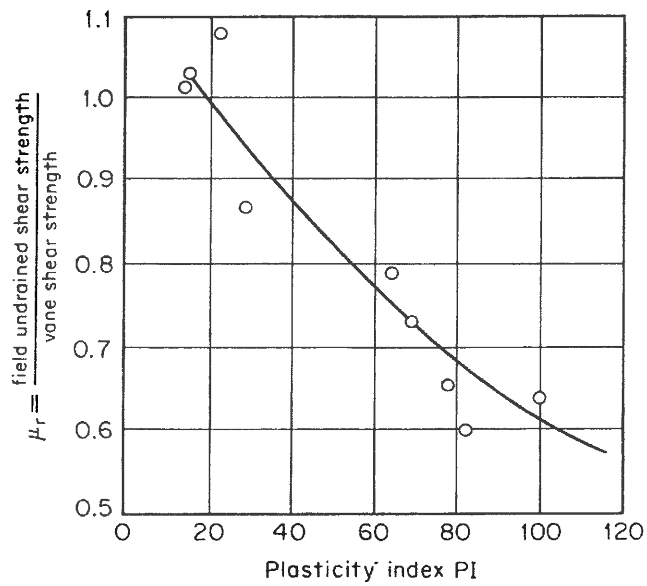
- Standardized procedures are in ASTM D 2573.



The in situ vane-shear test arrangement. *(Courtesy of Acker Drill Co.)*



—*Summary of the most commonly used dimensions and other details of the field vane test.*



Loading time-rate correction factor vs. plasticity index for undrained shear strength as measured by the field vane. [*From Bjerrum et al. (1972).*[55]]

Inclinometers

Introduction to Inclinometers

Why are inclinometers used?

An inclinometer is a device used to measure lateral movements over a period of time. These instruments are helpful in determining characteristics of landslides, slip-outs, storm damaged areas and slumps. Inclinometers are also used for measuring deflections in retaining walls and piles and deformations of excavation walls, tunnels and shafts.

Some characteristics measured are:

- Slip "plane" of landslides or depth of movement
- Rate of movement
- Type of movement, rotational etc.
- Magnitude
- Direction of movement

How is it installed and where?

Typically, inclinometers are installed after a borehole has been drilled in a specific location where information is needed. Special slotted inclinometer casing is installed into the borehole. Care should be taken to advance the borehole through the unstable soils and into stable ground below the suspected depth of movement. This "ensures" getting an accurate measurement of the depth of movement which may be critical to design elements used for stabilizing ground movements.

Inclinometers can be installed in the middle and at the lateral edges of the slide to closely define the nature of the movement. Installation away from the main part of the slide can also give limits to the areal extent of the movement.

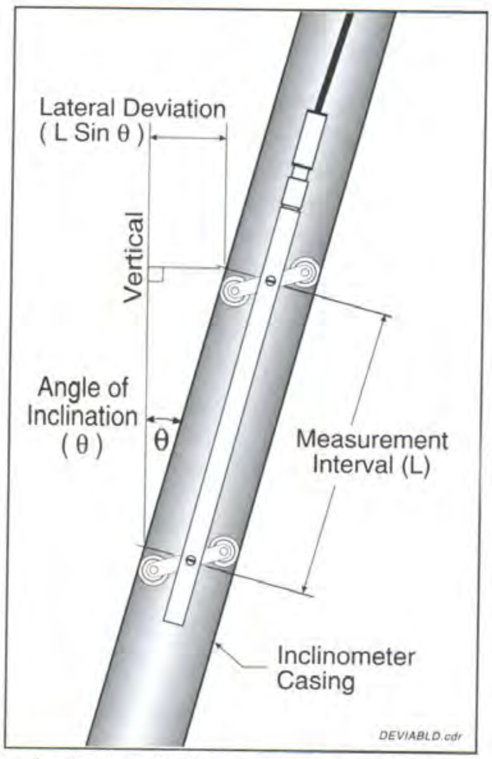
How does it work?

Baseline measurements are made immediately after installation to provide starting spatial and time elements that will be used to compare to at a later time. These parameters are measured using two accelerometers that measure the tilt of the inclinometer in two directions as the probe is reeled in from the bottom of the inclinometer casing. Measurements at a later date are then taken to compare to the initial readings and the characteristics mentioned above can be determined from the data.

Standards

Standard test methods for inclinometers are presented in ASTM D 6230.

Inclinometer Parts and Operation



Angle of Inclination and Lateral Deviation

Relation of angle of inclination to Lateral deviation.

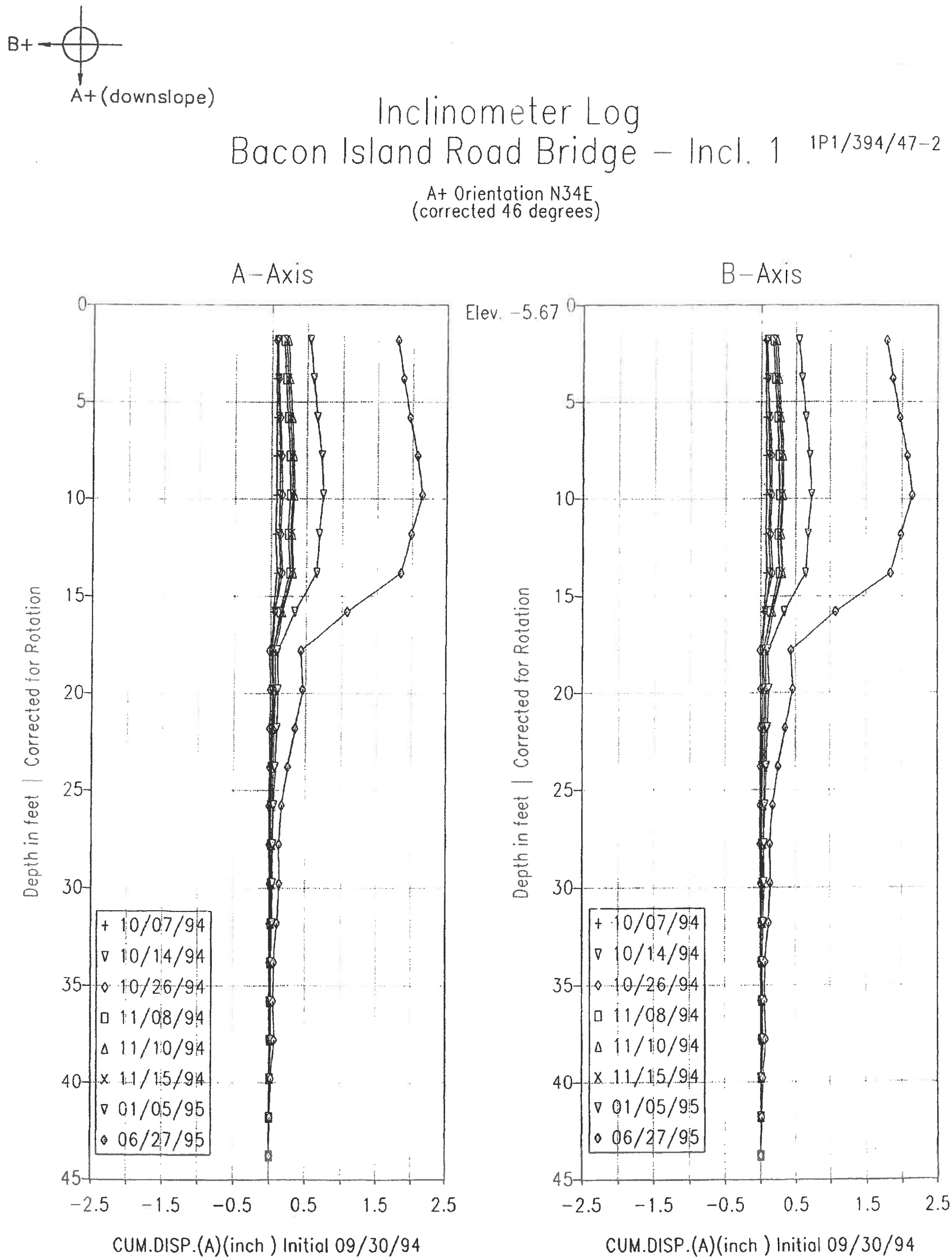
Inclinometer Casing

Inclinometer casing with the perpendicular groves visible. These grooves are used to align the probe.

Reproduced from Slope Indicator Catalog, January 1998.

Inclinometer Log
Bacon Island Road Bridge – Incl. 1 1P1/394/47-2

A+ Orientation N34E
(corrected 46 degrees)



INCLINOMETER BORING/CONSTRUCTION LOG

Job No.: 2P3/XXX/XXX

Project Name:

Client:

WELL CASING: 2.75" OD Inclinometer Casing	FROM 45 TO +0.3 ft.	Well No. I-1		Location: See Plan	
TYPE OF PERFORATION: N/A	FROM TO ft.	Elevation: -5.67		Reference: Per Survey	
SIZE AND TYPE OF FILTER: N/A	FROM TO ft.	Drilling Equipment: Diedrich 120			
TYPE OF SEAL NO. 1 Cement/bentonite slurry	FROM TO ft.	Drilling Method: 8-Inch Hollow Stem Auger			
NO. 2	FROM TO ft.	Notes:			
NO. 3	FROM TO ft.				

Elevation	Free Water Surface Observations	Graphic Log	Depth (feet)	Geologic Unit	REMARKS (drill rate, fluid loss, odor, etc.)	SOIL TESTS	BLOWS/FOOT 350 ft lb.	SAMPLE SIZE (inches)	SAMPLE No.	DEPTH (feet)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Material Description
			0–5				74	2.5	1			CL / SM	Dense to compact black and brown CLAYEY SILT/SILTY SAND with GRAVEL (fill)
			5				20	1.4	2	5		CL / SP	Semicompact and stiff black and brown CLAYEY SAND/SANDY CLAY (fill)
			10				2	1.4	3	10		Pt	Very soft dark brown and black PEAT and SILTY PEAT
			15				4	2.5	4	15		Pt / SM	Very soft dark brown and black PEATY SILT and SILT with PEAT stringers and thin SANDY SILT layers
			20				4	2.5	5	20			
			25				5	1.4	6	25		CL	Very soft to soft blue-gray interlayered SILTY CLAY/CLAYEY SILT and SILT
			30				14	2.5	7	30			
			35							35		SP	Semicompact gray fine SAND with medium SAND

— CONTINUED —

THE MONITOR WELL LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

LOGGED BY: G.D.A. | CHECKED BY: G.D.A. | DATE: 9-15&16-94 | FIGURE | Page of

7/93 DRILLOGLOG 1-1

INCLINOMETER BORING/CONSTRUCTION LOG Job No.: 2P3/XXX/XXX

Project Name:

Client:

Well No. I–1 (cont.)	Location: See Plan	Drilling Company/Equipment: Diedrich 120
Elevation: –5.67	Reference: Per Survey	

Elevation	Free Water Surface Observations	Graphic Log	Depth (feet)	Geologic Unit	REMARKS (drill rate, fluid loss, odor, etc.)	SOIL TESTS	BLOWS/FOOT 350 ft. lb.	SAMPLE SIZE (inches)	SAMPLE No.	DEPTH (feet)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Material Description
													Semicompact gray fine SAND with medium SAND
							18	1.4	8				
			40							40		SP	
							15	1.4	9				
			45							45			

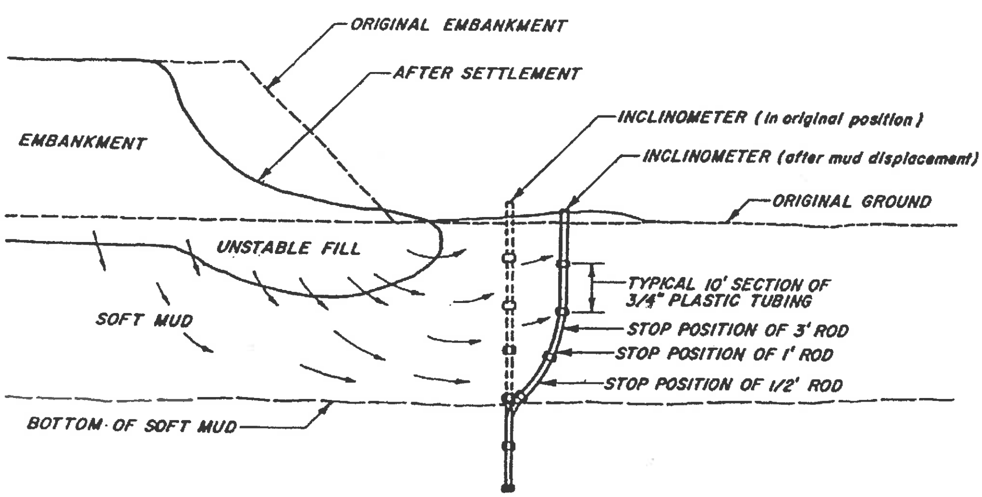
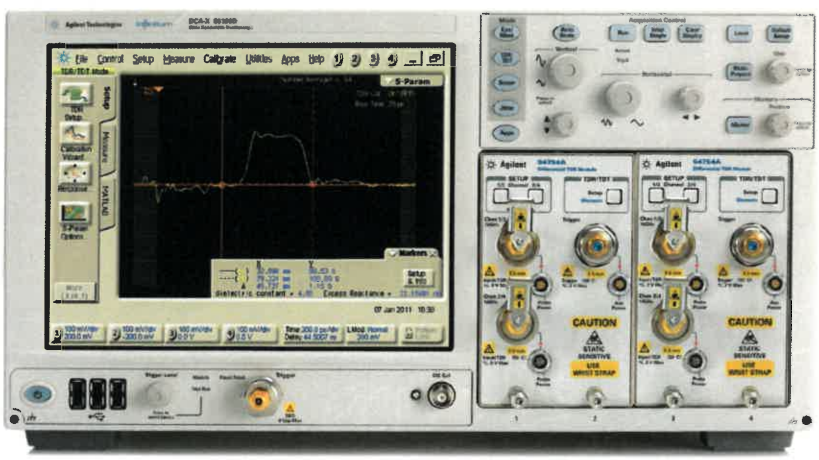


Figure G-21. An inclinometer installation placed to observe tilt associated with lateral deformation of the foundation soils underlying a roadway embankment. Inclinometer sensitivities are such that even minute movements can be closely associated with elevation and used to define the geometry and rate of deformation (From California Dept. of Transportation, 1973).

Time Domain Reflectometry

Time Domain Reflectometry Theory

Application Note



For Use with Agilent 86100 Infiniium DCA

Anticipate __Accelerate __Achieve

Agilent Technologies

Introduction

The most general approach to evaluating the time domain response of any electromagnetic system is to solve Maxwell's equations in the time domain. Such a procedure would take into account all the effects of the system geometry and electrical properties, including transmission line effects. However, this would be rather involved for even a simple connector and even more complicated for a structure such as a multilayer high-speed backplane. For this reason, various test and measurement methods have been used to assist the electrical engineer in analyzing signal integrity.

The most common method for evaluating a transmission line and its load has traditionally involved applying a sine wave to a system and measuring waves resulting from discontinuities on the line. From these measurements, the standing wave ratio (σ) is calculated and used as a figure of merit for the transmission system. When the system includes several discontinuities, however, the standing wave ratio (SWR) measurement fails to isolate them. In addition, when the broadband quality of a transmission system is to be determined, SWR measurements must be made at many frequencies. This method soon becomes very time consuming and tedious.

Another common instrument for evaluating a transmission line is the network analyzer. In this case, a signal generator produces a sinusoid whose frequency is swept to stimulate the device under test (DUT). The network analyzer measures the reflected and transmitted signals from the DUT. The reflected waveform can be displayed in various formats, including SWR and reflection coefficient. An equivalent TDR format can be displayed only if the network analyzer is equipped with the proper software to perform an Inverse Fast Fourier Transform (IFFT). This method works well if the user is comfortable working with s-parameters in the frequency domain. However, if the user is not familiar with these microwave-oriented tools, the learning curve is quite steep. Furthermore, most digital designers prefer working in the time domain with logic analyzers and high-speed oscilloscopes.

When compared to other measurement techniques, time domain reflectometry provides a more intuitive and direct look at the DUT's characteristics. Using a step generator and an oscilloscope, a fast edge is launched into the transmission line under investigation. The incident and reflected voltage waves are monitored by the oscilloscope at a particular point on the line.

This echo technique (see Figure 1) reveals at a glance the characteristic impedance of the line, and it shows both the position and the nature (resistive, inductive, or capacitive) of each discontinuity along the line. TDR also demonstrates whether losses in a transmission system are series losses or shunt losses. All of this information is immediately available from the oscilloscope's display. TDR also gives more meaningful information concerning the broadband response of a transmission system than any other measuring technique.

Since the basic principles of time domain reflectometry are easily grasped, even those with limited experience in high-frequency measurements can quickly master this technique. This application note attempts a concise presentation of the fundamentals of TDR and then relates these fundamentals to the parameters that can be measured in actual test situations. Before discussing these principles further we will briefly review transmission line theory.

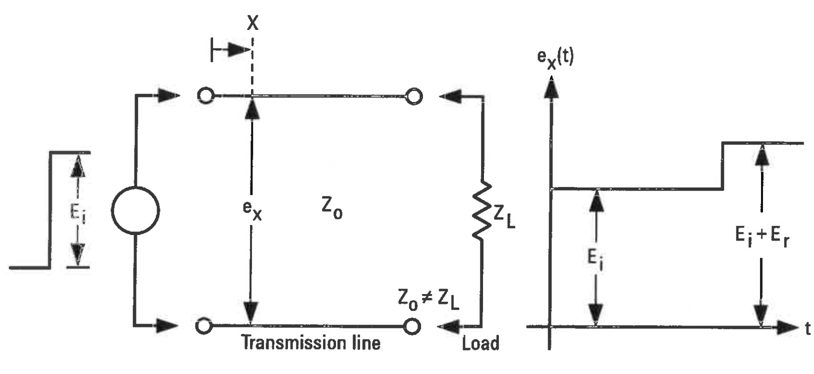


Figure 1. Voltage vs time at a particular point on a mismatched transmission line driven with a step of height E_i

Propagation on a Transmission Line

The classical transmission line is assumed to consist of a continuous structure of R's, L's and C's, as shown in Figure 2. By studying this equivalent circuit, several characteristics of the transmission line can be determined.

If the line is infinitely long and R, L, G, and C are defined per unit length, then

$$Z_{in} = Z_o \sqrt{\frac{R + j\,\omega L}{G + j_{\omega} C}}$$

where Z_o is the characteristic impedance of the line. A voltage introduced at the generator will require a finite time to travel down the line to a point x. The phase of the voltage moving down the line will lag behind the voltage introduced at the generator by an amount β per unit length. Furthermore, the voltage will be attenuated by an amount α per unit length by the series resistance and shunt conductance of the line. The phase shift and attenuation are defined by the propagation constant γ, where

$$\gamma = \alpha + j\beta = \sqrt{(R + j\omega L)\,(G + j\omega C)}$$

and α = attenuation in nepers per unit length
β = phase shift in radians per unit length

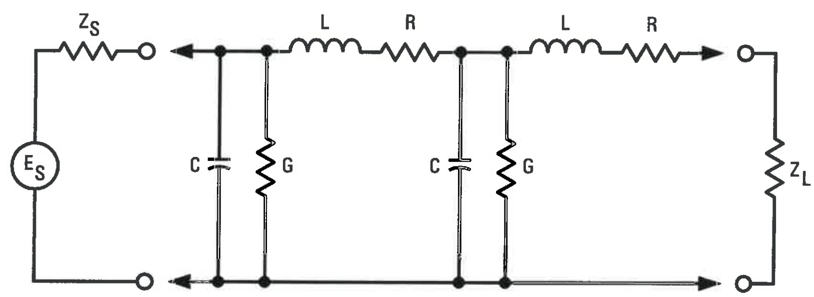


Figure 2. The classical model for a transmission line.

The velocity at which the voltage travels down the line can be defined in terms of β:

$$\text{Where } \nu_\rho = \frac{\omega}{\beta} \text{ Unit Length per Second}$$

The velocity of propagation approaches the speed of light, ν_c, for transmission lines with air dielectric. For the general case, where er is the dielectric constant:

$$\nu_\rho = \frac{\nu_c}{\sqrt{e_r}}$$

The propagation constant γ can be used to define the voltage and the current at any distance x down an infinitely long line by the relations

$$E_x = E_{in}e^{-\gamma x} \text{ and } I_x = I_{in}e^{-\gamma x}$$

Since the voltage and the current are related at any point by the characteristic impedance of the line

$$Z_o = \frac{E_{in}e^{-\gamma x}}{I_{in}e^{-\gamma x}} = \frac{E_{in}}{I_{in}} = Z_{in}$$

where E_{in} = incident voltage
I_{in} = incident current

When the transmission line is finite in length and is terminated in a load whose impedance matches the characteristic impedance of the line, the voltage and current relationships are satisfied by the preceding equations.

If the load is different from Z_o, these equations are not satisfied unless a second wave is considered to originate at the load and to propagate back up the line toward the source. This reflected wave is energy that is not delivered to the load. Therefore, the quality of the transmission system is indicated by the ratio of this reflected wave to the incident wave originating at the source. This ratio is called the voltage reflection coefficient, ρ, and is related to the transmission line impedance by the equation:

$$\rho = \frac{E_r}{E_i} = \frac{Z_L - Z_o}{Z_L + Z_o}$$

The magnitude of the steady-state sinusoidal voltage along a line terminated in a load other than Z_o varies periodically as a function of distance between a maximum and minimum value. This variation, called a standing wave, is caused by the phase relationship between incident and reflected waves. The ratio of the maximum and minimum values of this voltage is called the voltage standing wave ratio, σ, and is related to the reflection coefficient by the equation

$$\sigma = \frac{1 + |\rho|}{1 - |\rho|}$$

As has been said, either of the above coefficients can be measured with presently available test equipment. But the value of the SWR measurement is limited. Again, if a system consists of a connector, a short transmission line and a load, the measured standing wave ratio indicates only the overall quality of the system. It does not tell which of the system components is causing the reflection. It does not tell if the reflection from one component is of such a phase as to cancel the reflection from another. The engineer must make detailed measurements at many frequencies before he can know what must be done to improve the broadband transmission quality of the system.

TDR Step Reflection Testing

A time domain reflectometer setup is shown in Figure 3.

The step generator produces a positive-going incident wave that is applied to the transmission system under test. The step travels down the transmission line at the velocity of propagation of the line. If the load impedance is equal to the characteristic impedance of the line, no wave is reflected and all that will be seen on the oscilloscope is the incident voltage step recorded as the wave passes the point on the line monitored by the oscilloscope. Refer to Figure 4.

If a mismatch exists at the load, part of the incident wave is reflected. The reflected voltage wave will appear on the oscilloscope display algebraically added to the incident wave. Refer to Figure 5.

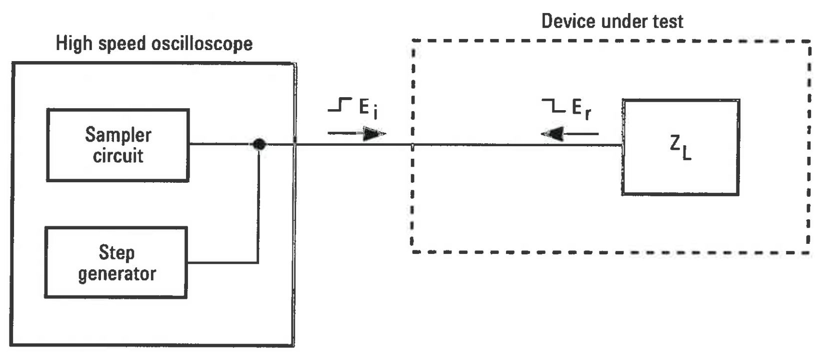


Figure 3. Functional block diagram for a time domain reflectometer

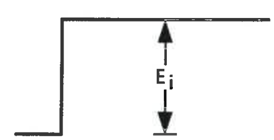


Figure 4. Oscilloscope display when $E_r = 0$

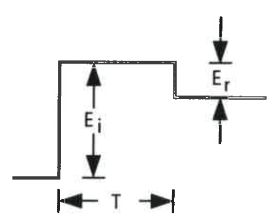


Figure 5. Oscilloscope display when $E_r \neq 0$

Locating mismatches

The reflected wave is readily identified since it is separated in time from the incident wave. This time is also valuable in determining the length of the transmission system from the monitoring point to the mismatch. Letting D denote this length:

$$D = v_\rho \bullet \frac{T}{2} = \frac{v_\rho T}{2}$$

where v_ρ = velocity of propagation

T = transit time from monitoring point to the mismatch and back again, as measured on the oscilloscope (Figure 5).

The velocity of propagation can be determined from an experiment on a known length of the same type of cable (e.g., the time required for the incident wave to travel down and the reflected wave to travel back from an open circuit termination at the end of a 120 cm piece of RG-9A/U is 11.4 ns giving $v_\rho = 2.1 \times 10^{10}$ cm/sec. Knowing v_ρ and reading T from the oscilloscope determines D. The mismatch is then located down the line. Most TDR's calculate this distance automatically for the user.

Analyzing reflections

The shape of the reflected wave is also valuable since it reveals both the nature and magnitude of the mismatch. Figure 6 shows four typical oscilloscope displays and the load impedance responsible for each. Figures 7a and 7b show actual screen captures from the 86100x DCA. These displays are easily interpreted by recalling:

$$\rho = \frac{E_r}{E_i} = \frac{Z_L - Z_o}{Z_L + Z_o}$$

Knowledge of E_i and E_r, as measured on the oscilloscope, allows Z_L to be determined in terms of Z_o, or vice versa. In Figure 6, for example, we may verify that the reflections are actually from the terminations specified.

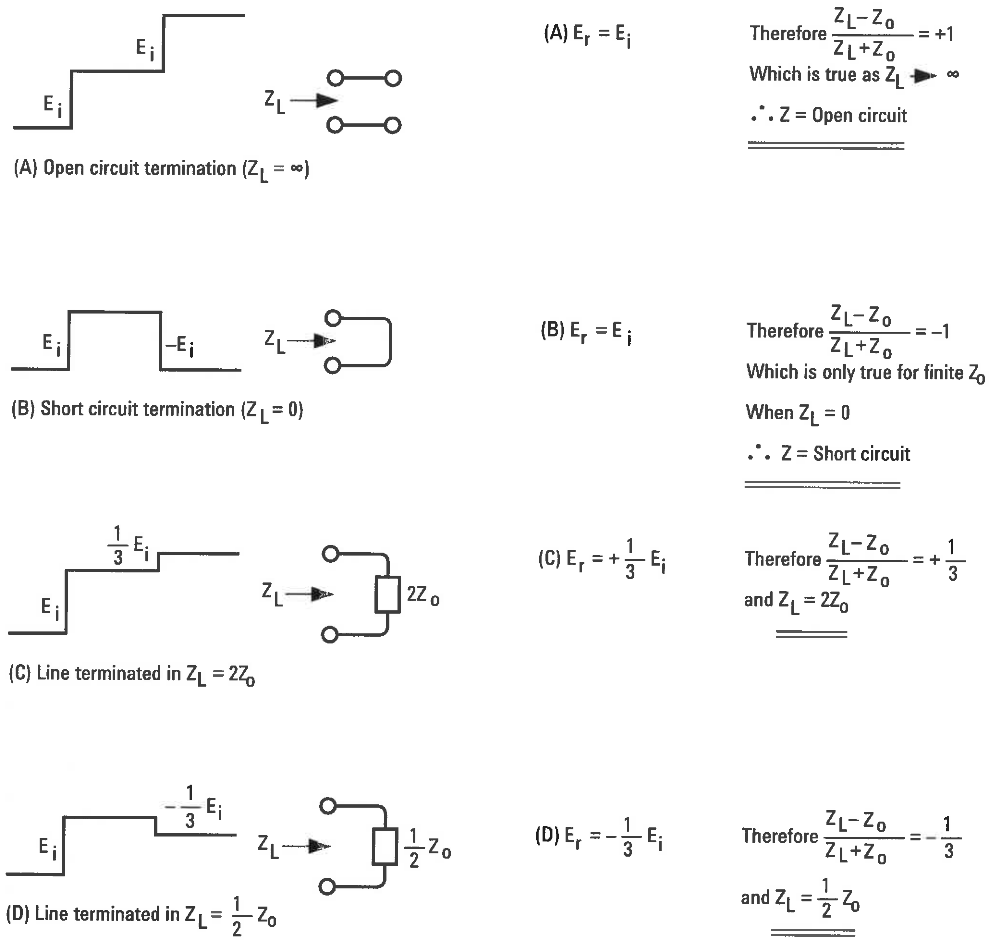


Figure 6. TDR displays for typical loads.

Assuming Z_O is real (approximately true for high quality commercial cable), it is seen that resistive mismatches reflect a voltage of the same shape as the driving voltage, with the magnitude and polarity of E_r determined by the relative values of Z_O and R_L.

Also of interest are the reflections produced by complex load imped-ances. Four basic examples of these reflections are shown in Figure 8.

These waveforms could be verified by writing the expression for ρ (s) in terms of the specific Z_L for each example:

$$\left(\text{i.e., } Z_L = R + sL\,,\ \frac{R}{1 + RC_s}\,,\ \text{etc.}\right),$$

multiplying ρ (s) by $\dfrac{E_i}{s}$ the transform of a step function of E_i, and then transforming this product back into the time domain to find an expression for $e_r(t)$. This procedure is useful, but a simpler analysis is possible without resorting to Laplace transforms. The more direct analysis involves evaluating the reflected voltage at t = 0 and at t = ∞ and assuming any transition between these two values to be exponential. (For simplicity, time is chosen to be zero when the reflected wave arrives back at the monitoring point.) In the case of the series R-L combination, for example, at t = 0 the reflected voltage is $+E_i$. This is because the inductor will not accept a sudden change in current; it initially looks like an infinite impedance, and $\rho = +1$ at $t = 0$. Then current in L builds up exponentially and its impedance drops toward zero. At t = ∞, therefore $e_r(t)$ is determined only by the value of R.

$$\left(\rho = \frac{R - Z_o}{R + Z_o} \text{ When } \tau = \infty \right)$$

The exponential transition of $e_r(t)$ has a time constant determined by the effective resistance seen by the inductor. Since the output impedance of the transmission line is Z_o, the inductor sees Z_o in series with R, and

$$\gamma = \frac{L}{R + Z_o}$$

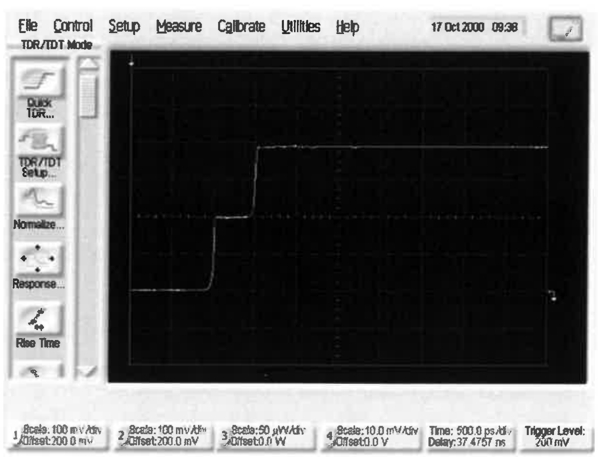


Figure 7a. Screen capture of open circuit termination from the 86100

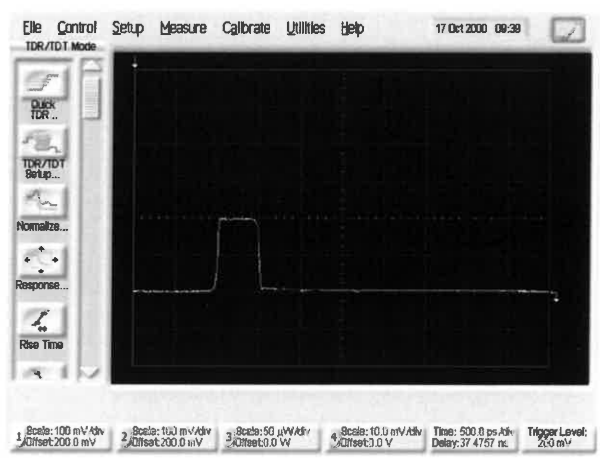


Figure 7b. Screen capture of short circuit termination from the 86100

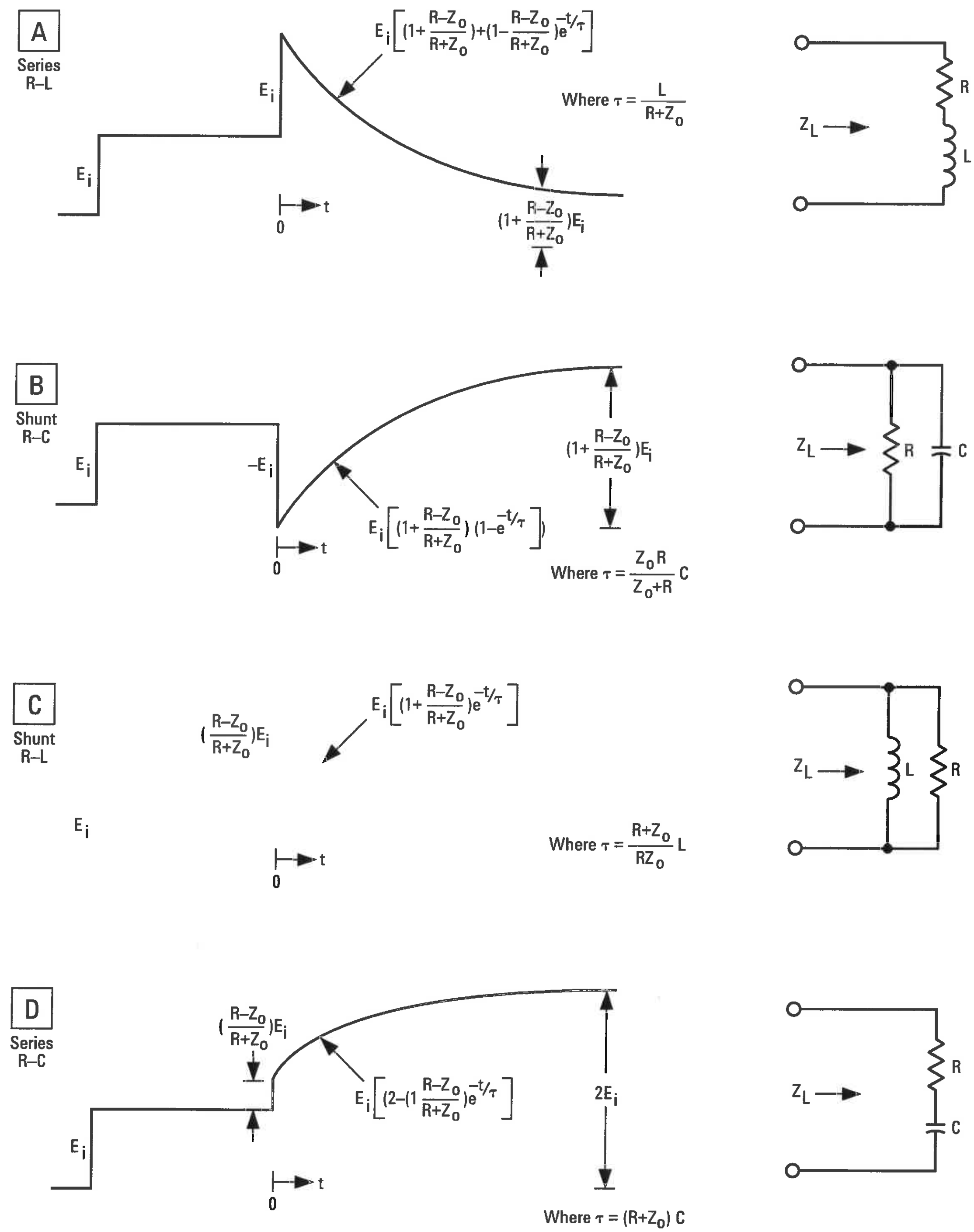


Figure 8. Oscilloscope displays for complex Z_L.

A similar analysis is possible for the case of the parallel R-C termination. At time zero, the load appears as a short circuit since the capacitor will not accept a sudden change in voltage. Therefore, $\rho = -1$ when $t = 0$. After some time, however, voltage builds up on C and its impedance rises. At $t = \infty$, the capacitor is effectively an open circuit:

$$Z_L = R \text{ and } = \frac{R - Z_o}{R + Z_o}$$

The resistance seen by the capacitor is Z_o in parallel with R, and therefore the time constant of the exponential transition of $e_r(t)$ is:

$$\frac{Z_o R}{Z_o + R} C$$

The two remaining cases can be treated in exactly the same way.The results of this analysis are summarized in Figure 8.

Discontinuities on the line

So far, mention has been made only about the effect of a mismatched load at the end of a transmission line. Often, however, one is not only concerned with what is happening at the load, but also at intermediate positions along the line. Consider the transmission system in Figure 9.

The junction of the two lines (both of characteristic impedance Z_o) employs a connector of some sort. Let us assume that the connector adds a small inductor in series with the line. Analyzing this discontinuity on the line is not much different from analyzing a mismatched termination. In effect, one treats everything to the right of M in the figure as an equivalent impedance in series with the small inductor and then calls this series combination the effective load impedance for the system at the point M. Since the input impedance to the right of M is Z_o, an equivalent representation is shown in Figure 10. The pattern on the oscilloscope is merely a special case of Figure 8A and is shown on Figure 11.

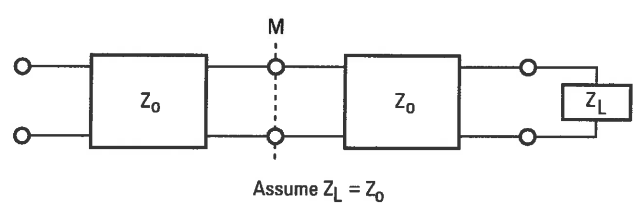


Figure 9. Intermediate positions along a transmission line

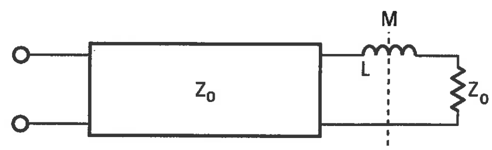


Figure 10. Equivalent representation

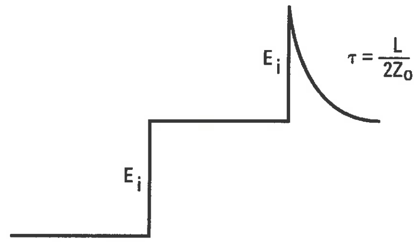


Figure 11. Special case of series R-L circuit

Evaluating cable loss

Time domain reflectometry is also useful for comparing losses in transmission lines. Cables where series losses predominate reflect a voltage wave with an exponentially rising characteristic, while those in which shunt losses predominate reflect a voltage wave with an exponentially-decaying characteristic. This can be understood by looking at the input impedance of the lossy line.

Assuming that the lossy line is infinitely long, the input impedance is given by:

$$Z_{in} = Z_o = \sqrt{\frac{R + j\omega L}{G + j\omega C}}$$

Treating first the case where series losses predominate, G is so small compared to ωC that it can be neglected:

$$Z_{in} = \sqrt{\frac{R + j\omega L}{j\omega C}} = \sqrt{\frac{L}{C}} \left(1 + \frac{R}{j\omega L}\right)^{1/2}$$

Recalling the approximation $(1 + x)^a \approx (I + ax)$ for $x < 1$, Z_{in} can be approximated by:

$$Z_{in} \approx \sqrt{\frac{L}{C}} \left(1 + \frac{R}{j2\omega L}\right) \text{ When } R < \omega L$$

Since the leading edge of the incident step is made up almost entirely of high frequency components, R is certainly less than ωL for t = 0+. Therefore the above approximation for the lossy line, which looks like a simple series R-C network, is valid for a short time after t = 0. It turns out that this model is all that is necessary to determine the transmission line's loss.

In terms of an equivalent circuit valid at t = 0+, the transmission line with series losses is shown in Figure 12.

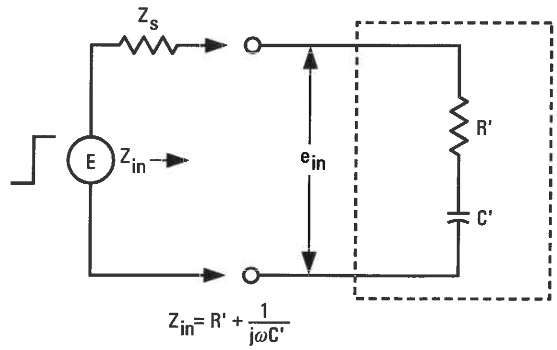


Figure 12. A simple model valid at t = 0+ for a line with series losses

The series resistance of the lossy line (R) is a function of the skin depth of the conductor and therefore is not constant with frequency. As a result, it is difficult to relate the initial slope with an actual value of R. However, the magnitude of the slope is useful in comparing conductors of different loss.

A similar analysis is possible for a conductor where shunt losses predominate. Here the input admittance of the lossy cable is given by:

$$Y_{in} = \frac{1}{Z_{in}} = \sqrt{\frac{G + j\omega C}{R + j\omega L}} = \sqrt{\frac{G + j\omega C}{j\omega L}}$$

Since R is assumed small, re-writing this expression for Y_{in}:

$$Y_{in} = \sqrt{\frac{C}{L}} \left(1 + \frac{G}{j\omega C}\right)^{1/2}$$

Again approximating the polynominal under the square root sign:

$$Y_{in} \approx \sqrt{\frac{C}{L}} \left(1 + \frac{G}{j2\omega C}\right) \text{ When } G < \omega C$$

Going to an equivalent circuit (Figure 13) valid at t = 0+,

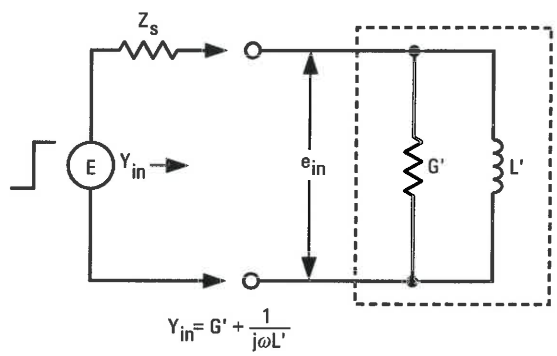


Figure 13. A simple model valid at t = 0+ for a line with shunt losses

A qualitative interpretation of why $e_{in}(t)$ behaves as it does is quite simple in both these cases. For series losses, the line looks more and more like an open circuit as time goes on because the voltage wave traveling down the line accumulates more and more series resistance to force current through. In the case of shunt losses, the input eventually looks like a short circuit because the current traveling down the line sees more and more accumulated shunt conductance to develop voltage across.

Multiple discontinuities

One of the advantages of TDR is its ability to handle cases involving more than one discontinuity. An example of this is Figure 14.

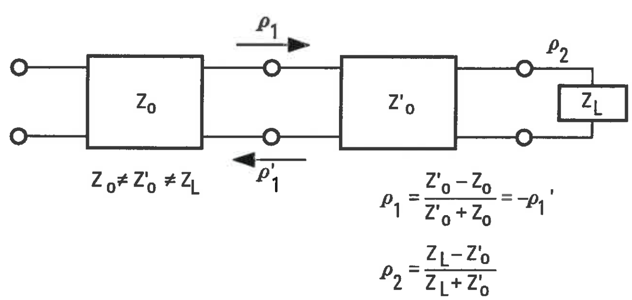


Figure 14. Cables with multiple discontinuities

The oscilloscope's display for this situation would be similar to the diagram in Figure 15 (drawn for the case where $Z_L < Z_o < Z'_o$):

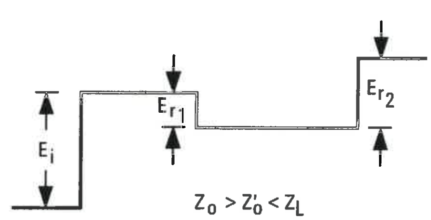


Figure 15. Accuracy decreases as you look further down a line with multiple discontinuities

It is seen that the two mismatches produce reflections that can be analyzed separately. The mismatch at the junction of the two transmission lines generates a reflected wave, E_r , where

$$E_r = \rho_1 E_i = \left(\frac{Z'_o - Z_o}{Z'_o + Z_o}\right) E_i$$

Similarly, the mismatch at the load also creates a reflection due to its reflection coefficient

$$\rho_2 = \frac{Z_L - Z'_o}{Z_L + Z'_o}$$

Two things must be considered before the apparent reflection from Z_L, as shown on the oscilloscope, is used to determine $\rho 2$. First, the voltage step incident on Z_L is $(1 + \rho_1) E_i$, not merely E_i. Second, the reflection from the load is

$$[\ \rho_2 (1 + \rho_1) E_i\ ] = E_{r_L}$$

but this is not equal to E_{r_2} since a re-reflection occurs at the mismatched junction of the two transmission lines. The wave that returns to the monitoring point is

$$E_{r_2} = (1 + \rho_1') E_{r_L} = (1 + \rho_1') [\ \rho_2 (1 + \rho_1) E_i\ ]$$

Since $\rho_1' = -\rho_1$, E_{r_2} may be re-written as:

$$E_{r_2} \quad E_{r_2} = [\ \rho_2\ (1 - \rho_1{}^2)\ ]\ Ei$$

The part of E_{r_L} reflected from the junction of

$$E_{r_L} \quad Z'_o \text{ and } Z_o \ \ (\text{i.e., } \rho_1' E_{r_L})$$

is again reflected off the load and heads back to the monitoring point only to be partially reflected at the junction of Z_o' and Z_o. This continues indefinitely, but after some time the magnitude of the reflections approaches zero.

In conclusion, this application note has described the fundamental theory behind time domain reflectometry. Also covered were some more practical aspects of TDR, such as reflection analysis and oscilloscope displays of basic loads. This content should provide a strong foundation for the TDR neophyte, as well as a good brush-up tutorial for the more experienced TDR user.

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APPENDIX - Army Corps Publication "Drilling in Earth Embankment Dams & Levees"

CECW-CE

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

ER 1110-1-1807

Regulation
No. 1110-1-1807

31 December 2014

Engineering and Design
DRILLING IN EARTH EMBANKMENT DAMS AND LEVEES

1. Purpose. This regulation establishes policy and requirements and provides guidance for drilling in dam and levee earth embankments and/or their earth and rock foundations. The primary purpose of this regulation is to prevent damage to embankments and their foundations from hydraulic fracturing, erosion, filter/drain contamination, heave, or other mechanisms during drilling operations, sampling, in-situ testing, grouting, instrumentation installation, borehole completion, and borehole abandonment.

2. Applicability. This regulation applies to all major subordinate commands (MSC), district commands, laboratories, and field operating activities having Civil Works and/or Military Program responsibilities. It applies to in-house and contracted drilling efforts for earth embankments or foundations associated with all dams and levees that have a federal interest.

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

4. References. References are listed in Appendix A.

5. Background. Drilling into, in close proximity to, or through embankment dams and levees and their foundations may pose significant risk to the structures. Water, compressed air, and various drilling fluids have been used as circulating media while drilling through earth embankments and their foundations. Although these methods have been successful in accomplishing the intended purposes, there have been incidents of damage to embankments and foundations. While using air (including air with foam), there have been reports of loss of circulation with pneumatic fracturing of the embankment as evidenced by connections to other borings and blowouts on embankment slopes. While using water and drilling mud as the circulating medium, there have been similar reports of erosion and/or hydraulic fracturing of the embankment or foundation materials. Conversely, there have been cases where heave, borehole collapse and significant disturbance have occurred while drilling in granular materials below the groundwater level. This typically has been the result of not using a proper drilling fluid to balance the water pressures in the soil or using high energy systems that induce heave in order to evacuate the cuttings. There is a delicate balance between too much induced fluid pressure that will cause hydraulic fracture and not enough fluid pressure that will result in borehole instability, heave, or significant sample disturbance. Other potential damaging effects include: creating preferential seepage paths due to improper backfilling, inadequate protection of embankment from drilling fluids during foundation rock coring, erosion and widening of cracks, and inadvertently clogging filters or drains with drilling fluid or grout. All drilling and associated activities that use fluid or other circulation or stabilization media need to be evaluated for the potential to hydraulically

This regulation supersedes ER 1110-1-1807, dated 1 March 2006

fracture the embankment or foundation. These activities include but are not limited to the use of drilling fluids, backfilling borings after completion, backfill grouting of instrumentation, backfill grouting of casings, water testing for permeability, piezometer rehabilitation, etc. The risk will vary with the selected methods and the site conditions. Every drilling operation must be well thought out and must have benefits of successful completion that confidently outweigh the risk of potential negative impacts. The following paragraphs describe the general concerns associated with each type of potential damage.

a. Hydraulic Fracturing. Excessive pressures from water, air, drilling fluid, or grout can fracture embankment and foundation materials. Hydraulic fracturing problems have occurred while drilling in embankments as evidenced by reports of loss of fluid circulation, blowouts into nearby borings, seepage of drilling fluids on the face of the embankment, and other similar situations. Hydraulic fracture can occur in both cohesive materials and cohesionless materials, and bedrock. It has been found that in soils, hydraulic fracturing can occur when the borehole pressure exceeds the lowest total confining stress (minimum principal stress σ_3) plus some additional strength. The additional strength can be approximated by the undrained strength of the soil. The minor principal confining stress (σ_3) in a normally consolidated soil with a level ground condition is typically the horizontal stress, which can be reasonably estimated. However, the minor principal confining stress in and under an embankment is difficult to determine and can vary significantly from idealized geostatic conditions. Effects from the side slope geometry, piezometric surface, abutment configuration, foundation rock geometry, embedded structures, compaction stress, and settlement history all are significant and can influence in-situ stress conditions. Typical drilling methods that use circulation fluids can quickly create induced fluid pressures that exceed the minimum confining stress. This often occurs when the return path for the fluid clogs and the induced pressures quickly increase. The use of non-pressurized stabilizing fluids is preferable, yet in some subsurface conditions, hydraulic fracture can occur under gravity pressure. Low stress zones may exist within and under embankments. It is possible for the confining stress in these locations to be much less than the gravity pressure exerted by a drilling fluid or grout. Certain embankment locations and conditions have a higher potential for hydraulic fracturing due to geometric configurations that create zones of low confining stress. Sherard 1973 and 1986 are good references that provide a comprehensive evaluation of the issues along with numerous case histories. Locations and conditions where hydraulic fracturing by drilling media is more likely to occur and have the higher potential of damaging the structure include the following:

(1) Near and over steep abutments that create low confining or tensile stress conditions.

(2) Adjacent to rock overhangs on abutments.

(3) Adjacent to buried structures or abrupt foundation geometry change that creates a differential settlement condition and a zone of lower soil stress transfer.

(4) Adjacent to conduits where narrow zones of soil backfill were placed between the structure and rock face.

(5) Dam cores that can experience more settlement than the adjacent shells.

(6) Dams in very narrow valleys. Arching keeps full confining stresses from developing.

(7) Near abutments where abrupt changes in geometry occur.

(8) In areas where the embankment is subject to differential settlement due to large differences in thickness of adjacent compressible foundation or embankment soils.

b. Erosion. The introduction of drilling fluids into cracks, either existing or formed by hydraulic fracture, can potentially cause erosion along the crack walls. This will enlarge the crack and could lead to an increased potential for internal erosion. Existing subsurface cracks are common in many dams and are often the result of differential settlement. The locations most at risk for existing cracks are typically the same areas that have low confining stress and have the highest risk for hydraulic fracture to occur.

c. Contamination of Filter/Drainage Features. In addition to hydraulic fracturing, the use of drilling fluids can pose a contamination risk for internal drainage features if the drill fluid or sealing grout migrates into and clogs the drain materials. Avoid drilling near drains or seepage blankets that may become contaminated by fluids. If drain penetration is justified, special provisions must be taken to prevent contamination. Special provisions may also be required for protecting the drainage features while backfilling the hole (such as placement of filter material through the zone of the drain or filter and installing lower and upper seals).

d. Heave and Sample Disturbance. Drilling programs that include performing in-situ tests or undisturbed sampling may require the use of drilling fluid to offset the confining stress relieved by the drilling of the hole. There have been cases where the failure to prevent stress relief or heave of granular soils below the water table have led to invalid in-situ test results and subsequently invalid interpretation of the subsurface conditions. This has occurred for both tests performed in drill holes and test performed in casings installed by methods that did not control heave or disturbance. Reclamation DSO 98-17 (1999) contains methods to deal with heaving sands while drilling and performing Standard Penetration Tests. If high quality undisturbed samples of fine grained soils are required for shear strength testing, then drilling mud may be required to prevent the soil from failing in undrained triaxial extension. See Ladd and DeGroot (2004) for a discussion on clay sample disturbance due to drilling.

6. Policy. This regulation provides guidance for investigation, maintenance, and remediation drilling in and near embankment dams and levees and/or their earth and rock foundations. including investigation planning, site preparation, borehole advancement, subsurface testing, instrumentation installation, piezometer and well rehabilitation, grouting, and borehole completion. It identifies drilling program plan requirements, restrictions on drilling fluids, drilling procedures to minimize risk of damage, borehole completion requirements, and prescribes personnel requirements, and the review and approval processes. It is the responsibility of the District Dam or Levee Safety Officer (DSO or LSO) to assure compliance with the restrictions and procedures outlined in this regulation.

a. Drilling Program Plan. An approved Drilling Program Plan (DPP) is required prior to any drilling, sampling, grouting, or any other invasive in-situ testing or exploration. This includes drilling activities related to investigation, maintenance, and remediation. When planning an investigation or remediation program, the data needs must be weighed against the potential risks of damage created by the drilling process. In general, all drilling and investigation should be targeted to obtain information related to a specific failure mode identified from a Potential Failure Mode Analysis (PFMA). For dams, the justification for drilling must include an approved recommendation from a risk assessment performed in support of the Dam Safety risk management process described in ER 1110-2-1156 Safety of Dams - Policy and Procedures. If the structure has not had a PFMA, a thorough evaluation similar to the PFMA process must be performed and presented in the DPP to show that the drilling is justified. It is paramount that all existing subsurface information is thoroughly evaluated and understood by the exploration team prior to developing a plan for additional drilling. In order to understand and communicate subsurface conditions and estimate drilling risks, the existing subsurface information must be assimilated into essential plan and section drawings showing the proposed drill holes, target sample areas and/or proposed instrumentation. For critical or complicated drilling programs the Geotechnical and Geology Community of Practice leads can be contacted to obtain recommendations for subject matter experts to assist in developing the DPP. Specific requirements for the DPP are included in Appendix B.

b. Restrictions on the Use of Drilling Fluids. All drilling programs in dams and levees should be designed to minimize the need for any drilling fluid such as air, gas, water, mud, polymers, slurries or any other drilling fluid that could pressurize the borehole soils. If the drilling objective can be performed using dry methods such as augers or sonic drilling they should be employed in lieu of methods that require fluids. If drilling fluids must be used due to the drilling objective or the subsurface conditions, the DPP must contain an analysis of the potential to cause damage and a plan that covers the measures that will be used to minimize the risk. The use of pressurized air or foam should only be considered when drilling in materials that will not transmit pressures to the soil core or other critical features or when the air pressure is reliably isolated from the borehole soils. Drilling in an open graded rockfill shell may be an example of when using air may be appropriate. All DPPs that propose the use of stabilizing or circulating fluids or other media will require additional review and approval as described in paragraph 6f.

c. Drilling Procedures. As there are many existing and potentially new methods for drilling and sampling that may be implemented on dams and levees, this regulation will not provide specific procedures. Most procedures are documented in applicable standards and reference documents. There are however, some general procedures that should be followed when using drilling fluids to limit the risk of damage.

(1) Tools should be sized and designed to minimize the likelihood of the return flow clogging. Methods that require the cuttings to flow through a small annulus between the tools or casing and the borehole wall should not be used.

(2) If possible, fluid discharges from the bit should be upward. A downward discharge increases the chance of clogging which could lead to a pressure spike. A lateral discharge into the sidewalls could lead to excessive disturbance or erosion.

(3) Lower and raise drill tools slowly to avoid pressure changes in the drill hole; this is especially important when using tools with restricted annulus space below the groundwater table as the pressure changes are more severe and can lead to suction and surging problems.

(4) Drilling feed rate must be slow enough to avoid crowding the bit and, thus, minimize the chance of inducing fracturing. The bit must be of a design such that pressure buildup is minimized.

(5) Drilling media properties, pressure, and return should be continuously monitored. A floating needle pressure valve is required to record maximum pressure spikes that can occur instantaneously and are often unnoticed.

(6) In some conditions, casing can be advanced ahead of the drilling bit to reduce the risk of hydraulic fracturing by confining the drilling fluids within the casing.

(7) When core drilling rock, the embankment or foundation soil above top of rock must be protected and isolated from the circulating drilling fluid. Fractures in the bedrock must be considered as potential flow paths in contact with the overlying soil.

(8) In situations where the presence of significant artesian pressure is suspected, which are common at the toe of dams, it may be necessary to use weighted drilling muds or raise the drill rig or install surface casing for pressure control along with the use of drilling mud. In some cases there may be a high risk of initiating internal erosion by drilling borings or excavating test pits in these areas. Emergency materials to stop progressive erosion in an excavation, a trench, or a borehole must be on site and readily available. For this situation, it is recommended to stockpile fine (C33 concrete sand) and coarse processed aggregates to filter and plug the excavation. Specific details such as height of the drill pad and amount of surface casing must be developed on a case-by-case basis dependent upon specific site conditions.

d. <u>Borehole Completion</u>. All boreholes and other penetrations (including direct push sampling, Cone Penetration Test soundings, Standard Penetration Testing, Becker Penetration Testing,) in and around embankment dams and levees must be sealed after completion. Completing a borehole by backfilling with drill cuttings is not acceptable. All boreholes and similar penetrations in the impervious portions of an embankment dam or levee and their foundations must be backfilled by tremie placed cement-bentonite grout or bentonite pellets/chips. The DPP must address the possibility of confined and separate ground water aquifers and demonstrate safe completion which avoids cross-contamination and leakage. The grout must be designed to obtain strength equal to or greater than the soil. Note that some instrumentation installations may require additional considerations for the grout strength. Gravity grouting techniques should be used for backfilling boreholes. For borings that penetrate zones with low confining stress it is possible to induce hydraulic fracturing from the gravity pressure. When grouting borings in these locations or if significant grout losses are observed, the grout backfilling should be done in stages allowing the grout to set between stages. For pervious portions of the dam or levee, the borehole must be backfilled by tremie placement of granular materials that are sized to provide drainage without being susceptible to migration through the pervious embankment or foundation materials or segregation during placement. Lutenegger, et.al. (1995) is a good source for borehole backfill guidelines. Special procedures and materials may be required for installation of instrumentation in boreholes.

e. <u>Drilling Personnel</u>. Drill rig operators must have a minimum of 5 years experience drilling with the equipment and procedures described in the drilling program. The drill rig operator must also be familiar with these guidelines. All drilling activities on USACE dams or levees must be conducted in the presence of a geotechnical engineer that is a licensed professional engineer or a licensed professional geologist who will be responsible for maintaining the integrity of the structure.

f. Approval Requirements. Drilling Program Plans must be reviewed and approved by the District Dam Safety Officer (Dams) or Levee Safety Officer (Levees). If any drilling fluid or other stabilizing or circulating media is proposed, a technical review performed by the Geotechnical and Materials Community of Practice (G&M CoP) Standing Committee on Drilling and Instrumentation is required. The plan will then require approval from the District DSO/LSO pending satisfactory resolution of the technical review comments. The Standing Committee on Drilling and Instrumentation will be chaired by the G&M CoP Lead, co-chaired and managed by the Risk Management Center, and staffed with G&M CoP experts.

g. Reporting. All incidents of damage or potential damage related to drilling and associated activities for dams must be reported following procedures outlined in Chapter 13 Reporting Evidence of Distress in Civil Works Structures of ER 1110-2-1156 Safety of Dams- Policy and Procedures. Damage in levees must be reported to the Levee Safety Officers and Levee Safety Program Managers in the District, MSC, and Headquarters.

h. Exemptions. Drilling required for immediate emergency measures where delays required to develop the DPP and obtain approvals would result in unacceptable risk of damage or failure, may be exempted from the requirements to prepare a DPP by the District DSO/LSO. Emergency drilling should be appropriately expedited but should follow the general guidelines presented in this regulation. No other exemptions or deviations from these requirements may be made.

7. Environmental Operating Principles. The user of this ER, as a member of a Project Delivery Team, is responsible for seeking opportunities to incorporate the Environmental Operating Principles (EOPs) wherever possible. A listing of the EOPs is available at: http://www.usace.army.mil/Missions/Environmental/EnvironmentalOperatingPrinciples.aspx.

FOR THE COMMANDER:

William H. Graham

WILLIAM H. GRAHAM
COL, EN
Chief of Staff

2 Appendices
Appendix A - References and Resources
Appendix B - Drilling Program Plan

APPENDIX A

References and Resources

Drilling Procedures

EM 1110-1-1804 Geotechnical Investigations (including Appendix F EM 1110-1-1906 Soil Sampling).

EM 1110-2-1908 Instrumentation of Embankment Dams and Levees.

EM 1110-2-1914 Design, Construction and Maintenance of Relief Wells.

EM 1110-2-2300 General Design and Construction Considerations for Earth and Rock-Fill Dams.

EM 1110-2-3506 Grouting Technology.

UFGS-02210 (August 2004) Subsurface Drilling, Sampling, and Testing.

ASTM D1452-09 Standard Practice for Soil Exploration and Sampling by Auger Borings, 2009.

ASTM D1586-11 Standard Test Method for Penetration Test (SPT) and Split-Barrel Sampling of Soils, 2011.

ASTM D1587-08 Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes, 2012.

ASTM D2113-08, Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation, 2008.

ASTM D5781/D5781M-13, Standard Guide for Use of Dual-Wall Reverse-Circulation Drilling for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices, 2013.

ASTM D5782-95, Standard Guide for Use of Direct Air-Rotary Drilling for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices, 2012.

ASTM D5783-95, Standard Guide for Use of Direct Rotary Drilling with Water-Based Drilling Fluid for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices, 2012.

ASTM D5872-95, Standard Guide for Use of Casing Advancement Drilling Methods for Geoenvironmental Exploration and Installation of Subsurface Water-Quality Monitoring Devices, 2006.

ASTM D5875-95, Standard Guide for Use of Cable-Tool Drilling and Sampling Methods for Geoenvironmental Exploration and Installation of Subsurface Water-Quality Monitoring Devices, 2006.

ASTM D5876-95, Standard Guide for Use of Direct Rotary Wireline Casing Advancement Drilling Methods for Geoenvironmental Exploration and Installation of Subsurface Water-Quality Monitoring Devices, 2012.

ASTM D6066-11, Determining the Normalized Penetration resistance for Sands for Evaluation of Liquefaction Potential, 2011.

ASTM D6151-08, Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling, 2008.

ASTM D6286-12, Standard Guide for Selection of Drilling Methods for Environmental Site Characterization, 2012.

ASTM D6914-04, Standard Practice for Sonic Drilling for Site Characterization and the Installation of Subsurface Monitoring Devices, 2010.

Standard Penetration Test: Drillers/Operators Guide, Report Number DSO 98-17, J. Farrar, Bureau of Reclamation, Dam Safety Office, Denver, Colorado, May 1999.

Earth Manual, Parts I & II, Third Edition, Bureau of Reclamation, Denver, Colorado.

Australian Drilling Manual, Third Edition, Australian Drilling Industry Training Committee, NSW 2113 Australia, 1992.

Drillers Manual, accessed at www.nda4u.com, National Drilling Association, Brunswick, Ohio 44212.

Lutenegger A.J., D.J. Degroot, C. Mizra and M. Bozozuk, "Recommended Guidelines for Sealing Geotechnical Exploratory Holes", Report 378, National Cooperative Highway Research Program, Transportation Research Board, National Academy Press, Washington, DC, 1995.

Hydraulic Fracture

Albritton, J., Jackson, L., and Bangert, R., "Foundation Grouting Practices at Corps of Engineers Dams", Technical Report GL-84-13, US Army Corps of Engineers, October 1984.

Alfaro, M.C., and Wong, C.K., "Laboratory studies of fracturing of low-permeability soils", Canadian Geotechnical Journal, 38, 303-315, 2001.

Andersen, K.H., Rawlings, C.G. Lunne, T.A., and By, T.H., "Estimation of hydraulic fracture pressure in clay", Canadian Geotechnical Journal, 31, 817-828, 1994.

Bjerrum, L., Nash, J. K. T. L., Kennard, R.M. & Gibson, R.E., "Hydraulic fracturing in field permeablity testing", Geotechnique, 22, 319-32, 1972.

Bozozuk, M., "Minor principal stress measurements in marine clay with hydraulic fracture tests", Proceedings, Engineering Foundation Conference on Subsurface Exploration for Underground Excavation and Heavy Construction, Henniker, N.H., August 1974.

Calcagno, Frank, Jr., USBR, "Hydraulic Fracture Study of the Tiber Spillway Cofferdam", AEG Newsletter 26/4 October 1983.

Casagrande, A. and Covaarrubias, S.W., "Cracking of earth and rockfill dams, tension zones in embankments caused by conduits and cutoff walls", Contract Report S-70-7, U.S. Army Engineer Waterways Experiment Station, July 1970.

Chang, H., "Hydraulic fracturing in particulate materials", Doctoral Thesis, Georgia Institute of Technology, November 2004.

Chen, Yu-jiong, and Zhang, Shu-lu, "Test embankment of fracture grouting", Journal of Geotechnical Engineering, Vol. 115, No. 11, November 1989.

Clough, R.W. and Woodward, R.J. III, "Analysis of embankment stresses and deformations", Journal of Soil Mechanics and Foundations Division, Proceedings of ASCE, Vol. 93, No. SM4, July 1967.

Elwood, D., and Moore, I., "Hydraulic fracture experiments in sand and gravel and approximations for maximum allowable mud pressure", North American Society for Trenchless Technology, No Dig Show, Mar-Apr 2009.

Hamouche, K.K., Leroueil, S., Roy, M., and Lutenegger, A.J., "In situ evaluation of K_0 in eastern Canada clays", Canadian Geotechnical Journal, vol 32, pgs 677-688, 1995.

Independent panel to review cause of Teton Dam failure, 1976, Failure of Teton Dam: Report to the U.S. Department of Interior and State of Idaho.

Kulhawy, F.H., and Duncan, J.M., "Stresses and movements in Oroville dam", Journal of Soil Mechanics and Foundations Division, Proceedings of ASCE, Vol. 98, No. SM7, July 1972.

Lo, K.Y. and Kaniaru, K., "Hydraulic fracture in earth and rock-fill dams", Canadian Geotechnical Journal, Vol 27, 496-506, 1990.

McCook, D.K, and Grotrian, K.O., "Using SIGMA/W to predict hydraulic fracture in an earthen embankment", Proceedings: Dam Safety, ASDSO, September 2010.

Mori, A. and Tamura, M., "Hydrofracturing pressure of cohesive soils", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 27, No. 1, 14-22, Mar 1987.

Schmertmann, J. H., "Measure and use of the insitu lateral stress", The Practice of Foundation Engineer, Department of Civil Engineering, Northwestern University, 1985.

Seed, H.B., and Duncan, J.M., 1981, “The Teton Dam - a retrospective review”, Proceedings of the Tenth international Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweden, June 1981, p. 219-238.

Sherard, J.L., “Embankment Dam Cracking,” Embankment Dam Engineering, S. Poulos and R. Hirschfeld, Eds., John Wiley and Sons, New York, N.Y., 1973,pp. 272-353.

Sherard, J.L., “Loss of water in boreholes in impervious embankment sections”, Proceedings, 10th ICOLD Congress, Montreal, Vol. VI, 1970, 377-381.

Sherard, J.L., Decker, R.S. and Ryker, N.L., “Hydraulic fracturing in low dams of dispersive clay,” Proceedings of the Specialty Conference on Performance of Earth and Earth-Supported Structures, ASCE, June, 1972, Vol. 1, Part I, pp. 563-590.

Sherard, James L., “Hydraulic fracturing in embankment dams,” ASCE Journal of Geotechnical Engineering, Volume 112, No. 10, October, 1986, pp. 905-927.

Staheli, K., Price, C.G., and Wetter, L., “Effectiveness of hydrofracture prediction for HDD design”, North American Society for Trenchless Technology, No Dig Show, May, 2010.

US Army Corps of Engineers, “Foundation completion report Patoka lake dam, Indiana”, Appendix E, Analysis of Grouting Effectiveness and Distribution as Observed During Excavation, July 1979.

US Army Corps of Engineers, “Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling, Technical Report CPAR-GL-98-1, April 1998.

Xia, H. and Moore, I.D., “Estimation of Maximum Mud Pressure in Purely Cohesive Material during Directional Drilling, Geomechanics and Geoengineering: An International Journal, Vol. 1, No.1, 3-11. 2006.

Yanagisawa, E. and Panah, A.K., “Two dimensional study of hydraulic fracturing criteria in cohesive soils”, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 34, No. 1, 1-9, Mar. 1994.

Sample Disturbance

Ladd, C. C., and D.J DeGroot, 2004 (revised). “Recommended Practice for soft ground site characterization, Arthur Casagrande lecture, Proc. 12th Pan-American Conf. on Soil Mech. and Geot. Engineering.

Filter Design

Filter for Embankment Dams – Best Practices for Design and Construction, Federal Emergency Management Agency, http://www.fema.gov/library/viewRecord.do?fromSearch=fromsearch&id=4943, December 2011.

Dam Safety Guidance

ER 1110-2-1156 Safety of Dams- Policy and Procedures.

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APPENDIX B

Drilling Program Plan

An approved drilling program plan (DPP) is required for any exploration or remedial drilling (including grouting) work to occur in or near an embankment dam or levee or their foundations. When drilling is justified, an exploration team must be formed to determine and document the drilling program components required to adequately and safely address the project needs. The exploration team must thoroughly discuss the drilling program to ensure that the program minimizes risk and meets the project goals. The drilling program must be prepared by experienced geotechnical engineers and/or engineering geologists familiar with subsurface exploration techniques and methods, with advice from drilling specialists. The Lead engineer on the exploration team must be a registered professional engineer. This section describes the basic information that must be developed and included in the drilling program.

a. Objective and Justification. The objective of the drilling program must be clearly summarized including the purpose of the drilling and how the borings, samples, testing, instrumentation, etc. will be used. The need for the drilling must be thoroughly justified. Drilling should be minimized by first utilizing non-destructive methods including parametric analysis, the use of published correlations, and non-destructive geophysical testing. The justification must include documentation that shows the purpose is based obtaining information related to potential failure modes identified in an approved risk assessment in support of the dam or levee safety program. If an approved PFMA or risk assessment has not been performed, the exploration team must perform a thorough evaluation similar to the PFMA process and present a valid justification demonstrating that the drilling is required to obtain information related to a credible potential failure mode.

b. Exploration Team. List members of the exploration team used in developing the DPP. Include name, organization, title, registration, and years of experience.

c. Existing Information Review. In order to understand subsurface conditions, justify additional drilling, and estimate drilling risks, all relevant existing information must be assimilated and reviewed by the exploration team and then concisely summarized in the DPP. Information review typically includes, but is not limited to:

(1) Geologic mapping, boring logs, driller's notes, and reports portraying information from previous investigations and construction.

(2) Geotechnical files and reports including Site Characterization Reports.

(3) Foundation Completion Reports.

(4) Embankment Construction Reports.

(5) Periodic Inspection or Periodic Assessment Reports.

(6) As-built drawings.

(7) Archived records.

(8) Other construction reports.

(9) Construction photos for both original embankment construction and any subsequent construction.

(10) Instrumentation plans, data, and reports.

(11) Project records available in district and project offices.

d. Essential Geologic and Engineering Drawings. The DPP must include a set of drawings depicting the current understanding of subsurface conditions, as they relate to the proposed work. This detailed set of foundation and embankment drawings typically requires a plan showing all previous and proposed subsurface investigation locations, profile drawings, and sections of the embankment in the areas proposed for exploration. The sections must be drawn to scale with no vertical exaggeration and must show the proposed borings along with all available factual information and appropriate geologic or engineering interpretations. The drawings should be updated regularly during the drilling operations to show conditions encountered and adjust geologic interpretations to help guide the program. The information on the plan, profile and sections must be detailed and include a summary of all data significant to the analytical and exploration needs such as:

(1) Embankment zones, including added berms, blankets, filters, and drains.

(2) Details of subsurface material classification.

(3) Geologic contacts and continuity interpretations supported by all nearby drilling and sampling details.

(4) Depth of the top of rock and all other zones of importance.

(5) Piezometer locations showing screened influence zones and recorded piezometric levels tied to the reservoir water level.

(6) Other instrumentation such as inclinometers, movement monuments, etc., shown in the context of the foundation geology contacts and interpretations.

(7) SPT blow counts or other test results defining engineering properties.

(8) Geophysical data, where useful (e.g. cross hole shear wave velocity profiles).

(9) Estimated extent of any zones of interest, including natural and made-made (grout holes).

(10) Seepage areas tied to geologic units, where possible.

(11) Location of all structures, including seepage control features, outlet works, etc.

(12) Location and types of any distress features (seepage, wet spots, sand boils, sinkholes, etc.).

Maintaining updated geologic sections and a plan during the drilling operations is important for making exploration changes and for responding to unusual or unexpected conditions or events. The process for accomplishing this must be outlined in the drilling program.

e. Drilling Scope and Methodology. The drilling program must include a summary of the scope and methods that will be used, including the following:

(1) Number and location of proposed borings.

(2) Utilities, surface and underground obstacles, and accessibility.

(3) Materials expected to be drilled, sampled, and tested.

(4) Depth, diameter, bearing, and inclination of borings.

(5) Required sample type (disturbed or undisturbed), size, location, and reason for sampling.

(6) Proposed laboratory testing.

(7) Drilling, sampling, and testing methods.

(8) Details of the proposed tools and drilling equipment.

(9) Instrumentation and borehole completion requirements (influence zone, seals, etc.). Drill rig operators: Name and years of experience.

(10) Field Supervision Personnel: Name, organization, title, registrations, years of experience.

(11) Personnel responsible for logging materials and assuring geologic drawings are updated regularly during the drilling program.

f. Risk Evaluation. Include an evaluation of the risk of hydraulic fracturing, erosion, contamination of drainage features, heave, or any other damage. This should include:

(1) A detailed description of any drilling fluid used including details on the circulation system, locations where fluid will contact soil, and circulation pressures that will be used.

(2) Monitoring needs during drilling, and a contingency plan if loss of drilling fluid or other complications are observed during drilling.

(3) Measures to minimize the risk of damage to the dam or foundation.

(4) Measures to prevent the possibility of cross-contamination and leakage from confined and separate ground water aquifers.

(5) Measures to prevent drill contact with structural features, such as conduits.

(6) Nearby instruments whose behavior will be monitored during the investigation and the expected response including threshold and limit values, and contingency plans for unexpected response.

(7) An emergency action plan including a list of emergency equipment and supplies to have onsite (phone/radio, filter materials, grout materials, etc.).

g. <u>DSO/LSO Certification</u>. Provide a certification page with the signature of the appropriate DSO/LSO. The certification must state: This Drilling Program Plan has been developed and reviewed by experienced professionals and is in compliance with all the requirements of ER 1110-1-1807. The proposed actions are justified and have been developed to minimize the likelihood of damage to the existing structure.

Federal Energy Regulatory Commission
Division of Dam Safety and Inspections

GUIDELINES FOR DRILLING
IN AND NEAR EMBANKMENT DAMS
AND THEIR FOUNDATIONS

Version 3.1 – Approved for Public Release
June 2016

GUIDELINES FOR DRILLING IN AND NEAR EMBANKMENT DAMS AND THEIR FOUNDATIONS

TABLE OF CONTENTS

1.0 INTRODUCTION/PURPOSE
- 1.1 Objective
- 1.2 Scope

2.0 BACKGROUND

3.0 PLANNING/PROJECT INFORMATION

4.0 DRILLING ACTIVITIES
- 4.1 General
- 4.2 Drilling Hazards
  - 4.2.1 Hydraulic Fracturing
  - 4.2.2 Artesian Conditions/Blowout
  - 4.2.3 Erosion
  - 4.2.4 Contamination of Filter/Drainage Features
  - 4.2.5 Heave and Sample Disturbance
- 4.3 Drilling Methods
- 4.4 In-Situ Testing/Sampling
- 4.5 Borehole Completion
- 4.6 Drilling Personnel
- 4.7 Other Considerations
- 4.8 Evaluation of Potential Risks

5.0 DRILLING PROGRAM PLAN (DPP)

6.0 REPORTING REQUIREMENTS

7.0 REFERENCES

Appendix A - Additional Hydraulic Fracturing References
Appendix B - Drilling Program Plan (DPP) Outline for Drilling in and Near Embankment Dams

GUIDELINES FOR DRILLING IN AND NEAR EMBANKMENT DAMS AND THEIR FOUNDATIONS

1.0 INTRODUCTION/PURPOSE

1.1 Objective

The primary purpose of this document is to provide guidance for drilling in and near embankment dams and their foundations. Of special emphasis is the prevention of damage to the embankment, structures, and their foundations from hydraulic fracturing, heave, erosion, filter/drain contamination, or other mechanisms during drilling-related activities.

The need for any investigation (drilling, testing, etc.) at a dam site should have been presented to and accepted by FERC prior to developing an investigation program requiring drilling activities in or adjacent to a dam. In addition, it should have been demonstrated that any potential damage to the structure created by the drilling and associated processes is outweighed by the need for the drilling data. It is not the purpose of this document to provide an all-inclusive guidance or best practices on considerations for the development of a subsurface exploration or investigation program for a dam.

A guiding principle inherent in any potential dam investigation or testing is DO NO HARM. In developing investigation plans it is important to identify the potential risks and develop and implement plans to mitigate, manage, or avoid those risks.

These guidelines are appropriate for FERC-regulated embankment dam or other earthen water retaining structures (levee, canal embankment, etc.) – any earthen structure that's responsible for holding back water or serves to provide direct support to the feature or element that is holding back the water, including its foundation.

Much of the information in Sections 3.0 and 4.0 of these guidelines have been taken from or modified from the following documents:

1. U.S. Army Corps of Engineers, "Drilling in Earth Embankments and Levees", ER 1110-1-1807, Washington, DC, December 31, 2014.

2. Bureau of Reclamation, "Guidelines for Drilling and Sampling in Embankment Dams", Denver, CO, August 2010.

1.2 Scope

Much of the information contained in this guideline has principles and applications to other forms and purposes of investigation, maintenance, construction, modification, or other activity that physically penetrates the dam or foundation, including:

- Test pits/trenches
- Drilling holes/borings and probes

- Dynamic loads/pulses/blasts
- Excavations, including grading/regrading and foundation installations/construction
- Utility installation, including buried conduits, utility vaults, utility poles, etc.
- Concrete removal/demolition
- Drain and relief well cleaning/maintenance
- Toe drain/drainage feature modifications/repair
- Penetrations, including conduits, horizontal drilling activities, etc.
- Grouting or other pressure injection/testing activities
- Removal of large vegetation, trees, and root balls

These apply to any area subject to seepage pressures, stability influences or have the potential to cause harm to the water retaining structure or its foundation.

2.0 BACKGROUND

There is a very real potential for damaging structures during the drilling process if these guidelines are not followed. Damage created by hydraulic fracturing during the drilling process (use of inappropriate drilling methods), improper in-situ sampling techniques, and/or unacceptable methods of completing (backfilling) borings can open seepage paths which could create conditions conducive to internal erosion (piping) and ultimately dam failure. Although not particularly well documented, there are a number of case histories that have highlighted the potential dangers that can happen as the result of improper planning, using improper drilling methods in dams, not having the appropriate drilling equipment and contingency plans, not having knowledgeable field staff present on-site during the drilling operations, and other factors (France, 2002).

There is also some not well publicized guidance on precautions in developing investigation programs, precautions on appropriate drilling methods, and other 'rules of thumb' that are important to consider and others that should be avoided.

Drilling in embankments often does not provide conclusive data related to seepage and piping problems within a structure. The chance of finding a disturbed zone in a dam by drilling is small, and there could be great risk. Piezometers can be installed to monitor seepage problems, but they are only effective if the problem area is known. One case for drilling into embankments could be to collect samples to evaluate filter criteria of transition zones within the structure. This can be accomplished with shallow drilling, preferably above the phreatic surface in the dam and sometimes at angles into the structure to target transition zones. Holes could be drilled from the crest or downstream shell of the structure, index tests performed, and soil samples obtained. Care must be taken during drilling to be sure that internal drainage features are not damaged or contaminated. If drilling must be performed in a dam subject to seepage and piping problems, seepage flows need to be monitored continuously during the investigation, and drilling fluids need to be controlled as discussed in the section on drilling.

Dams with seepage problems may require investigation to determine the condition, location, or even whether drains exist in the structure. In these cases, test pit excavations may be attempted. However, the possibility for piping of the foundation into an excavation or drill hole could exist and should be carefully assessed. Some dams already may have evidence of critical gradient development at the toe or into drains or manholes. Drilling at the toe of the dam is risky even if seepage is not evident. If there is concern about the occurrence of piping, a contingency plan must be developed. For example, for test pitting at the toe, if critical gradient piping is a concern, materials to stop progressive erosion in the trench must be ready. For this situation, it is recommended to stockpile fine (C33 concrete sand) and coarse processed aggregates and geotextiles at or near the site to filter and plug the excavation. If drill holes must be advanced under a critical gradient condition, one should consider the construction of drill berms at the toe.

Liquefaction investigations often require drilling through the shell or crest of a dam to perform standard penetration tests (SPTs) in embankment core and/or unconsolidated foundation materials under the structure. Testing can also be performed at the downstream toe, but these soils often are not consolidated like those under the dam, and at times it is preferable to test the material under the structure. However, holes are often drilled in alluvium at the toe of a structure. Materials also can be investigated in accessible test pits to evaluate the density of the soil. In some cases, drilling can be performed from the crest of the dam as long as the cutoff trench or wall is not penetrated. Access roads may be required on the downstream slope or, in rarer occurrences, on the upstream slope if water levels allow.

The preferred method of determining SPT results in loose sands below the water table is by fluid rotary drilling where the mud pressures and hydrostatic forces can be used to stabilize the sands. However, in locations that include concerns with possible hydraulic fracturing, use of hollow-stem augers (HSA) is preferred.

In 2000, a FEMA-sponsored workshop was convened with a group of experts with respect to dam safety issues associated with seepage through embankments and their foundations (FEMA, 2000). As part of that workshop, the participants offered the following recommendations relative to the investigation and monitoring of seepage problems and concerns:

- Although actual investigation practices vary widely, it was the consensus of the workshop participants that the recommended state-of-the-practice should be that drilling should not be done in the core of an existing embankment dam unless absolutely necessary, and then only with carefully planned precautions and dry drilling (e.g., auger) methods. The risk of hydraulic fracturing is too great to support drilling in the core without appropriate justification.

- It was the consensus of the workshop participants that drilling or test pitting should not be done at the downstream toe of a dam with water stored in the reservoir, without contingency plans and stockpiling of weighted filter materials (e.g., sand and gravel) to be used in the event of a seepage incident. It is also essential that such explorations be completed with the on-site presence of experienced personnel with the knowledge to react appropriately to any seepage incidents that may occur.

- It was the consensus of the workshop participants that they generally advised against installing piezometers in an embankment core, unless there were very compelling reasons for the instruments. The workshop participants felt that, in most cases, piezometers in the core do not provide significant additional understanding of the performance of the dam beyond that which can be obtained from piezometers in the upstream and downstream shells, which are much safer locations for the instruments.

- Piezometers are tools whose careful installation and subsequent data interpretation, in conjunction with other investigative techniques, may provide valuable information in diagnosing seepage conditions. However, the limitations of what the piezometers record must be recognized, and the piezometer data must be used in conjunction with other information (e.g., seepage rates, seepage locations, etc.) to correctly diagnose seepage conditions. Since piping channels in embankments are often relatively long, narrow features, it is highly unlikely that piezometers will be located at exactly the correct locations to provide direct data regarding the piping phenomenon.

3.0 PLANNING/PROJECT INFORMATION

When planning an investigation program, the first consideration is if the need for the data to be collected justifies the cost and potential risk to the structure created by the data collection process. A determination of potential consequences if no action is taken should be made. These consequences should include both risk and likelihood for worsening conditions, which could drive up future cost of remediation if required. When and where possible, the determination of consequences should be performed with available data. However, a scaled down investigation program may be required before an adequate assessment can be performed.

If data collection is justified, a multidiscipline exploration team should be formed to determine exploration components required to adequately address the data needs. The exploration team should consist of engineers, geologists, and others with the requisite knowledge and experience in planning and conducting field exploration programs for dams. The exploration team should thoroughly discuss data needs and investigation plans to ensure compatibility.

A thorough search of all available records should precede any investigation program. Sources of information that could be useful in evaluating the need to collect additional data include:

- Geologic mapping, logs, and reports from previous investigations and construction
- Owner and FERC project files
- Supporting Technical Information (STI) document
- Current and past consultant files
- Records of design and construction, including photographs
- Archived records
- Project records at field offices and at the project site.

The exploration program should consider:

- Purpose of the investigation
- Cost of the exploration
- Required sample type and size (disturbed or undisturbed)
- Acceptable drilling and investigative methods

- Depth, diameter, and inclination of drilling required
- Materials to be drilled and sampled
- Utilities, surface and underground obstacles, and accessibility
- Location of any seepage cutoff walls, blankets and drainage features and pipes
- Dam foundation geometry and drilling hazards
- Instrumentation and completion requirements

The investigation may also require clearances, permits, and traffic control plans. The investigation schedule must allow time to obtain clearances and permits. In most cases, National Environmental Policy Act (NEPA) compliance activities will be required. Under the National Historic Preservation Act, some sites may require inspection by an archeologist and a permit from the State Historic Preservation Officer (SHPO).

4.0 DRILLING ACTIVITIES

4.1 General

Drilling into, in close proximity to, or through dams and their foundations may pose significant risk to the structures. Water, compressed air, and various drilling fluids have been used as circulating media while drilling through dams and their foundations. Although these methods have been successful in accomplishing the intended purposes, there have been incidents of damage to embankments and foundations (Sherard, 1973). While using air (including air with foam), there have been reports of loss of circulation with pneumatic fracturing of the embankment as evidenced by connections to other borings and blowouts on embankment slopes. While using water and drilling mud as the circulating medium, there have been similar reports of erosion and/or hydraulic fracturing of the embankment or foundation materials. Conversely, there have been cases where heave, borehole collapse, and significant disturbance have occurred while drilling in granular materials below the groundwater level. This typically has been the result of not using a proper drilling fluid to balance the water pressures in the soil or using high energy systems that induce heave in order to evacuate the cuttings. There is a delicate balance between too much induced fluid pressure that will cause hydraulic fracture and not enough fluid pressure that will result in borehole instability, heave, or significant sample disturbance. Other potential damaging effects include: creating preferential seepage paths due to improper backfilling, inadequate protection of embankment from drilling fluids during foundation rock coring, erosion and widening of cracks, and inadvertently clogging filters or drains with drilling fluid or grout.

All drilling and associated activities that use fluid or other circulation or stabilization media need to be evaluated for the potential to hydraulically fracture the embankment or foundation. These activities include but are not limited to the use of drilling fluids, backfilling borings after completion, backfill grouting of instrumentation, backfill grouting of casings, water testing for permeability, piezometer rehabilitation, etc. The risk will vary with the selected methods and the site conditions. Every drilling operation must be well thought out and must have benefits of successful completion that confidently outweigh the risk of potential negative impacts.

4.2 Drilling Hazards

The following is a brief discussion of some common drilling hazards that must be considered, evaluated, and mitigated for in developing and implementing an exploration program.

4.2.1 Hydraulic Fracturing

Excessive pressures from water, air, drilling fluid, or grout can fracture embankment and foundation materials. Hydraulic fracturing problems have occurred while drilling in embankments as evidenced by reports of loss of fluid circulation, blowouts into nearby borings, seepage of drilling fluids on the face of the embankment, and other similar situations. Hydraulic fracture can occur in both cohesive materials and cohesionless materials, and bedrock. It has been found that in soils, hydraulic fracturing can occur when the borehole pressure exceeds the lowest total confining stress (minimum principal stress, $\sigma 3$) plus some additional strength (Sherard, 1986). The additional strength can be approximated by the undrained shear strength of the soil. The minor principal confining stress ($\sigma 3$) in a normally consolidated soil with a level ground condition is typically the horizontal stress, which can be reasonably estimated. However, the minor principal confining stress in and under an embankment is difficult to determine and can vary significantly from idealized geostatic conditions. Effects from the side slope geometry, piezometric surface, abutment configuration, foundation rock geometry, embedded structures, compaction stress, and settlement history all are significant and can influence in-situ stress conditions. Typical drilling methods that use circulation fluids can quickly create induced fluid pressures that exceed the minimum confining stress. This often occurs when the return path for the fluid clogs or blocks off and the induced fluid pressures quickly increase. The use of non-pressurized stabilizing fluids is preferable, yet in some subsurface conditions, hydraulic fracture can occur under gravity pressure. Low stress zones may exist within and under embankments. It is possible for the confining stress in these locations to be much less than the gravity pressure exerted by a drilling fluid or grout.

Certain embankment locations and conditions have a higher potential for hydraulic fracturing due to geometric configurations that create zones of low confining stress. Sherard 1973 and 1986 are good references that provide a comprehensive evaluation of the issues along with numerous case histories. Locations and conditions where hydraulic fracturing by drilling media is more likely to occur and have the higher potential of damaging the structure include the following:

- Near and over steep abutments that create low confining or tensile stress conditions.
- Adjacent to rock overhangs on abutments.
- Adjacent to buried structures or abrupt foundation geometry change that creates a differential settlement condition and a zone of lower soil stress transfer.
- Adjacent to conduits where narrow zones of soil backfill were placed between the structure and rock face.
- Dam cores that can experience more settlement than the adjacent shells.
- Dams in very narrow valleys. Arching keeps full confining stresses from developing.
- Near abutments where abrupt changes in geometry occur.

- In areas where the embankment is subject to differential settlement due to large differences in thickness of adjacent compressible foundation or embankment soils.

Accurately estimating in-situ embankment stresses can be difficult for the conditions listed above. In some cases, it may be helpful to calculate static stresses including seepage forces within the embankment. The results of such computations can aid in evaluating the maximum applied drilling fluid pressures or static grouting head for borehole backfill. However, with any such computation, judgment is required in applying the results.

Additional references on hydraulic fracturing are included in Appendix A.

4.2.2 Artesian Conditions/Blowout

In situations where the presence of higher fluid pressures in the subsurface materials is suspected, either at the ground surface or at depth, it may be necessary to install a surface casing to control artesian pressures if the pressures are anticipated to be significant and/or derived directly from reservoir head. Surface casing of slightly larger diameter than the augers or drill string to be used is grouted in place and allowed to set prior to advancing the borehole to depth. If flow from the borehole occurs, the surface casing provides a means of controlling it by blocking off the space between the augers/drill rods and well casing. When the static water level is very near the ground surface or artesian conditions prevail, one should consider elevating the drilling rig on a temporary drill berm to raise the drill hole collar elevation. In extreme cases, the berm should consist of filter zones. Specific details such as height of the drill pad and amount of surface casing must be developed on a case-by-case basis dependent upon specific conditions present at the site. Even if artesian pressures are not expected at a given site, potential risk requires contingency plans be in place in case these conditions arise.

If holes must be advanced at the toe of a dam that has a critical gradient condition, planning and precautions should be developed. In all cases, issues of this nature should be identified and addressed by the exploration team prior to commencement of work. In these areas, it is necessary to maintain a positive hydrostatic pressure on the drill hole to prevent a “blowout.” In instances when higher pressures are not anticipated, the addition of commercial densifiers to the drill mud may successfully address the concern.

4.2.3 Erosion

The introduction of drilling fluids into cracks, either existing or formed by hydraulic fracture, can potentially cause erosion along the crack walls. This will enlarge the crack and could lead to an increased potential for internal erosion. Existing subsurface cracks are common in many dams and are often the result of differential settlement. The locations most at risk for existing cracks are typically the same areas that have low confining stress and have the highest risk for hydraulic fracture to occur.

4.2.4 Contamination of Filter/Drainage Features

In addition to hydraulic fracturing, the use of drilling fluids can pose a contamination risk for internal drainage features if the drill fluid or sealing grout migrates into and clogs the drain or filter materials. Avoid drilling near drains or seepage blankets that may become contaminated by fluids. If drain penetration is justified, special provisions must be taken to prevent contamination. Special provisions may also be required for protecting the drainage features while backfilling the hole (such as placement of filter material through the zone of the drain or filter and installing lower and upper seals).

4.2.5 Heave and Sample Disturbance

Drilling programs that include performing in-situ tests or undisturbed sampling may require the use of drilling fluid to offset the confining stress relieved by the drilling of the hole. There have been cases where the failure to prevent stress relief or heave of granular soils below the water table have led to invalid in-situ test results and subsequently invalid interpretation of the subsurface conditions. This has occurred for both tests performed in drill holes and test performed in casings installed by methods that did not control heave or disturbance.

BOR (1999) contains methods to deal with heaving sands while drilling and performing Standard Penetration Tests. If high quality undisturbed samples of fine grained soils are required for shear strength testing, then drilling mud may be required to prevent the soil from failing in undrained triaxial extension. See Ladd and DeGroot (2004) for a discussion on clay sample disturbance due to drilling.

Prior to embarking on any drilling activity, the exploration team should consider, at a minimum, these potential drilling hazards and develop the drilling plans to avoid or mitigate these hazards. If the hazards cannot be avoided, then the risks must be evaluated and mitigated in the drilling plan.

4.3 Drilling Methods

There are numerous drilling methods available to perform geotechnical investigations. The American Society of Testing and Materials (ASTM D6286) provides a comprehensive guide for drilling methods and groups individual practices for eight drilling methods (ASTM, 2006). Other good texts on drilling include The Bureau of Reclamations Earth Manual, Part I, Third Edition, Chapter 2 (BOR, 1998), the Australian Drilling Manual (ADI, 1992), and the National Drill Association Drilling Manual (NDI, 1990). Details of these drilling methods are not discussed in-depth in this guide.

Nine major drilling methods are briefly discussed below. Table 1 provides a quick reference to each method. All drilling methods that use air or fluid media have the potential to create hydraulically-induced fractures. Air drilling methods use high pressures and are well known for causing fracturing with air traveling long distances. Therefore, drilling with air as the drilling medium should never be considered when there is potential to encounter the core of an embankment dam.

The drilling methods listed below are in order of preference for use in drilling and sampling in embankment dams. Only the first three are considered preferred methods.

All drilling programs in dams should be designed to minimize the need for any drilling fluid such as air, gas, water, mud, polymers, slurries or any other drilling fluid that could pressurize the borehole soils. If the drilling objective can be performed using dry methods such as augers or sonic drilling they should be employed in lieu of methods that require fluids. If drilling fluids must be used due to the drilling objective or the subsurface conditions, the drilling plan must contain an analysis of the potential to cause damage and a plan that covers the measures that will be used to minimize the risk (see Section 4.8 for additional information). The use of pressurized air or foam should only be considered when drilling in materials that will not transmit pressures to the soil core or other critical features or when the air pressure is reliably isolated from the borehole soils. Drilling in an open graded rockfill shell may be an example of when using air may be appropriate. All drilling programs that propose the use of stabilizing or circulating fluids or other media will require an additional level of review.

1. **Hollow-Stem Auger** – Hollow-stem auguring (HSA) is a preferred method of drilling in the core and most other areas of an embankment dam without restriction. Blowout prevention measures, such as sealable surface casing, should be used prior to advancing augers in areas where there is potential to encounter artesian conditions. If no fluid is added to the auger column, it does not pressurize the embankment and no potential for hydraulic fracturing exists. However, for SPT testing, it may be required to add some fluid to stabilize loose sands and gravels. In instances when groundwater is encountered or fluids are added to the process, the auger string should be raised and lowered slowly to avoid pressurization, negatively and positively, respectively, of any open hole. Using a hollow-stem auger permits sampling in the embankment and allows sampling/testing of the foundation through the auger's hollow-stem which acts as casing. Continuous sampling is described in ASTM D6151 (ASTM, 2008). Small diameter cores of 3 to 4 inches in diameter can be taken in 5-foot-lengths using the split inner sampling barrel. High quality, undisturbed samples can be taken with larger diameter HSA (6-inch ID and larger) in acrylic liners that provide samples suitable for laboratory testing.

2. **Sonic Drilling** ASTM D6914 (ASTM, 2010) – Sonic (vibratory) drilling is a preferred method of drilling in the core and other areas of embankment dams. This method uses a double casing system and vibrating drill head to set up standing waves or resonance to the drill steel to advance the boring. This method of drilling is favored due to its lack of drill fluid and rapid speed of drilling. The drilling process first advances a core barrel. The core barrel is removed, and the sample is extruded while the outer casing is then advanced to the end of the sampling run. There are no cuttings generated, and there is some compaction of soil around the annulus of the drill. Crowd-in and crowd-out bits are used depending on the formation. Some water (static water, not under pressure) is required for dry cohesive formations to lubricate the drill stem. The cores, typically 4 to 5 inches in diameter, are useful for lithology determination and samples may be adequate for standard engineering properties laboratory analysis, but does not meet criteria for many laboratory tests requiring undisturbed samples (Dustman, et al, 1992). Since there

is uncertainty as to the extent of disturbance to the adjacent foundation material from the vibratory drilling process, sonic drilling should not be used if SPT, undisturbed sampling, and certain in-situ testing are required.

3. **Cable Tool or Churn Drilling** ASTM D5783 (ASTM 2000a) – Cable tool or churn drilling, with minor restriction, is a preferred method of drilling in embankment dams. This is an older method of drilling that is infrequently used. Drill action is by up and down movement of the drill string and jars (bit). The drill string is regularly pulled and a bucket-grab tool is inserted to remove/sample the cuttings. Water is often added to the hole to mix the cuttings into slurry. SPTs can be completed below the bottom of the casing. This method of drilling is rated high in desirability because it does not use a full column of drilling fluid and, therefore, has low potential for fracturing. Drilling speed is fairly comparable to HSA drilling. One variation of this "chop and drive" technique employs continuous circulation of water to bring cuttings to the surface and should not be used in the core of an embankment dam.

4. **Dual Rotation Drilling** ASTM D5781 (ASTM 2000b) – Dual rotation drilling is not a preferred method for drilling in embankment dams, and its use in embankment core material must be approved by FERC prior to use. The dual rotary drilling method advances both the casing and the drill string/bit separately. The upper and lower rotary drives feed independently by use of separate hydraulic cylinders. Distances between the bit tip and casing shoe are adjustable. With the bit advancing ahead of the shoe, drilling becomes more aggressive. These bit to shoe relationships allow the pressurized drilling medium to come in contact with the unprotected hole wall, and potential for hydraulic fracturing increases. When drilling in embankment core material, the bit should not be advanced ahead of the shoe. In those instances when the bit advances ahead of the shoe they should be recorded on the daily drill report and, subsequently, geologic log for future reference. In all cases, use of clear water or air as a drilling medium is not allowed in embankment core material. Fluid pump pressure must remain low and pressures carefully monitored when this method is used in or near the embankment core. When starting circulation, pumping should be increased gradually to reduce the occurrence and increase the ability to observe evidence of hydraulic fracturing. A pressure relief valve set to the maximum allowable pressure is required.

5. **Fluid Rotary Drilling** ASTM D5783 (ASTM 2000c) – Fluid rotary drilling is not a preferred method for drilling in embankment dams, and its use in embankment core material must be approved by FERC prior to use. This drilling method uses a rotary cutting bit with circulation of water or drilling mud (bentonite or polymer). Cuttings are returned to the surface and dropped in settling tanks. Ideal bentonite drill mud mixtures do not exceed 72 lb/ft and have 60- to 70-second marsh funnel viscosities; however, higher viscosities may be necessary where artesian conditions are encountered. Casing is often advanced with the boring. In all cases, use of clear water as a drilling medium should not be allowed in embankment core material. Fluid pressure must be very low and carefully monitored when this method is used in or near the embankment core. When starting circulation, pumping should be increased gradually to reduce the occurrence and increase the ability to maximum allowable pressure is recommended.

Fluid rotary is the preferred method for SPT testing for liquefaction (see ASTM D6066), where it is recommended to keep the hole full of fluid during the test to stabilize sands. Since drilling fluid is being used, this method has a high potential for hydraulic fracturing. Raising and lowering drill bits, sampling tools, and drill rods should be done slowly so as not to induce negative fluid pressures or increase fluid pressures.

6. **Becker Drilling/Penetration Testing** – Becker drilling is not a preferred method for drilling in embankment dams, and its use in embankment core material must be approved by FERC prior to use. Becker drilling may be one of two methods. The closed bit system advances a closed bit by means of hammering with a double acting diesel hammer. This method frequently is used in coarser grained material where SPT data likely would be invalid. The open bit method advances an open bit by using of the double acting diesel hammer. In this method, disturbed samples may be collected. High-pressure air is forced down the outer annulus of the dual casing system and returned up the inner casing. The returning air carries soil cutting up to the ground surface. Open bit Becker drilling is prohibited when drilling in or near the core section of an embankment dam.

7. **Wire Line and Casing Advancer** ASTM D5876 (ASTM 2000d) – Wire line and casing advancer systems are not preferred methods for drilling in embankment dams, and their use in embankment core material must be approved by FERC prior to use. These drilling systems use fluid rotary action to remove the cuttings with the exception that the fluid flows up the annulus between the rods and the borehole wall. In all cases, use of clear water as a drilling medium should not be allowed in embankment core material. Fluid pressure must be very low and carefully monitored when this method is used in or near the embankment core. When starting or restarting circulation, pumping should be increased gradually to reduce the occurrence and increase the ability to observe evidence of hydraulic fracturing. A pressure relief valve set to the maximum allowable pressure is recommended. Since fluid is circulated up the annulus between the soil and drill rod, there is increased chance of blocking circulation and possible fracturing. The drill rods act as casing and are equipped with a cutting bit. Either a core barrel or cleanout bit lock into the lead section of the drill rods and is latched by wire line. This results in rapid drilling and reduced rod trip time during coring operations. Some wire line drilling systems have soil core barrels, but their success is limited. Wire line diamond drilling is the primary method of rock core drilling (see ASTM D2113 on Diamond Drilling (ASTM 1999)). Typically, augers, casing, or other methods are used to set a protective casing through the embankment and foundation soils and then fluid rotary drilling is used to core and water test the foundation rock.

8. **Drill Through/Drive Casing Advancer** ASTM D5872 (ASTM 2000e) - Drill through/ drive casing advancers are not preferred methods for drilling in embankment dams and their use in embankment core material should not be considered. The drills have a casing driver (hammer) and a rotary rock bit or down hole hammer that may be rotated through the casing hammer. Down-the-Hole hammers (DTH) and air are used in coarse boulders deposits and hard rock while rock bits and fluids might be used in dirtier gravel cobble soils. One version of DTH, known as ODEX, has a swing out bit which over-reams the

hole for the casing. Air flow to circulate cuttings has to be rather high, but can be reduced by introduction of foam. To minimize fracturing when drilling with air, the drill bit should be held just inside the casing so a protective seal remains at the bottom of the casing. This practice is not possible when using ODEX, which requires the bit to advance before the casing.

9. **Air Rotary** ASTM D5782 (ASTM 2000f) - Air rotary is not a preferred method for drilling in embankment dams and its use should not be considered in embankment core material. This class of drilling is very similar to drill through drive casing systems except the hole may be left open (uncased) exposing the complete borehole wall to air flow. Without the protection casing provides, the possibility exists for circulation blockage, possible fracturing, and degradation/opening/erosion of any weak seam exposed along the sides of the borehole. One example of this type is the air track drill.

Table 1 – Drilling in Embankment Dams – Drilling Methods

<table>
<tr><th></th><th>Drilling Methods</th><th>Restriction</th><th>Recommendations</th></tr>
<tr><td rowspan="3">Preferred Drilling Methods</td><td>Auger</td><td>None</td><td>Raise and lower auger string slowly when fluid in hole</td></tr>
<tr><td>Sonic/Vibratory</td><td>None</td><td>Core not suitable for higher level laboratory testing</td></tr>
<tr><td>Cable Tool/Churn</td><td>Chop and drive variation not allowed</td><td>Samples are of cuttings and are highly disturbed</td></tr>
<tr><td rowspan="4">Restricted Drilling Methods</td><td>Dual Rotation</td><td rowspan="4">Approval of drilling method required

Clear water as drilling media not allowed

Fluid pressure must be very low

Bit must not be advanced beyond shoe

Open bit methods are not allowed</td><td rowspan="4">Monitor fluid pressure closely

Use pressure relief valves to cap fluid pressure

Increase pump pressure gradually

Monitor fluid viscosity closely</td></tr>
<tr><td>Fluid Rotary</td></tr>
<tr><td>Becker</td></tr>
<tr><td>Wireline/Casing Advancers</td></tr>
<tr><td rowspan="2">Prohibited Drilling Methods</td><td>Drill Through/ Drive Casing Advancers</td><td rowspan="2">Not allowed in or near the core of embankment dams. Approval of drilling method required for other areas. Will only be considered in extraordinary circumstances</td><td rowspan="2">Not allowed in or near the core of embankment dams. Approval of drilling method required for other areas. Will only be considered in extraordinary circumstances</td></tr>
<tr><td>Air Rotary</td></tr>
</table>

There are some general procedures that should be followed when using drilling fluids to limit the risk of damage:

- Tools should be sized and designed to minimize the likelihood of the return flow clogging.

- Methods that require the cuttings to flow through a small annulus between the tools or casing and the borehole wall should not be used.

- Fluid discharges from the bit should always be upward, not downward into the formation material or lateral into the sidewalls that could lead to excessive disturbance or erosion.

- Lower and raise drill tools slowly to avoid pressure changes in the drill hole; this is especially important when using tools with restricted annulus space below the groundwater as the pressure changes are more severe and can lead to suction and surging problems.

- Drilling feed rate must be slow enough to avoid crowding the bit and, thus, minimize the chance of inducing fracturing. The bit must be of a design such that pressure buildup is minimized.

- Drilling media properties, pressure, and return should be continuously monitored. A floating needle pressure valve is required to record maximum pressure spikes that can occur instantaneously and are often unnoticed.

- When media circulation is required, a pressure controlled release ("pop off") valve should be on the pump.

- In some conditions, casing can be advanced ahead of the drilling bit to reduce the risk of hydraulic fracturing by confining the drilling fluids within the casing.

- Great care should be taken during washing of the hole.

- Casing should be pushed or driven and not jetted. Except in special circumstances, casing must precede the drilling.

- When core drilling rock, the embankment or foundation soil above top of rock must be protected and isolated from the circulating drilling fluid. Fractures in the bedrock must be considered as potential flow paths in contact with the overlying soil.

- A pause or suspension in drilling operations (breaks, meals, overnight/weekend, etc.) should not leave the borehole in a critical state that could result in damage to the embankment.

4.4 In-Situ Testing/Sampling

The actual process of advancing the boring is not the only potential hazard that can lead to hydraulic fracturing and other adverse impacts of the drilling, sampling, disturbance, and performance of the structure. Raising and lowering drill rods, casing, or other drill steel too quickly can induce significant positive or negative fluid pressures.

In-situ testing that includes applying hydraulic pressures through static head (falling head or constant head permeability tests) or pressure induced head (packer pressure tests, etc.) can result in excessive hydraulic pressures that could lead to hydraulic fracturing. In-situ testing and sampling methods and procedures must be aware of the potential to create these conditions. The Bureau of Reclamations' *Engineering Geology Field Manual* is an excellent reference to assist in determining applied and total hydraulic fluid pressures from in situ tests (BOR, 1998).

4.5 Hole Completion

All boreholes and other penetrations (including direct push sampling, Cone Penetration Test soundings, Standard Penetration Testing, Becker Penetration Testing, etc.) in and around embankment dams must be sealed after completion. Completing a borehole by backfilling with drill cuttings is not acceptable. There are a variety of acceptable methods to complete a borehole.

All boreholes and similar penetrations in the impervious portions of an embankment dam and their foundations must be backfilled by tremie-placed cement-bentonite grout or bentonite pellets/chips, except when an alternative backfill method compatible with instrument installation is approved. The drilling plan must address the possibility of confined and separate groundwater aquifers and demonstrate safe completion which avoids cross-contamination and leakage. The grout must be designed to obtain strength equal to or greater than the soil or rock. Note that some instrumentation installations may require additional considerations for the grout strength. Gravity grouting techniques should be used for backfilling boreholes.

For borings that penetrate zones with low confining stress it is possible to induce hydraulic fracturing even from gravity pressure alone. When grouting borings in these locations or if significant grout losses are observed, the grout backfilling should be done in stages allowing the grout to set between stages.

For pervious portions of the dam (drainage features, filters, etc.), the borehole must be backfilled by tremie placement of granular materials that are sized to provide drainage without being susceptible to migration through the pervious embankment or foundation materials or segregation during placement.

Lutenegger, et.al. (1995) is a good source for borehole backfill guidelines.

Special procedures and materials may be required for installation of instrumentation in boreholes.

Borehole completion is often not well documented. Recommended inclusions in borehole completion documentation include intervals of various backfilling materials, calculated volume of material necessary to fill each interval, and actual volume of material required to fill each interval. Detailed records of borehole completion are important and, as in the case of backfill material volumes significantly higher or lower than calculated, may be indicative of conditions significantly different than anticipated.

Below are some general guidelines that can be considered in borehole completion.

- **High Solids Bentonite Grout** - Tremie grouting with high solids bentonite is an acceptable method of completing boreholes in embankment dams. Mixes which yield 20 to 30 percent solids should be used. Stage up tremie grouting methods should be used in the embankment with the casing (i.e. hollow-stem augers, rods, etc.) pulled incrementally to ensure hole wall stability. The bentonite slurry should always be injected through a tremie pipe to ensure the best possible placement and most thorough borehole completion.

- **Neat Cement Grout** - Neat cement grout is another acceptable method of completing boreholes in embankment dams. The best results are achieved when the mix consists of 5 to 7 gallons of water to one sack, 94 lbs of Type I or Type II Portland cement (using higher water contents may result in excessive shrinkage, cracking, and bleed water). Commonly, the addition of up to 3 percent powdered bentonite by dry mass of cement is used for pumping ease and to reduce shrinkage and cracking after curing – although a myriad of other compounds are also available. Additives such as calcium chloride or carboxylic acid can be used to control set times, but shrinkage factor must be considered. Using type K cement or adding up to 1 percent gypsum or aluminum powder by weight will give the cement expansive properties, which may be advantageous in embankment dams where internal seepage is an issue. As with the bentonite grout, stage up tremie grouting methods should be used in the embankment core with the casing pulled incrementally to ensure borehole wall stability. The grout should always be injected through a tremie pipe to ensure the best possible placement and most thorough borehole completion.

- **Bentonite Pellets or Chips** - The use of bentonite pellets or chips may be an acceptable method of completing boreholes in embankment dams. However, there are some conditions under which bentonite pellets or chips should not be considered and only tremie grouting is acceptable. Bentonite pellets or chips, including those treated to retard or delay flocculation, should not be used in cases where there is a chance the depth of water in the hole could slow the bentonite fall and allow flocculation prior to the bentonite reaching hole bottom. Additionally, even in a dry hole, there must be adequate annular space available to allow the bentonite to fall to the borehole bottom without bridging. It is advisable to always place both solid bentonite and grout through a tremie pipe.

- **Instrument Installations** - Instrumentation installations require special completions. For piezometers, sand packs are placed in the influence zone and a bentonite seal is placed above the sand pack to prevent any contamination of the sand pack from sealing materials placed above it. The bentonite seal is typically bentonite pellets. A common error in placing the seal is not allowing bentonite time to hydrate. Pellets should be allowed a minimum of 1 to 2 hours to hydrate prior to placing additional backfill material above the seal. Alternatively, piezometers can be installed in fully grouted holes. While it is possible to place two piezometers in a typical 4-inch inside diameter hollow stem auger or casing, only one piezometer is recommended, and no more than two instruments should be allowed in a single boring. Difficulty in providing a good seal between multiple riser pipes may result in communication between influence zones. Other instrument installations (slope inclinometer casing, geophysical casing, etc.) will require additional considerations.

4.6 Drilling Personnel

Because of the potential to do harm, drilling in a dam should only be performed by experienced and qualified personnel. This includes the lead drill rig operator and the engineer or geologist who is the on-site representative responsible for the drilling program and the safety of the dam. Schedule, budget, and other issues should be considered secondary to the safety and integrity of the structure and those potentially impacted by its compromise.

Drill rig operators must have a minimum of 5 years of experience drilling with the equipment and procedures described in the drilling program. When the drilling plan includes drilling in or in the vicinity of dam or appurtenant structure foundations or abutments or within an embankment dam, the drill rig operators must have demonstrated embankment dam drilling experience clearly indicated in their resume.

All drilling activities must be conducted in the presence of a qualified geotechnical engineer or engineering geologist who will be responsible for maintaining the integrity of the structure and the inspection of the drilling operation. Qualified is by combination of education, training, and experience as indicated in Table 2.

Table 2 – Minimum Qualifications of Responsible On-site Personnel

Factor	**Low Hazard Dams**	**Significant and High Hazard Dams**
Education	Minimum B.S. in Civil Engineering or Geology (or licensed as a professional engineer, professional geologist, or certified engineering geologist)	
Training	Independent study or formal training in the identification and mitigation of drilling hazards in embankment dams	
Experience	Minimum of two years of general drilling experience	Minimum of four years of embankment dam drilling experience

While there are many inspectors with significant years of experience with drill procedures, classifying soils and rock, and in-situ testing methods, they may only have limited knowledge and experience with dams and may be unaware of potential damage to critical dam features caused by certain drilling procedures. Therefore it is critical that a combination of education, training, and experience be demonstrated and clearly shown on the resume of the geotechnical engineer or geologist inspecting the work.

The project manager directing the drilling program must also be an experienced geotechnical engineer that is a licensed professional engineer or a licensed professional geologist or certified engineering geologist with at least ten years' experience in dams-related work.

Both the drill rig operator and the on-site geotechnical engineer/engineering geologist must also be familiar with these guidelines. It is essential that drill rig operators and the geotechnical engineer/engineering geologist be well trained and aware of the causes of and the problems resulting from hydraulic fracturing and artesian conditions and have the equipment, materials, and experience to correct and remediate damage to the embankment and foundation.

4.7 Other Considerations

Emergency Communications - No dam should be drilled or investigated without a thorough review of the Emergency Action Plan (EAP). FERC-regulated dams have EAPs in place. The EAP lists the key individuals who should be contacted and informed of proposed activities. There are documented case histories where drilling has caused incidents with dams and knowledge of the EAP and good communications were key contributors to safely solving the problems.

Monitoring - During drilling operations, the dam embankment should be continuously inspected and monitored using appropriate procedures and instrumentation at the dam site. The proposed monitoring should be used to evaluate any impacts from of the drilling activity and assist in detecting any unanticipated changes. The type of monitoring (piezometer, inclinometers, etc.), frequency of readings, and purpose for monitoring should be carefully considered. If appropriate, threshold limits could be determined for specific drilling scenarios. It may be necessary to perform daily inspections of the dam for a period of time after the drilling operations have concluded.

Reporting - All incidents of damage or potential damage related to drilling and associated activities for dams must be reported. If a sudden loss of drill media occurs during any embankment drilling within the core, drilling must be stopped immediately. Action should be taken to stop the loss of drill fluid. The reason for loss should be determined and if hydraulic fracturing may have been the reason for the fluid loss, FERC should be notified immediately.

Construction/Remediation Drilling Activities - Drilling activities performed during construction or remediation phases are often overlooked as opposed to drilling that occurs under the traditional exploration phases. There are numerous examples of dams which required remediation after reservoir filling and the embankment or foundation was damaged. Many of these dams required remedial grouting immediately after construction, and the grouting contractor used air drilling, rapidly resulting in fracturing of blankets and foundations. Jet grouting contractors drill holes with very high air/fluid pressures at rapid rates. Contractors want to drill fast, but drilling fast may cause blockage and loss of the circulating fluid and hydraulic fracturing. It is imperative that, for remediation construction projects, and instrumentation installation contracts, project geologists and engineers identify drilling methods and confirm they are appropriately screened to avoid damage to the dam or foundation. If there is concern, a team should be formed to review the drilling methods and ensure the contract documents have appropriate provisions to avoid damage to the dam and foundation.

Exemptions - Drilling required for immediate emergency measures where delays required to develop the drilling plans and to obtain the necessary reviews and acceptances would result in unacceptable risk of damage or failure, may be exempted from the requirements to prepare a drilling plan, as approved by the Regional Engineer. Emergency drilling should be appropriately expedited but should follow the general guidelines presented in this guideline.

4.8 Evaluation of Potential Risks

The licensee must thoroughly evaluate the risks associated with the proposed drilling and indicate how they intend to mitigate them. Among other topics, the potential risks of causing hydraulic fracturing of the embankment, as well as the potential risks of causing seepage, instability, or other potential dam safety issues as a result of the proposed drilling program must be evaluated and addressed. The risk evaluation must include an assessment of the potential impact of the drilling operations and the location of the boreholes in relation to areas of the dam that may be more susceptible to hydraulic fracturing, as discussed in Section 4.2.1.

Aside from comparing the planned drilling locations with the areas of the embankment and soil types that are more susceptible to hydraulic fracturing, the proposed drilling procedures must also be evaluated with respect to their likelihood of causing hydraulic fracturing or other dam safety issues. This includes the instrumentation installation procedures, borehole completion/abandonment procedures, and emergency procedures if a potential dam safety issue is identified during the drilling. Special attention should be given to highlighting the specific procedures and contingency plans that will be utilized to protect the dam from potential hydraulic fracturing and other potential risks.

5.0 DRILLING PROGRAM PLAN (DPP)

An approved Drilling Program Plan (DPP) is required for any exploration drilling, instrument installation, or remediation drilling (including grouting) work to occur on an embankment dam, in proximity of the dam in which the drilling methods could pose a risk to the dam, or the dam's foundation and abutments. DPPs shall be prepared and reviewed by experienced geotechnical engineers and/or engineering geologists familiar with subsurface exploration techniques and methods. It is paramount that all existing subsurface information is thoroughly evaluated and understood by the exploration team prior to developing a plan for additional drilling. In order to understand and communicate subsurface conditions and estimate drilling risk, the existing subsurface information must be assimilated into essential plan and section drawings showing proposed drill holes and depths, target sample areas and proposed instrumentation. The DPP must also comply with good environmental practices and comply with site environmental provisions/restrictions, which may need coordination with DHAC and outside agencies.

The DPP must be reviewed and accepted by the FERC Regional Engineer prior to beginning the drilling program. Depending on the particular dam and scope of the project involved, the review process may also require additional coordination with FERC headquarters staff in Washington, D.C. and/or DHAC. As stated in our Annual Letter, this plan must be submitted for our review a minimum of **30 days** prior to beginning the drilling work. However, licensees are encouraged to inform the FERC project engineer of the planned drilling program and begin discussions with him or her regarding the proposed drilling well in advance of this deadline.

In addition, the licensee is encouraged to set up either a face-to-face meeting or conference call with the Regional Engineer and headquarters staff, as appropriate, once the specifics of the proposed drilling program have been developed. Ideally, this meeting should take place as soon as possible but no later than *a minimum of two weeks* prior to submission of the DPP. The purpose of this meeting and early coordination with the FERC project engineer is to ensure that both the licensee and FERC share a common understanding of the requirements of the project and the DPP, and there are no delays associated with FERC's review or potential issues with the plan.

FERC's primary concern in evaluating the licensee's DPP will be ensuring that the planned drilling program will "do no harm" to the existing dam. A thorough, well-organized, and well-developed DPP, including the various items highlighted in these guidelines, will assist FERC in its review by demonstrating that the licensee fully understands the risks associated with the drilling program, and is taking the appropriate measures to mitigate them.

In general, the DPP must include the following information, as a minimum:

1. Name and description of project.

2. Purpose of site disturbing activity.

3. Description of the proposed site exploration activity (drilling, test pitting, etc.). Include plan view showing location of activity (ies), proposed drill hole depths, sampling intervals, insitu testing, and instrument installations.

4. Describe and show anticipated site conditions. Show location of known subsurface conditions and features. Describe subsurface units. Describe understanding of ground water conditions and phreatic surface, including the potential to encounter artesian conditions. Use cross sections and profiles to graphically illustrate.

5. Describe proposed equipment, methods, and processes. For example, for any activity that introduces a fluid in or near the water retaining feature or its foundation, detail how fluid pressures will be measured and monitored. For example, for falling head permeability tests, show how the introduction of a column of water will not cause excess water pressures in the embankment that could lead to hydraulic fracturing. Likewise, for grouting of boreholes, describe how if staged grouting will be required and how the maximum height of grout column will be determined to prevent hydraulic fracturing.

6. Identify project personnel and qualifications/experience, including resumes.

7. Risk identification and mitigation plan. Identify and describe potential risks imposed by site disturbing activities. Identify and describe risk mitigation plan. For example, for any activity at the toe of a water retaining feature, describe the risk mitigation plan should unexpected artesian conditions be encountered.

8. Identify communication plan with names and phone numbers. Include a list of emergency equipment and supplies to have on site (phone/radio, filter materials, grout materials, light plant, etc.).

9. Provide an overall schedule and duration of drilling activities.

Specific requirements for the DPP are included in Appendix B.

6.0 REPORTING REQUIREMENTS

The DPP should provide details on the documentation, logging, and submission of drilling data. The field inspector's boring log should be submitted to FERC within 24 hours after completion of backfilling the boring. When feasible, draft field boring logs should be submitted daily, along with daily work logs. Since there is always a possibility that some changes will need to be made in the field due to the specific subsurface conditions encountered, the DPP should describe how changes and deviations from the approved DPP will be communicated and coordinated with FERC. Also, any significant differences from expected conditions which could be an indication of a potentially serious dam safety issue must be reported immediately to the FERC Regional Engineer.

7.0 REFERENCES

ADI, 1992, "Australian Drilling Manual", Third Edition, Australian Drilling Industry Training Committee, NSW 2113, Australia.

ASTM, 1999, "Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigations", D2113, West Conshohocken, Pennsylvania.

ASTM, 2000a, "Standard Guide for Use of Cable-Tool Drilling and Sampling Methods for Geoenvironmental Exploration and Installation of Subsurface Water- Quality Monitoring Devices", D5875-95, West Conshohocken, Pennsylvania.

ASTM, 2000b, "Standard Guide for Use of Dual-Wall Reverse-Circulation Drilling for Geoenvironmental Exploration and the Installation of Subsurface Water- Quality Monitoring Devices", D5781-95, West Conshohocken, Pennsylvania.

ASTM, 2000c, "Standard Guide for Use of Direct Rotary Drilling with Water-Based Drilling Fluid for Geoenvironmental Exploration and the Installation of Subsurface Water- Quality Monitoring Devices", D5783-95, West Conshohocken, Pennsylvania.

ASTM, 2000d, “Standard Guide for Use of Direct Rotary Wireline Casing Advancement Drilling Methods for Geoenvironmental Exploration and Installation of Subsurface Water-Quality Monitoring Devices”, D5876-95, West Conshohocken, Pennsylvania.

ASTM, 2000e, “Standard Guide for Use of Casing Advancement Drilling Methods for Geoenvironmental Exploration and Installation of Subsurface Water- Quality Monitoring Devices”, D5872-95, West Conshohocken, Pennsylvania.

ASTM, 2000f, “Standard Guide for Use of Direct Air-Rotary Drilling for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices”, D5782-95, West Conshohocken, Pennsylvania.

ASTM, 2006, “Standard Guide for Selection of Drilling Methods for Environmental Site Characterization”, D6286-98, West Conshohocken, Pennsylvania.

ASTM, 2008, “Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling”, D6151-08, West Conshohocken, Pennsylvania.

ASTM, 2010, “Standard Practice for Sonic Drilling for Site Characterization and the Installation of Subsurface Monitoring Devices”, D6914-04, West Conshohocken, Pennsylvania.

BOR, 2010, “Guidelines for Drilling and Sampling in Embankment Dams”, Bureau of Reclamation, Denver, Colorado.

BOR, 1990, “Earth Manual, Part II”, Third Edition, Bureau of Reclamation, Denver, Colorado.

BOR, 1998, “Earth Manual, Part I”, Third Edition, Bureau of Reclamation, Denver, Colorado.

BOR, 1999, “Standard Penetration Test: Drillers/Operators Guide, Report Number DSO- 98-17,” J. Farrar, Dam Safety Office, Bureau of Reclamation, Denver, Colorado.

Dustman, J.R., R. Davis, and T. Oouthoudt, 1992, “Soil, Bedrock, and Groundwater Sampling Using Rotasonic Drilling Techniques,”, Proccedings of the Sixth National Outdoor Action Conference on Aquifer Restoration, Groundwater Monitoring, Geophysical Methods, National Ground Water Association, Las Vegas, Nevada.

FEMA, 2000, “Research Needs Workshop: Seepage through Embankment Dams”, Federal Emergency Management Agency, Washington, DC.

France, J.W., 2002, “Two Embankment Dam Seepage Incidents,” Technical Workshop No. 9, Responding to Dam Safety Emergencies, National Dam Safety Program, National Emergency Training Center, Emmitsburg, Maryland.

Ladd, C. C., and D.J DeGroot, 2004 (revised). “Recommended Practice for Soft Ground Site Characterization,” Arthur Casagrande lecture, Proc. 12th Pan-American Conf. on Soil Mech. and Geot. Engineering.

Lutenegger, A.J., D.J. Degroot, C. Mizra and M. Bozozuk, 1995, “Recommended Guidelines for Sealing Geotechnical Exploratory Holes, Report 378,” National Cooperative Highway Research Program, Transportation Research Board, National Academy Press, Washington, DC.

NDA, 1990, “Drillers Manual,” National Drilling Association, accessed at www.ndu4u.com, Brunswick, Ohio.

Schaefer, J.A., D.B. Paul, and D.D. Boyer, 2011, “Safe Grouting Pressures for Dam Remediation”, USSD Annual Meeting, Nashville, Tennessee.

Sherard, J.L., 1973, “Embankment Dam Cracking,” Embankment Dam Engineering, S. Poulos and R. Hirschfeld, Eds., John Wiley and Sons, New York, N.Y., pp. 272-353.

Sherard, J.L., 1986, “Hydraulic Fracturing in Embankment Dams”, ASCE Journal of Geotechnical Engineering, Volume 112, No. 10, pp. 905-927.

USACE, 2014, “Drilling in Earth Embankments and Levees”, ER 1110-1-1807, U.S. Army Corps of Engineers, Washington, DC.

APPENDIX A

ADDITIONAL HYDRAULIC FRACTURING REFERENCES

Albritton, J., Jackson, L., and Bangert, R., “Foundation Grouting Practices at Corps of Engineers Dams”, Technical Report GL-84-13, US Army Corps of Engineers, October 1984.

Alfaro, M.C., and Wong, C.K., “Laboratory studies of fracturing of low-permeability soils”, Canadian Geotechnical Journal, 38, 303-315, 2001.

Andersen, K.H., Rawlings, C.G. Lunne, T.A., and By, T.H., “Estimation of hydraulic fracture pressure in clay”, Canadian Geotechnical Journal, 31, 817-828, 1994.

Bjerrum, L., Nash, J. K. T. L., Kennard, R.M. & Gibson, R.E., “Hydraulic fracturing in field permeablity testing”, Geotechnique, 22, 319-32, 1972.

Bozozuk, M., “Minor principal stress measurements in marine clay with hydraulic fracture tests”, Proceedings, Engineering Foundation Conference on Subsurface Exploration for Underground Excavation and Heavy Construction, Henniker, N.H., August 1974.

Bureau of Reclamation, “Engineering Geology Field Manual,” Vol I and II, Second Edition, Denver, CO, 1998.

Calcagno, Frank, Jr., USBR, “Hydraulic Fracture Study of the Tiber Spillway Cofferdam”, AEG Newsletter 26/4 October 1983.

Casagrande, A. and Covaarrubias, S.W., “Cracking of earth and rockfill dams, tension zones in embankments caused by conduits and cutoff walls”, Contract Report S-70-7, U.S. Army Engineer Waterways Experiment Station, July 1970.

Chang, H., “Hydraulic fracturing in particulate materials”, Doctoral Thesis, Georgia Institute of Technology, November 2004.

Chen, Yu-jiong, and Zhang, Shu-lu, “Test embankment of fracture grouting”, Journal of Geotechnical Engineering, Vol. 115, No. 11, November 1989.

Clough, R.W. and Woodward, R.J. III, “Analysis of embankment stresses and deformations”, Journal of Soil Mechanics and Foundations Division, Proceedings of ASCE, Vol. 93, No. SM4, July 1967.

Elwood, D., and Moore, I., “Hydraulic fracture experiments in sand and gravel and approximations for maximum allowable mud pressure”, North American Society for Trenchless Technology, No Dig Show, Mar-Apr 2009.

Hamouche, K.K., Leroueil, S., Roy, M., and Lutenegger, A.J., “In situ evaluation of K0 in eastern Canada clays”, Canadian Geotechnical Journal, vol 32, pgs 677-688, 1995.

Independent panel to review cause of Teton Dam failure, 1976, Failure of Teton Dam: Report to the U.S. Department of Interior and State of Idaho.

Kulhawy, F.H., and Duncan, J.M., “Stresses and movements in Oroville dam”, Journal of Soil Mechanics and Foundations Division, Proceedings of ASCE, Vol. 98, No. SM7, July 1972.

Lo, K.Y. and Kaniaru, K., “Hydraulic fracture in earth and rock-fill dams”, Canadian Geotechnical Journal, Vol 27, 496-506, 1990.

McCook, D.K, and Grotrian, K.O., “Using SIGMA/W to predict hydraulic fracture in an earthen embankment”, Proceedings: Dam Safety, ASDSO, September 2010.

Mori, A. and Tamura, M., “Hydrofracturing pressure of cohesive soils”, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 27, No. 1, 14-22, Mar 1987.

Schmertmann, J. H., “Measure and use of the insitu lateral stress”, The Practice of Foundation Engineer, Department of Civil Engineering, Northwestern University, 1985.

Seed, H.B., and Duncan, J.M., 1981, “The Teton Dam - a retrospective review”, Proceedings of the Tenth international Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweden, June 1981, p. 219-238.

Sherard, J.L., “Loss of water in boreholes in impervious embankment sections”, Proceedings, 10th ICOLD Congress, Montreal, Vol. VI, 1970, 377-381.

Sherard, J.L., Decker, R.S. and Ryker, N.L., “Hydraulic fracturing in low dams of dispersive clay,” Proceedings of the Specialty Conference on Performance of Earth and Earth-Supported Structures, ASCE, June, 1972, Vol. 1, Part I, pp. 563-590.

Staheli, K., Price, C.G., and Wetter, L., “Effectiveness of hydrofracture prediction for HDD design”, North American Society for Trenchless Technology, No Dig Show, May, 2010.

US Army Corps of Engineers, “Foundation completion report Patoka lake dam, Indiana”, Appendix E, Analysis of Grouting Effectiveness and Distribution as Observed During Excavation, July 1979.

US Army Corps of Engineers, “Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling, Technical Report CPAR-GL-98-1, April 1998.

Xia, H. and Moore, I.D., “Estimation of Maximum Mud Pressure in Purely Cohesive Material during Directional Drilling, Geomechanics and Geoengineering: An International Journal, Vol. 1, No.1, 3-11. 2006.

Yanagisawa, E. and Panah, A.K., "Two dimensional study of hydraulic fracturing criteria in cohesive soils", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 34, No. 1, 1-9, Mar. 1994.

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APPENDIX B

DRILLING PROGRAM PLAN (DPP) OUTLINE FOR DRILLING IN AND NEAR EMBANKMENT DAMS

Drilling in and near embankment dams must subscribe carefully to the "do no harm" philosophy. Dams are not places for standard investigation techniques; they require different drilling procedures because there are significant risks. Incorrect drilling, grouting, or borehole abandonment procedures could lead to damage of the structure by hydrofracturing, erosion, drain contamination or other mechanisms, during drilling operations, instrumentation installation, borehole completion, and borehole abandonment. All design and field personnel need to understand the existing subsurface conditions and potential problems and damage that the drilling could trigger.

An approved Drilling Program Plan (DPP) is required for any exploration drilling or remediation drilling (including grouting) work to occur in or near an embankment dam. DPPs shall be prepared and approved by experienced geotechnical engineers and/or engineering geologists familiar with subsurface exploration techniques and methods.

The following outline describes the basic information that should be developed and included in the DPP that is to be submitted by the licensee. Additional information, discussion, and recommendations on the items presented in Appendix B are provided in the guidelines. It is strongly recommended that the DPP follow the following organizational structure.

1. Purpose

The purpose of the drilling program needs to be clearly defined and summarized in the plan. The DPP should provide sufficient discussion, details, and figures to ensure that the proposed exploration will accomplish its goals and prevent damage to the dam. The need for any investigation (drilling, testing, etc.) at a dam site should have been presented to and accepted by FERC prior to developing a DPP program. In addition, it should have been demonstrate that any potential damage to the structure created by the drilling and associated processes is outweighed by the need for the drilling data.

2. Existing Information

Before preparing a DPP, the licensee or its consultant should review the subsurface, design, and construction information available in the Supporting Technical Information Document (STI) and their files to properly evaluate the risks associated with the proposed drilling program. The information review typically includes, but is not limited to:

- Subsurface profiles and piezometric conditions;
- Geologic mapping, logs, and reports portraying information from previous investigations and construction;
- Foundation reports;
- Embankment construction reports;

- As-built drawings;
- Archived records;
- Construction reports;
- Construction photos;
- Instrumentation plans; and
- Available laboratory analyses.

Based on this review, a summary of the existing information should be included in the DPP.

3. Essential Geologic and Engineering Drawings

The DPP should include a complete set of drawings depicting the current subsurface conditions. This detailed set of foundation and embankment drawings typically requires a plan drawing showing all previous subsurface investigation locations, profile drawings, and sections of the embankment in the areas of proposed exploration. The sections should be drawn to scale (no vertical exaggeration) and should show the locations and depths of the proposed borings along with all available factual information and appropriate geologic or engineering interpretations. The information on the plan, profile and sections should be detailed, include all available data significant to the planned explorations, and be supplemented by additional discussion in the text of the DPP, as appropriate. At a minimum, the following information should be included, as applicable:

- Embankment zones, including added berms, filters, blankets, and drains;
- Estimated extent of any other zones of interest;
- Details of subsurface material classifications, including relevant laboratory test results such as Atterberg Limits, grain size analyses, and dispersivity test results, as applicable;
- Geologic contacts and continuity supported by all nearby drilling and sampling details;
- Contours of the top of rock or any other layer of particular interest;
- Piezometer locations showing screened influence zones and recorded piezometric levels tied to the reservoir water level. Whether or not the dam includes active piezometers, the estimated phreatic surface through the embankment should be clearly shown on all relevant cross-section drawings included in the DPP. In addition, the basis for determination of the estimated phreatic surface should be clearly described in the DPP.
- Inclinometer locations showing any shear zones or areas of deformation;
- Standard Penetration Test (SPT) blow counts or other in-situ test results;
- Geophysical data, where useful (e.g. downhole and/or crosshole shear wave velocity profiles);
- Seepage areas tied to geologic units; and
- Location of all structures, including seepage control features, outlet works, etc.

4. Drilling Scope and Methodology

The plan should thoroughly describe the scope and methods that will be used for the drilling program. At a minimum, the following information should be included:

- Number, location, depth, diameter, and inclination of the proposed borings;
- Drilling and sampling methods, including a description of the drilling equipment to be used (e.g. track-mounted vs. truck-mounted drill rig). The DPP should include justification for the proposed methods and equipment based on the expected subsurface conditions. In particular, if any drilling fluids will be used to advance the borings, the DPP must include a detailed explanation of why these procedures must be used, how the potential for hydraulic fracturing will be mitigated, and how continuous monitoring of the fluid pressures will be accomplished during the drilling. The allowable fluid pressures so as to prevent hydraulic fracturing should be included in the DPP, along with supporting calculations, as appropriate.
- List of ASTM standards and methods that will be followed to perform the drilling.
- Anticipated materials to be drilled and sampled;
- Required sample types (disturbed or undisturbed), sizes, and anticipated depths;
- Procedures for identifying underground utilities, and other surface or subsurface obstacles prior to the drilling; and
- Site Access and accessibility of the boring locations (see paragraph 11). .

5. Field and Laboratory Testing Program

The DPP should provide information on the proposed testing program, which should include both field and laboratory testing. A detailed description of the in-situ testing proposed at each boring should be provided, including the type, location (depth), and specific testing method(s) (i.e. ASTM standards, etc.) to be used. The plan should also describe the anticipated laboratory testing program.

6. Instrumentation Installation

If instrumentation is being installed in one or more borings, the materials, location, and procedures that will be used to construct and install the proposed instrument should be described in the DPP. Appropriate figures including installation details for the instruments should also be drafted and included in the plan. For piezometers and monitoring wells, these details should include the following items, at a minimum:

- Installation depth;
- Pipe material type, length, and diameter, as well as the methods that will be used to centralize the pipe;
- Depth of screened interval and the slotted screen size;
- Type, gradation, depth range, and annular thickness of the filter/drain pack material. The DPP must demonstrate that the proposed filter/drain pack material will adequately meet filter and drainage compatibility criteria with both the surrounding embankment soils and the slotted screen size of the piezometers/wells.
- Type, mixture, depth range, and annular thickness of the bentonite or cement grout seal, as applicable;
- Procedures for monument installation or other near-surface (i.e. within the upper five feet) abandonment methods, as applicable; and
- Procedures for developing the piezometers/wells. In particular, if water or air pressures

will be introduced, the DPP must include reasons why these pressures must be used in order to develop the piezometer/well and indicate how this will be implemented so as to avoid causing any damage to the piezometer/well or surrounding embankment. The DPP must indicate how continuous monitoring of the fluid pressures will be accomplished during the development process, state an allowable fluid pressure that will not be exceeded, and include supporting calculations, as appropriate.

7. Monitoring

The DPP should provide details on any proposed monitoring and evaluation of the drilling activity. The plan should describe the type of monitoring (piezometer, inclinometers, etc.), frequency, and purpose for monitoring. If appropriate, threshold limits could be determined for specific drilling scenarios.

8. Emergency Procedures

A discussion should be provided as to what materials and methods will be used to prevent damage to the dam should problems such as loss of drilling fluids, artesian pressures or seepage be encountered during the explorations. The plan should include an emergency contact list and personnel notification flow chart.

9. Borehole Completion

All boreholes in and around embankment dams should be sealed after completion. Completing a borehole by backfilling with drill cuttings is <u>not acceptable</u>. The proposed materials (grout mix) and field procedures that will be used to backfill the borehole should be described in the DPP, along with the estimated quantities required to backfill the borehole. Additional information on backfilling of boreholes is provided in the guidance.

10. Personnel Experience

The DPP should clearly indicate the specific personnel that will be on site either performing or observing the drilling work, and their respective roles and responsibilities. Resumes for all of the relevant project personnel (including the project manager, field geologist/engineer, and lead driller) should be included in the DPP or submitted prior to start of work. The level of experiences required for each of the specific personnel performing the work is described in the guidelines.

11. Site Access, and Environmental Consideration

The DPP should include information on the proposed procedures to access the boring locations, which may include details for constructing and maintaining access roads and for mitigating any adverse impacts that might be caused by its construction. The DPP, if applicable, should address any adverse impact to the embankment stability or seepage from the construction of access roads within the footprint of the dam. For access roads which will be constructed through areas of previously undisturbed ground, additional consultation with FERC's Division of Hydropower

Administration and Compliance (DHAC) will be required prior to FERC approval of the DPP. The DPP should describe the procedures for identifying underground utilities, and other surface or subsurface obstacles prior to the drilling.

12. Documentation and Coordination

The DPP should provide details on the documentation, logging, and submission of drilling data. Since there is always a possibility that some changes will need to be made in the field due to the specific subsurface conditions encountered, the DPP should describe how changes and deviations from the approved DPP will be communicated and coordinated with FERC. Also, any significant differences from expected conditions which could be an indication of a potentially serious dam safety issue must be reported immediately to the FERC Regional Engineer.

In addition, the DPP should include an overall schedule and duration of drilling activities.

13. Evaluation of Potential Risks

The DPP must document the licensees' assessment of the risks associated with the proposed drilling and indicate how they intend to avoid or mitigate them. Among other topics, this section should address the risks of causing hydraulic fracturing of the embankment, as well as the risks of causing erosion, blowout, contamination of drainage materials, or other potential dam safety issues as a result of the proposed drilling program. The DPP should also outline the nearby instruments whose behavior will be monitored during the investigation, their expected response, and contingency plans for unexpected response.