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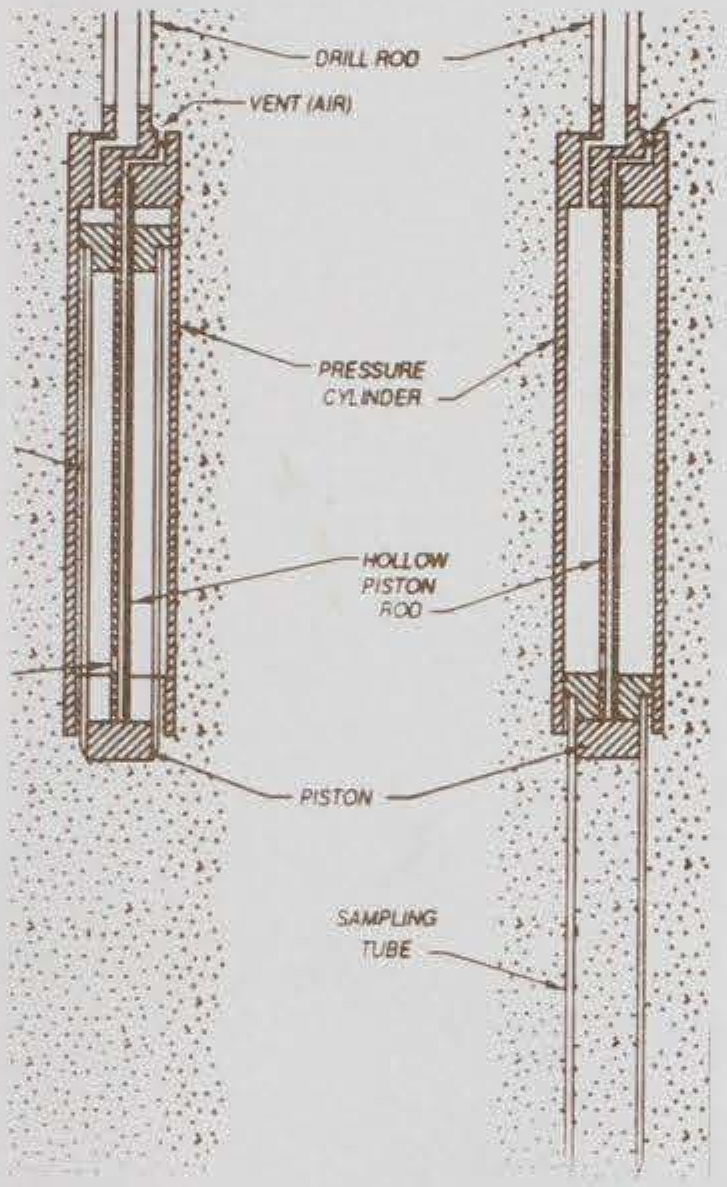

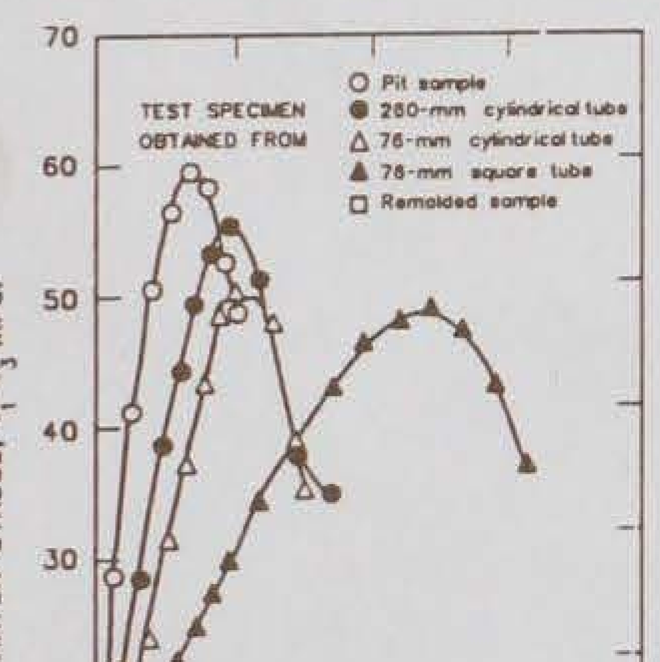

GEOTECHNICAL
LABORATORY

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EFFECT OF SAMPLING DISTURBANCE ON LABORATORY-MEASURED SOIL PROPERTIES

by

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Soil properties must be accurately determined for reliable and economical design of geotechnical structures. Typically, "undisturbed" soil samples are taken from a prospective foundation and brought to the laboratory where the "in situ" properties of interest are measured. However, laboratory-determined properties are impacted by the amount and kind of disturbance which occurs during sampling. Design of foundations and soil structures based on properties different from true in situ properties could result in structures with performance and maintenance problems, or unnecessarily expensive structures if designs are made very conservative to compensate for sampling disturbance.

This report discusses some of the causes of sampling disturbance, how various investigators have analyzed mechanisms causing disturbance, and how equipment design and sampling techniques have been modified to minimize

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disturbance. The effect of sampling disturbance on various soil properties (strength, Young's modulus, compressibility, permeability) is discussed along with laboratory techniques for minimizing the effects of sampling disturbance. Severity of sampling disturbance as the result of (a) release of hydraulic and earth pressure, (b) compression, shear, and tension during sampling, (c) mechanical shock and temperature change during storage, and (d) compression and shear during extraction and specimen preparation in the laboratory are discussed.

14. (Concluded).

Area ratio	Laboratory testing	Sampling disturbance
Compressibility	Perfect sampling	Shear strength
Inside clearance ratio	Permeability	Thin walled sampling tube

PREFACE

A literature study to review current research on the effect of sampling disturbance and how laboratory-measured properties are affected by sampling disturbance was requested by the Civil Works Research and Development (CWR&D) Geotechnical Field Review Group and authorized by the Office, Chief of Engineers (OCE), US Army. The work was performed under CWR&D Work Unit 32676 at the US Army Engineer Waterways Experiment Station (WES) during March and April 1992. CWR&D Work Unit 32676, Laboratory Determination of Soil Properties, is appropriate because soil sampling disturbance will impact laboratory determination of soil properties.

The study was performed by Mr. Paul A. Gilbert, Soils Research Center (SRC), Soil and Rock Mechanics Division (S&RMD), Geotechnical Laboratory (GL), who wrote this report under the direct supervision of Mr. G. P. Hale, Chief, SRC, and the general supervision of Dr. Don Banks, Chief, S&RMD, and Dr. W. F. Marcuson III, Director, GL. Mr. Richard F. Davidson was the OCE Technical Monitor.

Director of WES during publication of this report was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
pounds (force) per square inch	6894.757	pascals

EFFECT OF SAMPLING DISTURBANCE ON LABORATORY-MEASURED SOIL PROPERTIES

PART I: INTRODUCTION

Background

1. The weight of any and all engineering structures ultimately comes to bear on the earth through foundations on soil or rock. The transfer of load from the foundation of a structure to underlying soil or rock produces shear stress, internal deformation, and settlement of the ground surface under and adjacent to the structure. If the foundation load is sufficiently large, the foundation will fail in shear or undergo settlement so large that the function of the supported structure may be impaired even though the foundation does not fail completely or catastrophically.

2. A critical requirement in civil engineering today is to provide economical designs of foundations and compacted earth structures with acceptable levels of safety with respect to shear failure or excessive amounts of settlement. A typical procedure for achieving a required design is to acquire specimens of a prospective foundation soil, perform laboratory tests to measure the property or properties of interest, then design the required structure based on analyses using laboratory-measured soil properties and temper the design with experience (that is, apply an appropriate safety factor for the type of structure under consideration).

3. Two main classes of soil samples may be taken for laboratory testing. In the first case, it is only necessary to obtain a sample which is relatively complete and representative of the mineralogy, grain-size distribution and, in most instances, water content of the surrounding in situ soil. Such samples are obtained for classification tests such as grain-size analyses and Atterberg limits. Structural disturbance of the soil while taking such samples is obviously unimportant because soils must be completely disturbed in preparation for these soil index and classification tests. These are called "disturbed" soil samples. In the second case, soil disturbance must be reduced to an absolute minimum because these soil samples will be used to determine stress-strain and strength characteristics, in place density (or void ratio), degree of saturation, compressibility, and perhaps coefficient of

permeability. These are called "undisturbed" soil samples. Sampling disturbance in this case will result in laboratory measurement of properties which may be substantially different from those of the in situ (undisturbed) soil. Design of foundations and soil structures based on properties different from true in situ properties could either result in structures with chronically impaired performance which require continuous expensive maintenance, or in unnecessarily expensive structures if very conservative judgment is used to overcompensate for sample disturbance in "undisturbed" samples. In either event sampling disturbance could have undesirable economic consequences.

Objective

4. Considerable uncertainty can enter the process of sampling, testing, analysis, and design for economical and safe soil structures through sampling disturbance in "undisturbed" soil samples. This uncertainty can be at least partially removed if the way in which disturbance affects laboratory-measured soil properties is understood. The objective of the investigation reported herein is to enhance understanding of how sampling disturbance influences laboratory-measured soil parameters by reviewing and summarizing pertinent recent studies performed to evaluate the effect of sampling disturbance on laboratory measured soil properties.

PART II: SOIL SAMPLING

Early Work by Committee on Sampling and Testing

5. A very comprehensive body of work prepared by M. Juul Hvorslev (1949) began in 1937 with organization of the Committee on Sampling and Testing. The committee was organized under the Soil Mechanics and Foundations Division of the American Society of Civil Engineers and the work summarized the state of the art in subsurface exploration and soil sampling up to the time when the report was finished. The effort was sponsored jointly by the Engineering Foundation, Harvard University, and the US Army Engineer Waterways Experiment Station. The work represents the cooperative effort of many significant practicing civil engineers, academicians, and organizations of the period. A final report was completed in late 1947 by Hvorslev while working at Harvard University; the report was published in 1949.

6. It should be noted that this major work is still regarded as a standard in subsurface exploration and soil sampling and is referenced in most of the publications reviewed for this summary. For example, guidance on dimensions and geometric properties given by Hvorslev in the 1949 work is used today without modification in the design and sizing of drive samplers. The most widely used parameters are area ratio, C_a , and inside clearance ratio, C_i . These ratios are defined to be

$$C_a = \frac{D_w^2 - D_e^2}{D_e^2} \tag{1}$$

and

$$C_i = \frac{D_s - D_e}{D_e} \tag{2}$$

where

D_w = outside diameter of the tube that enters the soil during sampling

D_e = inside diameter of the cutting edge of the sampling device

D_s = inside diameter of the sampling tube above the cutting edge

7. The area ratio is approximately equal to the ratio between the volume of soil displaced by the sampling tube to the volume of the sample taken. Hvorslev suggests that penetration resistance of the sampler, possibility of excess soil entering the sampler, and danger of disturbance in the soil sample all increase with increasing area ratio; therefore the ratio should be limited to values of the order of about 10 percent.

8. Because soil is under great stress as it enters a sampler, it has a tendency to expand laterally. The inside clearance should be large enough to allow some lateral expansion but not so large as to allow excessive deformation and the associated sample disturbance. Additionally some wall friction must be maintained between sampler and soil, otherwise the soil will be lost during withdrawal. Hvorslev suggests that, for general practice, an inside clearance ratio between 0.75 and 1.5 percent is appropriate for long samplers and between 0 and 0.5 percent for very short samplers. However, he states that best results are obtained when the clearance is customized to accommodate characteristics of a specific soil.

Methods of Undisturbed Soil Sampling

9. Two methods of obtaining undisturbed soil samples are generally used in practice: (a) a procedure in which a block soil sample is hand-cut from soil exposed in an excavation; and (b) sampling with a thin-walled tube sampler of the type described by Hvorslev (1949). A table presented by Marcuson and Franklin (1979) in a work describing undisturbed sampling of cohesionless soil summarizes main features of the two methods and is included here as Table 1. A third method devised and used by Geotechnical Engineers, Inc., (GEI) of Winchester, MA, is included in the table; however the GEI procedure is a variation of the hand-cut block procedure in that an in-place soil sample is hand-trimmed into a cylindrical sample tube supported on and guided by a tripod.

10. Because of the size of the sample recovered and the absence of boundary stresses and displacements applied as a sampler is pushed into a soil medium, it is generally acknowledged that hand-cut blocks yield the best quality in undisturbed soil samples. However, excavation down to the level of sample recovery is required and dewatering is necessary if the level is below the water table. Additionally, the state of in situ stress in hand-cut blocks is unavoidably changed by excavation down to the level of the block.

Table 1

Methods of Undisturbed Sampling of Cohesionless Soil

Method	Procedure	Applicability	Limitations and Pitfalls
Hand-cut Block or Cylindrical Sampler	Sample is cut by hand from soil exposed in excavation (USBR 1960, pp 346-349; Terzaghi and Peck 1968, pp 312-314).	Highest quality undisturbed samples in cohesive soils, cohesionless soils, and soft rock.	Requires accessible excavation and dewatering if below water table. Extreme care is required in sampling cohesionless soils. The state of stress is changed by the excavation.
GEI Sampler	Sample is hand-trimmed into cylindrical sample tube that is supported and guided by a tripod holder (Geotechnical Engineers, Inc., 1976; Marcuson 1978).	Undisturbed samples in cohesionless soils, of quality comparable to hand-cut block sample.	Requires accessible excavation and dewatering if below water table. The state of stress is changed by the excavation.
Thin-Walled Tube Samplers	Thin-walled tube is pushed into soil at bottom of boring. (ASTM D 1587-67; US Army 1972, Ch. 4).	Undisturbed or representative samples in cohesive soils and cohesionless soils that are free of gravel particles.	Not suitable for use in extremely hard soils, gravel, or stony soils. Strict attention to details of equipment and procedure is required to obtain undisturbed samples of good quality (US Army 1972, Ch. 3 & 4; Hvorslev 1949, pp 83-139).

11. Sampling using thin-walled tube samplers described by Hvorslev (1949) yields good quality undisturbed soil samples but this technique is most appropriate for clays and granular soils that do not contain gravel particles; additionally, the technique is not suitable for very stiff soils. Laboratory test results performed on soil specimens prepared from hand-cut blocks and thin-walled tubes of various sizes will be compared and discussed later.

12. The major types of thin-walled tube samplers are compiled and described in Table 2 presented by Marcuson and Franklin (1979). The term "specific recovery ratio" used in the table is defined by Hvorslev (1949) as the ratio of the increment of length of sample entering the tube to the increment of tube advance. Each sampler described in Table 2 has its own advantages and disadvantages and some samplers are better suited for certain material types and soil consistencies than others. Piston samplers with small area ratios are generally acknowledged to furnish high quality samples of cohesive soils even if the materials are very soft and sensitive (Terzaghi and Peck 1968; Hvorslev 1949). Samplers of the fixed-piston type are particularly advantageous in minimizing disturbance in soft soils. For example, when an empty sampler begins to be pushed into a soil mass, friction and adhesion on the outside of the tube tend to aggravate instability in the bottom of the

Table 2

Major Types of Thin-Walled Tube Samplers

Sampler	Procedure	Applicability	Limitations and Pitfalls
a. Fixed-Piston Sampler	Thin-walled tube is pushed into soil, with fixed piston in contact with top of sample during push. (US Army 1972, Ch. 3; Hvorslev 1949, pp 128-130; USBR 1960, pp 349-379.)	Undisturbed samples in cohesive soils, silts, and sands, above or below the water table.	Some types do not have positive prevention of piston movement.
b. Hydraulic Piston Sampler (Osterberg)	Thin-walled tube is pushed into soil by hydraulic pressure. Fixed piston in contact with top of sample during push. (Osterberg 1952 and 1973; US Army 1972, Ch. 3).	Undisturbed samples in cohesive soils, silts, and sands, above or below the water table.	Not possible to limit the length of push or determine amount of partial sampler penetration during push. Earlier version does not have vacuum breaker in piston.
c. Stationary Piston Sampler	Thin-walled tube is pushed into soil. Piston at top of sample is free to move upward but is restrained from downward movement by a friction lock.	Undisturbed samples in stiff cohesive soils; representative samples in soft to medium cohesive soils, silts, and some sands.	Piston does not provide positive control of specific recovery ratio.
d. Free-Piston Sampler	Thin-walled tube is pushed into soil. Piston rests on top of soil sample during push (US Army 1972 Ch. 3; Hvorslev 1949, p 131).	Undisturbed samples in stiff cohesive soils; representative samples in soft to medium cohesive soils, and silts.	Not suitable for sampling in cohesionless soils. Free piston provides no control of specific recovery ratio.
e. Open-Drive Sampler	Thin-walled, open tube is pushed into soil (US Army 1972, p 133; USBR 1960, pp 361-367).	Undisturbed samples in stiff cohesive soils. Representative samples in soft to medium cohesive soils and silts.	Not suitable for sampling in cohesionless soils. No control of specific recovery ratio.
f. Pitcher Sampler	Thin-walled tube is pushed into soil by spring above sampler while outer core bit reams hole. Cuttings removed by circulating drilling fluid (Terzaghi and Peck 1968, pp 310-312).	Undisturbed samples in hard, brittle, cohesive soils and sands with cementation. Representative samples in soft to medium cohesive soils and silts. Disturbed samples may be obtained in cohesionless materials with variable success.	Frequently ineffective in cohesionless soils.
g. Denison Sampler	Hole is advanced and reamed by core drill while sample is retained in nonrotating inner core barrel with core-catcher. Cuttings removed by circulating drilling fluid. (US Army 1972, pp 312-313; UBSR 1960, pp 355-361).	Undisturbed samples in stiff to hard cohesive soil, sands with cementation, and soft rocks. Disturbed samples may be obtained in cohesionless materials with variable success.	Not suitable for undisturbed sampling in loose cohesionless soils or soft cohesive soils.

(Continued)

Table 2. (Concluded)

Sampler	Procedure	Applicability	Limitations and Pitfalls
h. Submersible Vibratory (Vibracore) Sampler	Core tube is driven into soil by vibrator. (Tirey 1972)	Continuous representative samples in unconsolidated marine sediments.	Because of high area ratio and effects of vibration, samples are disturbed.
i. Underwater Piston Corer	Core tube attached to drop weight is driven into soil by gravity after a controlled height of free fall. Cable-supported piston remains in contact with soil surface during drive (Noorany 1972).	Representative samples in unconsolidated marine sediments.	Samples may be seriously disturbed (McCoy 1972).
j. Gravity Corer	Open-core tube attached to drop weight is driven into soil by gravity after free fall (Noorany 1972).	Representative samples at shallow depth in unconsolidated marine sediments.	No control of specific recovery ratio. Samples are disturbed.

drill hole (particularly in soft deposits) and to force soil into the tube, actually causing it to rise in the tube faster than the rate of descent of the tube. However, after the tube has been partially filled, friction and adhesion on the inside of the tube oppose the rise of material into the tube and (under extreme conditions) can completely block the tube, displacing soft underlying layers and seams so that they either do not enter the tube or are badly disturbed before or during entry. Consequent disturbance resulting from these circumstances can be minimized, to an extent, by providing a piston inside the sampling tube. A fixed-piston type of sampler (which is hydraulically operated) is shown schematically in Figure 1, which was taken from Terzaghi and Peck (1968). As seen in Figure 1, the internal piston plugs the lower end of the tube as the sampler is placed on the surface in solid contact with the soil to be taken. The piston is held at this elevation/position as the tube is pushed into the soil. Initially, the piston prevents entry of a greater length of sample than the length of tube penetration. In the final stages of the stroke/push, the piston holds soil inside the tube in place, since the sample cannot pull away from the piston without creating a vacuum. Therefore, the piston facilitates the rise of a sample into the tube and helps control certain mechanisms which cause and aggravate disturbance. Additionally, the piston serves to assist in the control of disturbance during removal. When the tube has been pushed to the bottom of its stroke, the

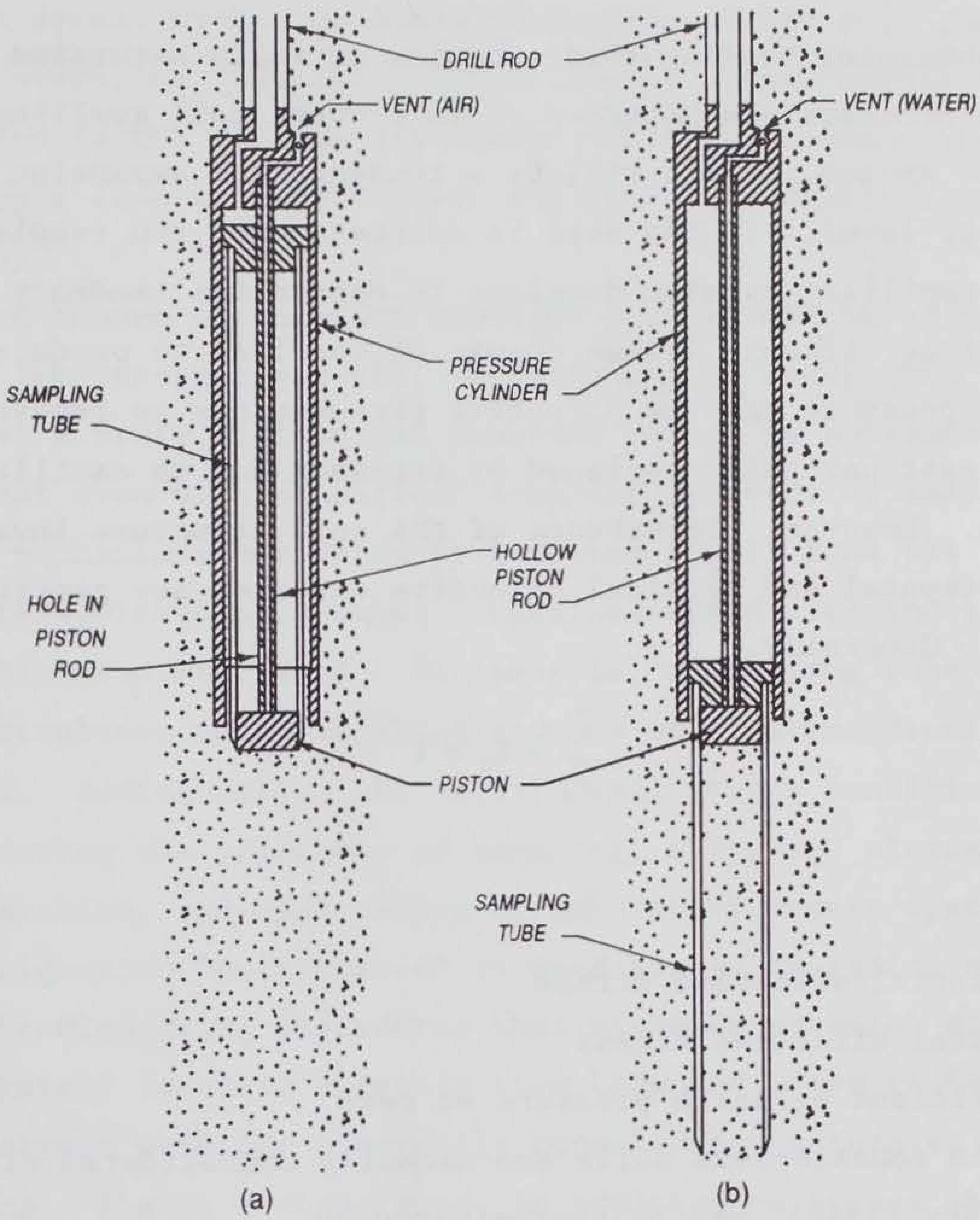

Figure 1. Hydraulically operated piston. (a) Lowered to bottom of drill hole, drill rod clamped in fixed position at ground surface. (b) Sampling tube after being forced into soil by water supplied through drill rod (after Terzaghi and Peck 1968)

piston is locked in position relative to the tube and the entire assembly is rotated to separate the sample from underlying soil. The piston and tube are then withdrawn from the hole. Presence of the piston and the sequence of operations described minimize disturbance in soil samples that occurs as the result of compression, extension, and shear.

13. In obtaining "undisturbed" samples of fully saturated cohesive soil, capillary stresses are depended on to prevent soil swelling as it is removed from the ground. There will be a tendency for expansion in response to reduced stress level. If the soil is saturated and the sample has no access to water, capillary tension develops to oppose the tendency of the soil to expand. Little, if any, volume change is required to produce this negative pore water pressure and the effective stresses due to removal of overburden are at least partially replaced by stresses due to capillary tension/suction. However, disturbance of the soil structure invariably occurs because the horizontal and vertical effective stresses are generally not equal in soil masses. Generally,

$$\frac{\bar{\sigma}_x}{\bar{\sigma}_y} = K_o \neq 1 \tag{3}$$

where

$\bar{\sigma}_x$ = horizontal effective stress

$\bar{\sigma}_y$ = vertical effective stress

K_o = coefficient of earth pressure at rest

14. K_o in cohesionless soils and normally consolidated clays, as determined by Jaky (1948), may be approximated by

$$K_o = 1 - \sin(\phi') \tag{4}$$

where

ϕ' = effective angle of internal friction

K_o is generally less than unity except in the case of highly overconsolidated clays and clay shales.

15. After sampling, when the total stresses in the soil (in the horizontal and vertical directions) are approximately zero, the effective stresses are equal to the (negative) pore water pressure produced by surface tension of water acting in the soil pores. The capillary suction/negative pore water pressure produced in the pores is a function of pore size, pore size distribution, degree of saturation, and, to some extent, temperature (since surface tension is a function of temperature). Therefore effective stresses are

changed as the result of removing the soil from its in situ environment and this change in stress system produces distortion of the soil skeleton and unavoidable disturbance. Skeletal distortion of a soil sample from the mechanisms described is the minimum disturbance which will occur as the result of removing a sample from its environment and is termed "perfect sampling" (Ladd and Lambe 1964).

16. Even though the need to minimize disturbance in soil samples on which certain laboratory tests will be performed is recognized, it is also recognized that a truly undisturbed specimen cannot be obtained. It has been shown above that even with a "perfect sampling" procedure, sample disturbance occurs due to mechanisms activated by removing a soil from its in situ stress and temperature environment. Other mechanisms act during and after sampling to cause additional disturbance. For example, a sampling tube, no matter how thin walled, displaces material which induces strains and density changes in soil recovered. Additionally, the in situ soil stress condition is irreversibly changed during the processes of handling, shipping, storage, extrusion, specimen preparation, and application of laboratory stress system. In this sense, the designation "undisturbed" is used simply to indicate a sample obtained and handled using procedures that minimize material disturbance.

17. Figure 2 is taken directly from Ladd and Lambe (1964) and shows a hypothetical stress path for a normally consolidated saturated clay element during sampling. Figure 2 shows that, in addition to stress changes as the result of drilling, sampling, and removal of the soil from the tube, unknown and possibly significant stress changes and disturbance occur from trimming and application of triaxial cell pressure. Since the constitutive behavior of clays is affected by stress history, stress-strain and strength properties measured during laboratory strength tests are, without exception, influenced by activities conducted before testing, beginning with material sampling.

Ideal Sampling Approach

18. An account of sampling disturbance similar to, but more detailed than, the "perfect sampling" procedure given by Ladd and Lambe (1964) is given by Baligh, Azzouz, and Chin (1987), who list mechanisms which cause soil disturbance in tube sampling in the chronological order of their occurrence. They state that sample disturbance occurs as the results of: (a) changes in

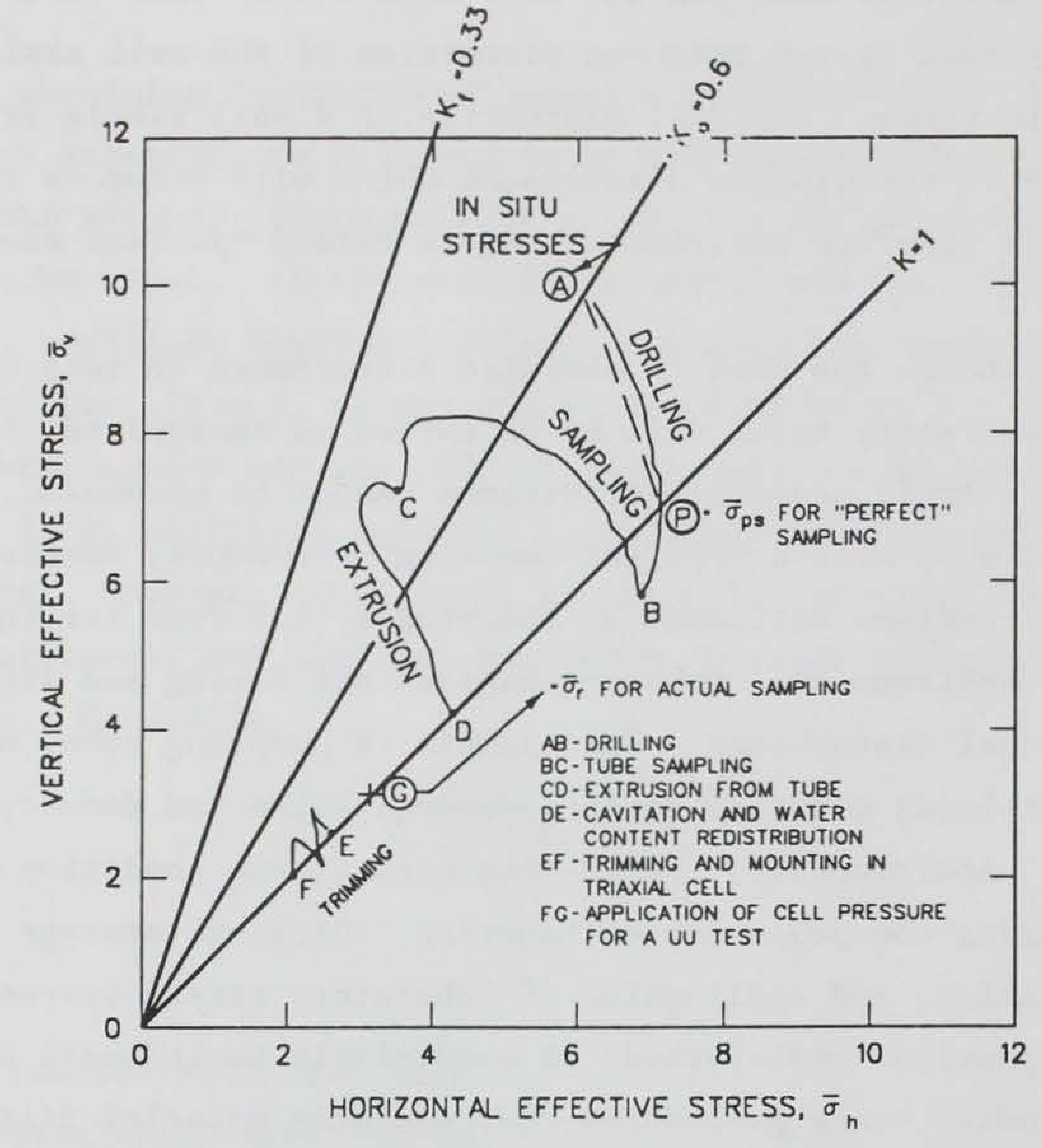

Figure 2. Hypothetical stress path for a normally consolidated clay element during tube sampling (after Ladd and Lambe 1964)

soil conditions ahead of the advancing borehole during drilling operations; (b) penetration of the sampling tube and sample retrieval to ground surface; (c) water content redistribution in the tube; (d) extrusion of the sample from the tube; (e) drying and/or changes in water pressure; and (f) trimming and other activities required to prepare specimens for laboratory testing. Additional examples of sample disturbance which occur in special applications are given; they include expansion of dissolved gases when very deep offshore samples are brought to the surface, the effects of rough handling and transportation, and the effect of temperature changes in chemically or biologically active deposits.

19. Baligh, Azzouz, and Chin (1987) proposed an "ideal sampling approach" (ISA) which is stated to be an extension of the "perfect sampling approach" (PSA) of Ladd and Lambe (1964). The ISA allegedly incorporates the effects of tube penetration, sample retrieval to the surface, and extrusion from the tube, but neglects all other types of disturbance, including

operator-dependent disturbance and soil water content changes. The only difference between the ISA and the PSA is that the ISA attempts to take tube-penetration disturbance into account since the authors suggest that tube-penetration effects are significant. Estimates of tube penetration disturbance at the center line of the sample are made from an analysis of the sample tube geometry based on the inside clearance ratio, and a quantity defined as the aspect ratio which is the ratio of the sampler diameter to the wall thickness. Analysis was carried out by means of a procedure called the strain path method (SPM) which is an extended version of cavity expansion theory. In describing the procedure, Baligh (1985) states that cavity expansion solutions are a one-dimensional and simplistic subset of strain path solutions which are two dimensional and describe the intrusion of a general geometric shape into the region of interest. The SPM technique used in the analysis is based on a graphical procedure derived by Baligh (1975); it shows that a soil sample forced into a thin-walled tube first undergoes a cycle of strain in extension followed by strain in compression. Subsequent laboratory studies demonstrated that, depending on geometric dimensions of the tube and characteristics of the soil being taken, strain level applied to the center line of the sample could be sufficient to produce failure in the soil before it entered the tube. It was also shown in the analysis that soil disturbance defined in terms of shear distortion decreases toward the center line of a tube sampler; therefore soil located on or near the center line is the least disturbed in a tube.

20. Based on a procedure where conditions determined by strain path analysis were enforced on laboratory specimens to simulate distress experienced during sampling, Baligh, Azzouz, and Chin (1987) determined that sampling disturbance effects on the undrained behavior of clays can be reduced by reconsolidating the soil before shear. The two methods suggested are (a) reconsolidate the soil under conditions of no lateral strain (K_0-consolidation) to an effective vertical pressure equal to the in situ vertical overburden pressure, and (b) use the SHANSEP method proposed by Ladd and Foote (1974) in which the soil is consolidated to 1.5 to 2 times the in situ vertical effective overburden pressure, then rebounded to the estimated in situ overconsolidation ratio before undrained shearing. It should be noted, however, that different results are obtained from the two procedures, the differences being most pronounced in normally consolidated soils.

21. In evaluating the effects of sample disturbance, Baligh, Azzouz, and Chin (1987) draw conclusions regarding how effective stress, undrained

shear strength, strain, and stiffness in resedimented Boston Blue clay are affected by simulating sampling disturbance on soil specimens in laboratory apparatus. As determined in the laboratory, mean effective stress in the soil prior to shear is typically reduced from 66 percent of the in situ vertical effective stress to 19 percent or less by (simulated) sampling. Undrained shear strength is reduced as the result of tube sampling from 32 percent of the in situ effective vertical stress to 24 percent or less (however, it must be noted that in actual sampling, undrained shear strength will be a function of sampler and tube dimensions). The study showed that strain at peak strength is significantly increased by sampling disturbance; strain level at peak strength typically increased from 0.16 to 5 percent or more. Initial soil stiffness is reduced by about a factor of 5 by sampling disturbance.

Variation of Strength With Density in Cohesionless Soils

22. The release of overburden pressure as the result of sampling is significant in cohesive soils and represents a dilemma in obtaining "undisturbed" samples since stress release is bound to cause disturbance. "Undisturbed" sampling in cohesionless soils may present an even more formidable problem since the smallest disturbance may destroy structure and alter density in such soils. The importance of restricting the area ratio of a thin-walled sampler to a value of the order of 10 to 15 percent is emphasized above; it is crucial to minimize this value in taking samples of cohesionless soil since it is a direct measure of the amount of material displacement and therefore the amount of soil densification during sampling. Bowles (1974) presents data on the variation of friction angle in cohesionless soils with density; the relationship, shown in Figure 3, confirms the critical importance of minimizing density disturbance in cohesionless soils during sampling operations. Data on Banding sand were added to (the original five sands in) Figure 3 from results presented by Gilbert (1984). Banding sand is a fine, uniform fraction of Ottawa sand that has a D_{50} size of about 0.2 mm. The figure demonstrates that a density change of 0.1 g/cc (6.2 pcf) in cohesionless material can result in a change in angle of internal friction from 2 to 11 degrees,* depending on the specific material and the initial density. Generally, the materials show a

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

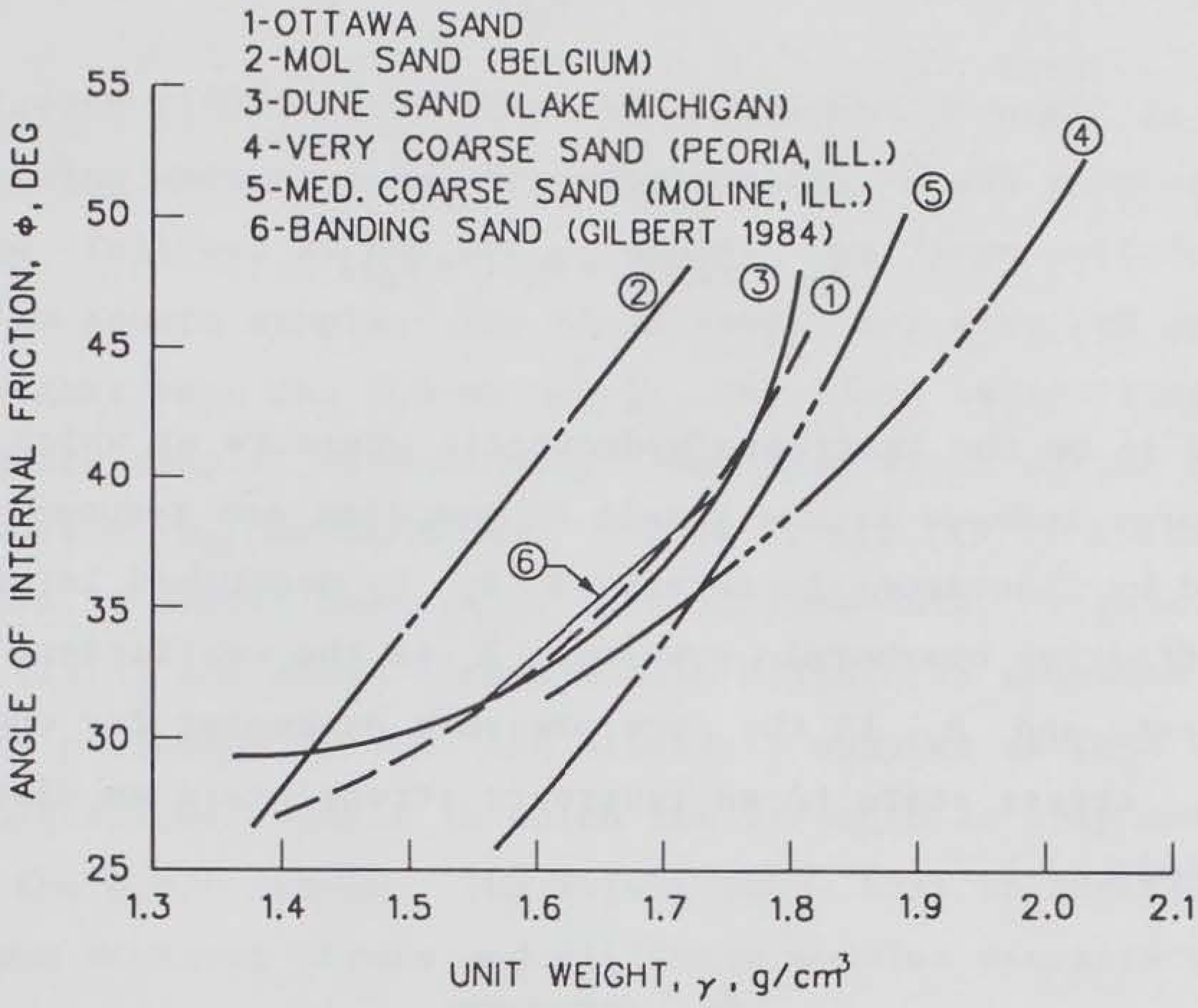


Figure 3. Friction angle variation with unit weight for some sands. (After Bowles 1974)

curvilinear relationship between angle of internal friction and density. The relationships are generally curved such that density changes at higher initial densities result in greater changes in the angle of internal friction.

Disturbance Research by Chantawong

23. Limited research has been performed in Thailand by Chantawong (1973) to assess sampling disturbance in soft clay sampled with various sizes and shapes of tubes as compared with block samples. The sampling tubes used were: (a) a 76-mm square tube with an area ratio of 14 percent; (b) a 76-mm cylindrical tube with an area ratio of 9 percent; and (c) a 260-mm cylindrical tube with an area ratio of 4 percent. The inside clearance was zero for all tubes used in this investigation. Block samples were also taken in the study and used as the control group. An attempt was made to quantify sample disturbance in terms of the degree of disturbance parameter, D_d , defined by Nelson et al. (1971), who used and extended equations developed by Ladd and Lambe (1964).

$$D_d = \frac{\bar{\sigma}_{ps} - \bar{\sigma}_r}{\bar{\sigma}_{ps}} \qquad (5)$$

where

$$\bar{\sigma}_{ps} = \bar{\sigma}_{ov}(K_o + A_u(1 - K_o)) \qquad (6)$$

$\bar{\sigma}_r$ is defined to be the isotropic/hydrostatic pressure at which the capillary suction pressures induced as the result of sampling are reduced to zero. The procedure used by Chantawong to determine $\bar{\sigma}_r$ is described later. $\bar{\sigma}_{ov}$ is the in situ effective overburden pressure, K_o is the coefficient of earth pressure at rest, and A_u is the pore pressure parameter for undrained loading from a K_o stress state to an isotropic stress state as discussed by Ladd and Lambe (1964);

$$A_u = \frac{\Delta_u - \Delta\sigma_h}{\Delta\sigma_v - \Delta\sigma_h} \qquad (7)$$

Δ_u is the change in pore water pressure from in situ condition (u_o) to the residual condition (u_{ps}) after perfect sampling.

$$\Delta_u = u_o - u_{ps} = u_o + \bar{\sigma}_{ps} \qquad (8)$$

24. Ladd and Lambe (1964) use the ratio $\bar{\sigma}_{ps}/\bar{\sigma}_r$ as a quantitative indicator of sample disturbance. If the ratio is equal to unity, then perfect sampling has occurred; the greater the ratio, the greater the sample disturbance. Nelson et al. (1971) defined the "degree of disturbance" factor given by Equation 5. For a perfect sample, D_d is equal to zero; if D_d is equal to unity, the sample has been completely disturbed.

25. Chantawong (1973) performed unconsolidated undrained triaxial tests on recovered samples and determined the residual pore pressure after sampling, u_r , by measuring water pressure at the base of the specimen after increasing the cell pressure in increments (of $\Delta\sigma$) until the pore pressure response, $(\Delta u/\Delta\sigma)$, exceeded 95 percent. The residual pore pressure was then taken to be the difference between the cell pressure and the pore pressure (which is also the negative of the residual hydrostatic stress after sampling, $\bar{\sigma}_r$);

$$-u_r = \Delta\sigma - u = -\bar{\sigma}_r \tag{9}$$

26. Chantawong (1973) found that, if diminished strength is used as a measure for assessing amount of sampling disturbance, block samples showed the least disturbance, followed by the 260-mm sample, the 76-mm cylindrical sample, and the 76-mm square sample. The block sample had strength about 10, 20, and 50 percent higher than the 260-mm sample, the 76-mm cylindrical sample, and the 76-mm square sample, respectively. Analysis based on the degree of disturbance parameter (D_d) as defined by Nelson et al. (1971) did not produce conclusive results, probably because of insufficient data. Figure 4 is taken from Chantawong (1973) and shows stress-strain properties measured in triaxial compression tests performed on tube and block/pit samples as well as a completely remolded/disturbed sample in which the strength is only about 10 percent of that of the block sample. The figure shows that in addition to the facts that maximum deviator stress and stiffness modulus decrease with disturbance, axial strain to maximum deviator stress increases with disturbance.

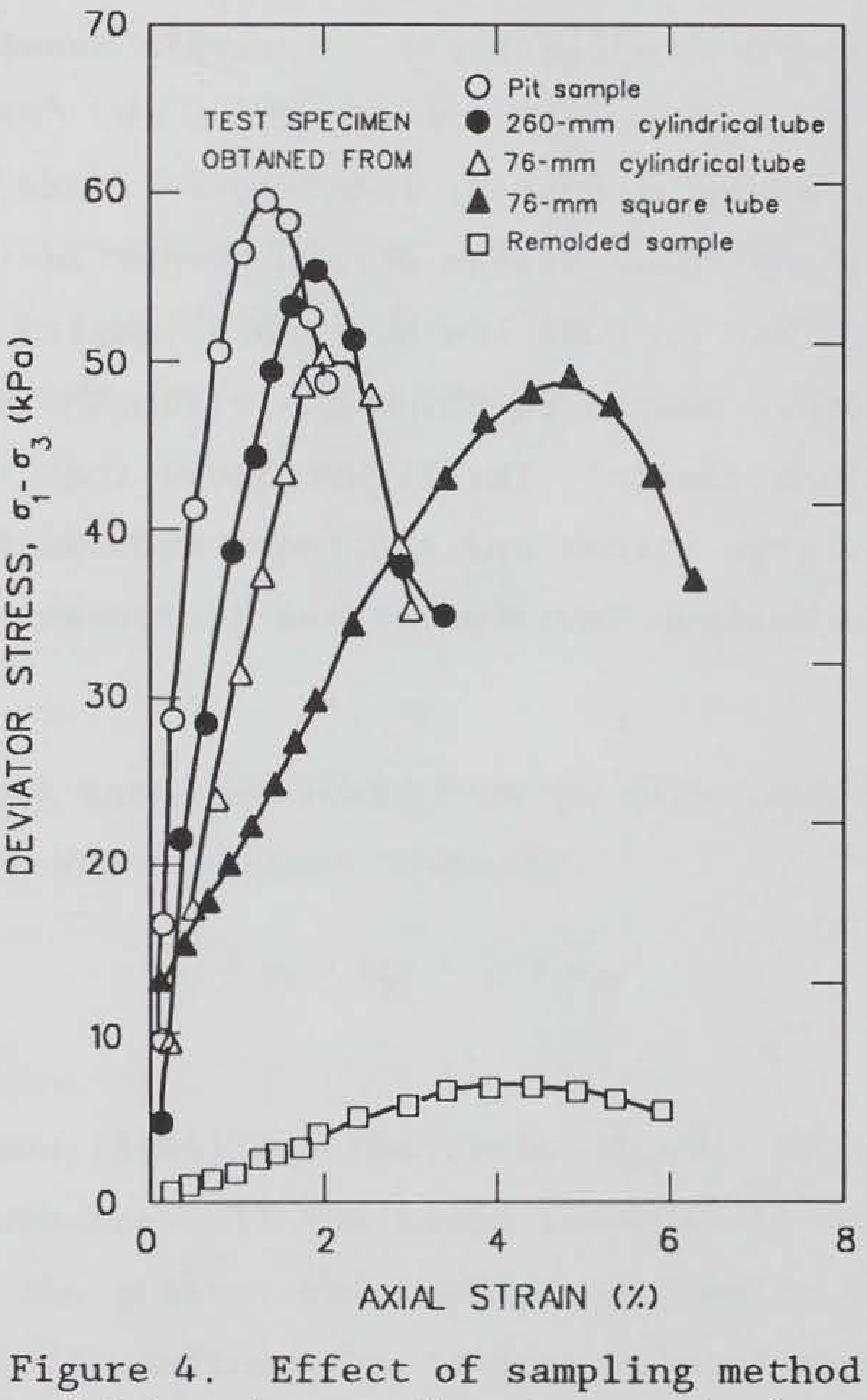


Figure 4. Effect of sampling method and tube size on the stress-strain relationship in unconsolidated tests (Chantawong 1973)

PART IV: WORK AT THE NORWEGIAN GEOTECHNICAL INSTITUTE (NGI)

27. In 1952, NGI designed a 54-mm thin-walled sampler with a fixed piston and removable sample cylinder based on the recommendations of Hvorslev (1949). The area ratio of the 1952 NGI device was 12 percent and the inside clearance ratio was 0.9 percent with a complete description and specifications given by Andresen and Kolstad (1979). It has been the experience at NGI that suction/vacuum between the sample and piston must be avoided when removing the sample from the hole, otherwise substantial sampling disturbance would result. Undisturbed samples taken by NGI are stored in humidity controlled rooms at a temperature of 7 °C, which is the average annual soil temperature in the Oslo area. At NGI, samples are extruded at a constant speed in special extruders where the upper sealing piston is clamped and pulled at the same speed as the speed of extrusion to avoid stressing the specimen at the upper end.

28. NGI usually consolidates simple shear and triaxial specimens to the same effective stress as that in the field (Bjerrum 1973). During consolidation, the relative volume decrease is taken to provide an indication of the quality of the soil tested. NGI uses the following criteria for soft clays:

Volume change, %	Test Specimen Quality
<1	Very good to excellent
1 - 2	Good
2 - 4	Fair
4 -10	Poor
>10	Very poor

29. Stress-strain curves of unconfined compression tests are also used by NGI as a qualitative indicator of the degree of disturbance in samples (Andresen and Kolstad 1979). Strains at failure in soft clays of the order of 3 to 5 percent indicate good quality samples; failure strains of 10 percent indicate significant disturbance. However, these criteria are not valid for heavily overconsolidated clays.

30. Andresen and Kolstad (1979) state that, at NGI, the quality of samples is also evaluated by visual inspection and that high quality sampling is usually achieved using the NGI tube samplers. However, they state that the tendency for greater sample disturbance increases with depth and laboratory tests on soft sensitive lean silty clay from depths greater than 15 to

20 meters appear to confirm this fact when laboratory-determined strengths are compared with in situ vane-determined strengths.

PART V: WORK AT UNIVERSITÉ LAVAL

Université Laval Sampler

31. La Rochelle et al. (1981) describe a 200-mm sampler designed at Université Laval based on a description of soil behavior provided by a model developed by Tavenas and Leroueil (1977) from the concepts of limit analysis and critical state soil mechanics. In the model, the yield locus of a natural clay is represented by a curve having the approximate shape of an ellipse which is roughly centered on the K_o line of normally consolidated clay as shown in Figure 5. The ϕ' line shown in the figure is the failure envelope in the normally consolidated stress range; intersection of the K_o line with the yield curve corresponds approximately to the preconsolidation stress σ_p' in the manner shown on Figure 5. The fact that σ_p' corresponds to the intersection of a 45-deg line from the point of intersection of the K_o line and the yield surface to the horizontal axis was established by empirical correlation. There is no theoretical basis for the correspondence. In the representation of the Tavenas and Leroueil (1977) model, natural clays are usually overconsolidated with the in situ effective stress condition located within the yield curve and likely be-low the K_o line. A typical in situ stress condition in a natural clay deposit might be point A in Figure 5.

32. La Rochelle et al. (1981) identify four common causes of disturbance in tube sampling:

- a. Disturbance of the soil to be sampled before the beginning of sampling, either as a result of poor drilling operation or of direct pushing of a piston sampler.
- b. Mechanical distortion during penetration of the sampling tube into the soil.
- c. Mechanical distortion and suction effects during the retrieval of the sampling tube.
- d. Release of the total in situ stresses.

33. La Rochelle et al. (1981) suggest that the first listed cause of sampling disturbance can be controlled by properly cleaning the borehole and using bentonite slurry. The second and third causes are associated with sampling tube design and can be minimized with proper tube design. The fourth cause is unavoidable and variable, depending on sampling depth and material properties. Limit state analysis was used to show that the structure of a

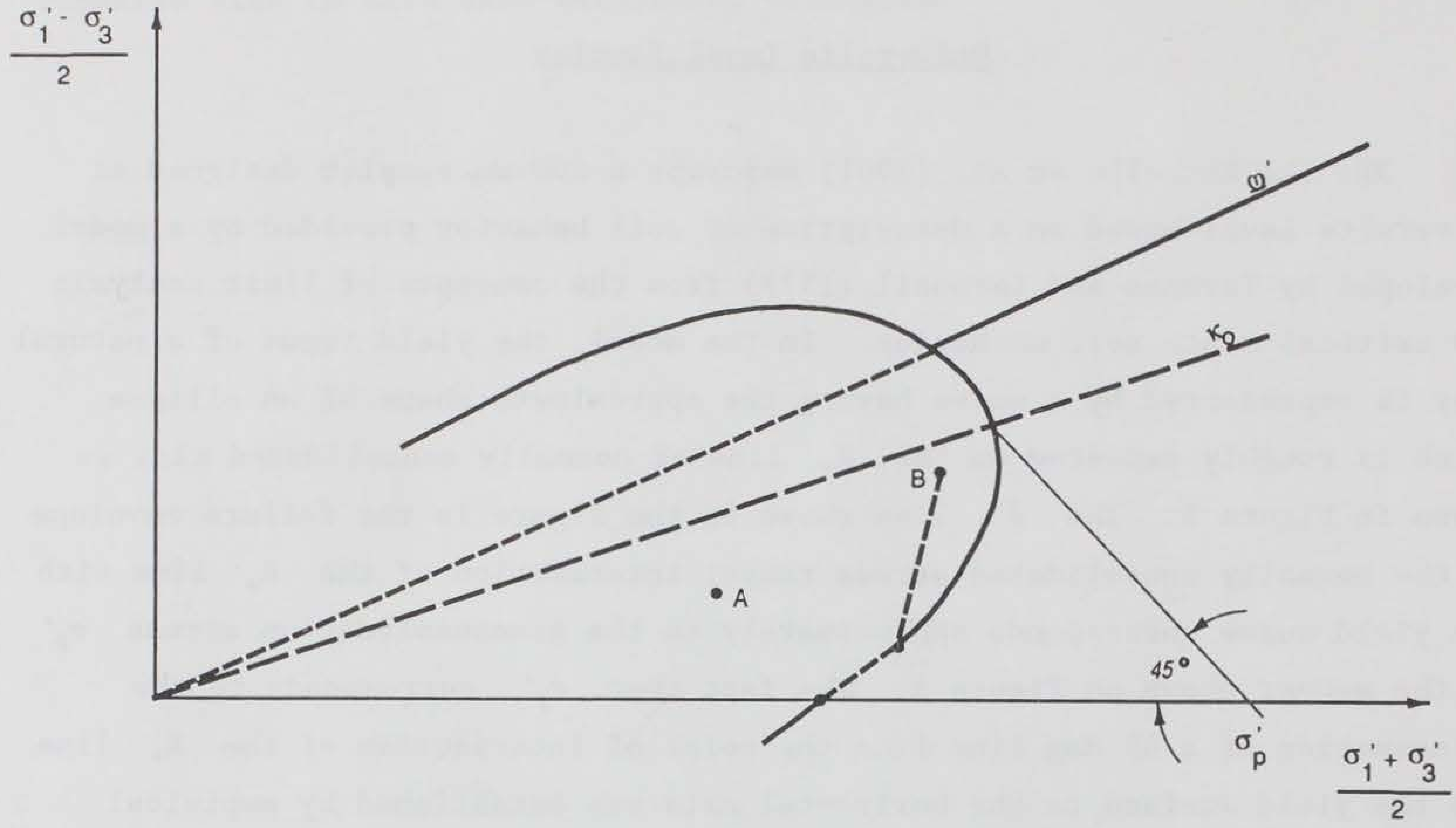


Figure 5. Typical limit state curve for natural clays (after La Rochelle et al. 1981)

though the technique of sampling is otherwise "perfect." Analysis showed, in fact, that it can be virtually impossible to sample lightly overconsolidated clay without substantial disturbance because the in situ stresses may be such that the stress path cannot avoid touching the limit state curve as the in situ stresses are released, as in the case of point B in Figure 5.

Release of In Situ Stresses

34. "Perfect sampling," La Rochelle et al. (1981) suggest, is impossible even in block samples because as block samples are carefully and gradually carved out, stresses are released nonuniformly and a thin layer of clay is remolded at the surface of the sample. Surface remolding releases negative pore water pressure which causes water migration toward the center of the specimen. With time, water migration to the center of the specimen causes swelling and completely alters the characteristics of the clay there (at the center) as the effective stress path moves to zero. Bjerrum (1973) has observed water content differences of 3 to 4 percent over the cross section of tubes of Norwegian clays; the influence of pore water migration was further demonstrated by the fact that specimens trimmed and tested in the field

immediately after sampling yielded higher compressive strengths than those of similar specimens tested three days after sampling. However, an investigation by La Rochelle et al. (1976) on Canadian clay did not show pore water migration or any consistent differences in compressive strength immediately after sampling in the field or after a few days in the laboratory.

Mechanical Distortion

35. Volume change from intruding a sampling tube into a soil mass is known to produce distortions in clay samples. Sampling tubes and techniques have been refined in the attempt to improve the quality of soil samples, and satisfactory results are considered to be achieved if tubes of good design are used along with careful sampling technique. However, disturbance in sampling due to mechanical distortion is acknowledged and arguments have been advanced that even the best samplers can produce enough distortion to remold and alter the mechanical properties of a large part of a clay sample. The total stress-time sequences which occur during the intrusion of a sampling tube and extraction from the ground are so complex and poorly understood that a limit/critical state analysis of the events to determine the extent of the damage is impossible. However various research cited by La Rochelle et al. (1981) indicates that the maximum difference between peak strengths determined (by isotropically consolidated drained and undrained triaxial tests) on block and 54-mm tube samples of Champlain clay is 30 percent. Additionally, Young's tangent modulus (at 50 percent of peak strength) was reduced on average by about 50 percent and oedometer recompression indices were doubled as the result of tube sampling. For example, the effect of disturbance is shown very strikingly by La Rochelle and Lefebvre (1971) in stress-strain curves determined by performing unconfined compression tests on tube and block samples of Champlain clay as shown in Figure 6. The curves show the dramatic reduction in Young's modulus and peak compression stress as the result of sampling disturbance in the tube sample.

36. La Rochelle et al. (1981) present an argument suggesting that the allowance of internal clearance in a sampler produces distortion and disturbance during tube withdrawal after sampling and conclude that internal clearance should be eliminated from sampling tubes (inside clearance ratio should be made zero). The argument for eliminating clearance ratio is that the area ratio of a tube is effectively increased by the existence of an inside

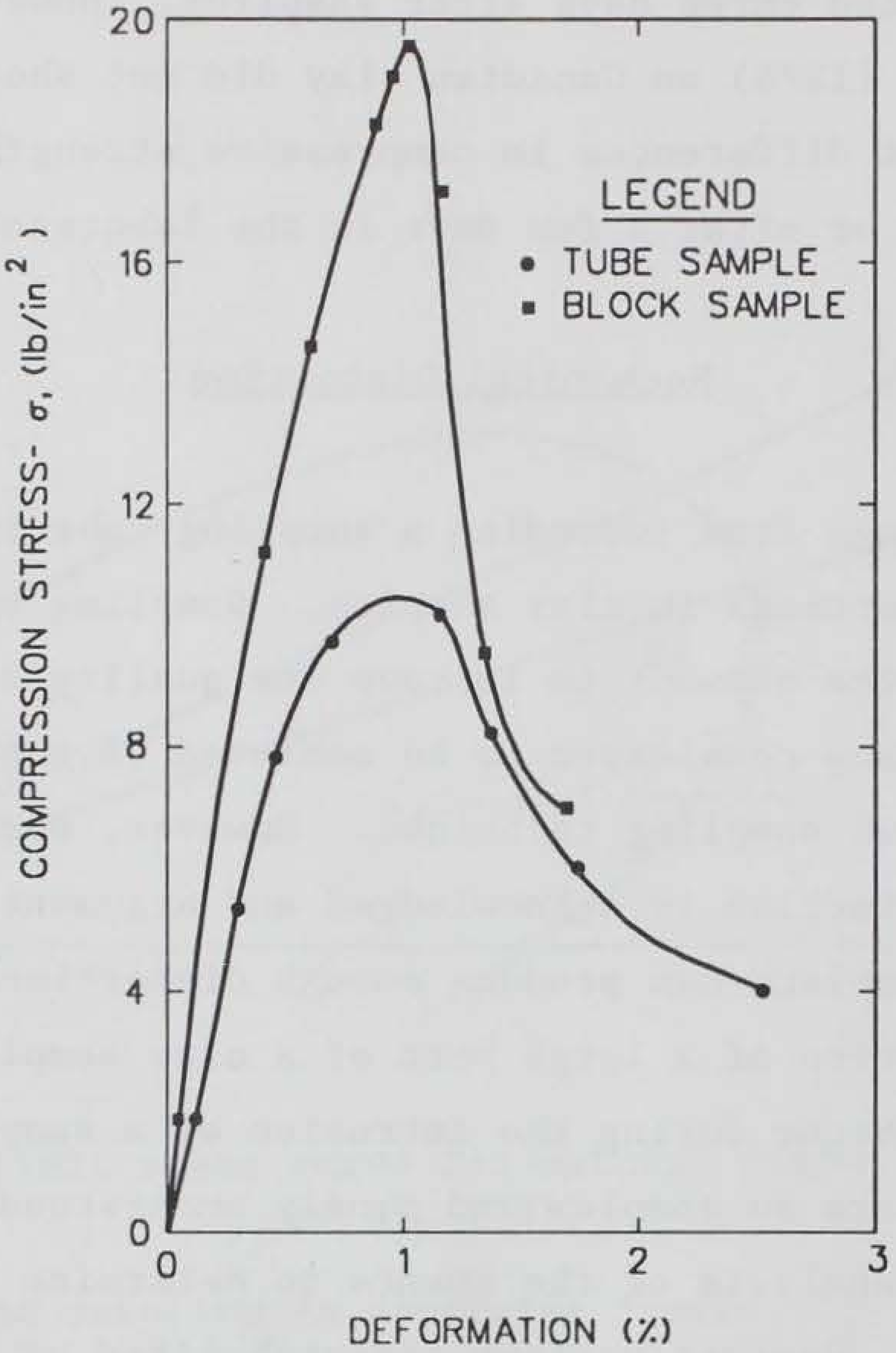


Figure 6. Examples of stress-strain curves from unconfined compression tests on block samples of Champlain clay (after La Rochelle and Lefebvre 1971)

clearance ratio. Additionally, a clearance ratio allows and even forces lateral expansion of a sample within the tube because suction (vacuum) develops in the clearance space due to the tightness of the piston inside the tube. Therefore volume change and distortion will occur as the result of soil intruding into this space. Since this effect is believed to be one of the main causes of sampling disturbance, the Laval researchers decided to eliminate inside clearance as well as the piston since use of a piston tends to produce suction. Suction produced by any operation, the Laval team concluded, is very damaging in terms of sampling disturbance and should be avoided. Friction which would result in shear stress and distortion between the tube and soil is not believed to be significant since the sensitive clays investigated in this study were thought to be self-lubricating as the result of remolding at the interface between soil and tube. Data are presented in

Table 3 from three sites and three tube sizes to demonstrate the advantage of eliminating internal clearance. Reference to "reshaped" tubes in the table indicates that inside clearance has been eliminated; the ratio c_{uf}/c_{uv} is the ratio of undrained strength determined by unconfined compression tests to that determined in the field vane test. Tubes designated as "standard" in the table are tubes provided with an inside clearance ratio. Tangent modulus is considered to be a good indicator of sample quality and is presented in the table. The tangent modulus, E_u, presented in Table 3 is the Young's modulus determined at 50 percent of the peak stress. The table shows that elimination of inside clearance increased the strength ratio and tangent modulus values by an average of about 50 and 75 percent, respectively. Additionally it is shown that as the size of the tube increases, the quality indicators generally show improvement in sample quality, suggesting that increasing sample size decreases sample disturbance.

Table 3

Comparative Results of Quality Tests

Site	Type of Tube	c_{uf}/c_{uv}	E_u (kPa)
Saint-Vallier (from 7.9 m)	54 mm standard	0.32	1,800
	54 mm reshaped	0.66	3,500
	75 mm reshaped	0.81	4,100
	100 mm reshaped	0.80	5,200
Saint-Vallier (from 14.0 m)	54 mm standard	0.75	4,100
	54 mm reshaped	0.90	7,600
	75 mm reshaped	0.88	5,900
	100 mm reshaped	0.84	9,000
Yamaska (from 6.1 m)	54 mm standard	1.02	5,500
	54 mm reshaped	1.24	8,300
	75 mm reshaped	1.11	7,200
	100 mm reshaped	1.57	9,000

(After La Rochelle et al. 1981)

200-mm Sampler

37. Based on observations and consideration of previous work in sampling sensitive Canadian clays, La Rochelle et al. (1981) determined to design a large diameter sampler on the following principles:

a. The tube should have no inside clearance.

b. The internal diameter of the tube should be precisely machined to meet strict tolerance with respect to roundness.

c. The cutting edge should be very sharp and shaped to force displaced material toward the outside of the tube.

d. The piston should be eliminated.

e. Suction or negative stress is very damaging to a sample and should be avoided at all stages of sampling. This is done by eliminating the use of a piston in the initial stage of sampling and by overcoring around the sampling tube in the final stage.

f. The sample diameter should be large enough to reduce the relative amount of disturbed material around the intact core. The investigators determined that 200 mm was a sufficiently large tube diameter by showing that the yield curves were essentially the same for block samples and 200-mm tube samples.

38. The results of laboratory tests on block samples obtained by Lefebvre (1970) and La Rochelle and Lefebvre (1971) with laboratory tests performed on 200-mm tube samples in the investigation by La Rochelle et al. (1981) are compared in Table 4.

Table 4

Comparative Results of Undrained Compression Tests

	Block	20-mm tube
	Unconfined Compression tests	
Number of tests	9	7
c_{uf} (kPa)	65	60
ϵ_f (%)	1.06	1.03
	Unconsolidated Undrained Compression Tests	
Number of tests	11	6
c_{uf} (kPa)	61.5	62.75
ϵ_f (%)	1.07	1.02

As can be seen from the table, undrained strength and strain level at maximum stress are virtually identical for block and 200-mm tube samples in unconfined compression as well as unconsolidated undrained compression tests. Because of the favorable comparison of mechanical properties with those measured in block samples, the 200-mm Laval sampler is offered by La Rochelle et al. (1981) as a cost-effective alternative to block samples in investigations for projects requiring elaborate laboratory testing. However, they acknowledge that the use of this large sampler is not economically feasible for routine investigations.

PART VI: WORK BY LACASSE, BERRE, AND LEFEBVRE

39. Lacasse, Berre, and Lefebvre (1985) compared the quality of 300-mm-diameter-block samples with 95-mm-fixed-piston samples of three Norwegian marine clays. Two of the clays were quick and the remaining clay was sensitive. Oedometer tests, unconfined compression tests, triaxial compression tests, and direct simple shear tests were performed on the three clays.

40. Unconfined compression tests performed on one of the sensitive clays clearly showed a much higher peak strength and lower failure strain in the block samples than those in the 95-mm tube samples. A factor of 2 between the ratio of peak strength of block to tube sample and 4 with respect to percent strain at failure (between block and tube) was not an unusual finding in this investigation. The sensitive clay was quite brittle in that samples from the block reached peak shear stress at 0.5 percent strain in unconfined compression.

41. Triaxial compression and extension tests were performed on specimens which were anisotropically consolidated to the in situ effective stresses. Undrained shear strengths on block samples were 10 to 33 percent higher than those on specimens from the 95-mm blocks. Young's modulus at 50 percent of the peak shear stress was larger by a factor of 4 in some block specimens of quick clay (relative to tube specimens). Smaller differences were observed in the nonquick clay. Differences were not large between block and tube specimens in triaxial extension tests.

42. A significant difference was observed in only one direct simple shear test. In that test the horizontal shear stress was 50 percent higher in the block sample relative to the tube sample. However, the fact that the stress system and stress and strain concentrations are very different in the simple shear test from those in the triaxial compression test was pointed out by Lacasse, Berre, and Lefebvre (1985).

43. The authors conclude that quick clays tend to lose strength and resistance to deformation as the result of disturbance, but such was not the case for nonquick more plastic clay tested. The effect of disturbance varies with the type of test. Disturbance effects are smallest in tests where confining pressure is greatest; results were least affected in the oedometer test with the triaxial test intermediate and the greatest influence (of sampling disturbance) observed in the unconfined compression test. The authors state that reconsolidation appears to correct for a large amount of sample

disturbance, as determined by comparing unconfined compression tests with undrained triaxial compression tests. However, even after reconsolidation, Young's modulus of tube samples remained significantly lower than that of block samples. It appears that once the natural structure of a soil is damaged by sampling, the original structure and mechanical characteristics are not recoverable.

PART VII: OEDOMETER TESTS ON COHESIVE SOILS

Background

44. Consolidation or oedometer tests are generally performed on "undisturbed" specimens of clay to characterize the material in terms of its compressibility and, when applied pressure is released, rebound characteristics. However, material characteristics measured in the oedometer test are affected by sampling disturbance. Since the aim of oedometer testing and the associated foundation analysis is to enable accurate determination of consolidation settlement, attention must be paid to procedures that allow reconstruction of the in situ void-ratio/effective stress relationship. Figure 7 represents a laboratory void ratio-effective stress relationship for one-dimensional compression. Schmertmann (1955) devised a method for correcting the laboratory compression curve to determine the field virgin compression curve. The procedure will be described briefly for completeness because the Schmertmann procedure addresses and, to an extent, quantifies the effects of sample disturbance on soil compressibility.

Schmertmann Reconstruction Procedure

45. Schmertmann (1955) determined that the effect of disturbance in samples of cohesive soil is diminished by reconsolidation. Based on the study of many laboratory compression tests, he found that the laboratory compression curve intersects the field virgin compression curve at approximately $0.42e_o$, where e_o is the initial void ratio of the soil under test as removed from the sampling tube. Referring to Figure 8, the procedure to determine the field virgin compression curve from disturbed samples of normally consolidated soils consists of the following steps:

- a. Point B is determined by the Casagrande graphical method as the preconsolidation pressure, which (in the case of normally consolidated soils) is also the in situ overburden pressure.
- b. Point C is the intersection of a horizontal line through $0.42e_o$ with the laboratory virgin compression curve.
- c. Point D is determined by the intersection of a horizontal line through e_o intersecting a vertical line through point B, the preconsolidation/in situ pressure.
- d. The line CD is the field virgin compression curve.

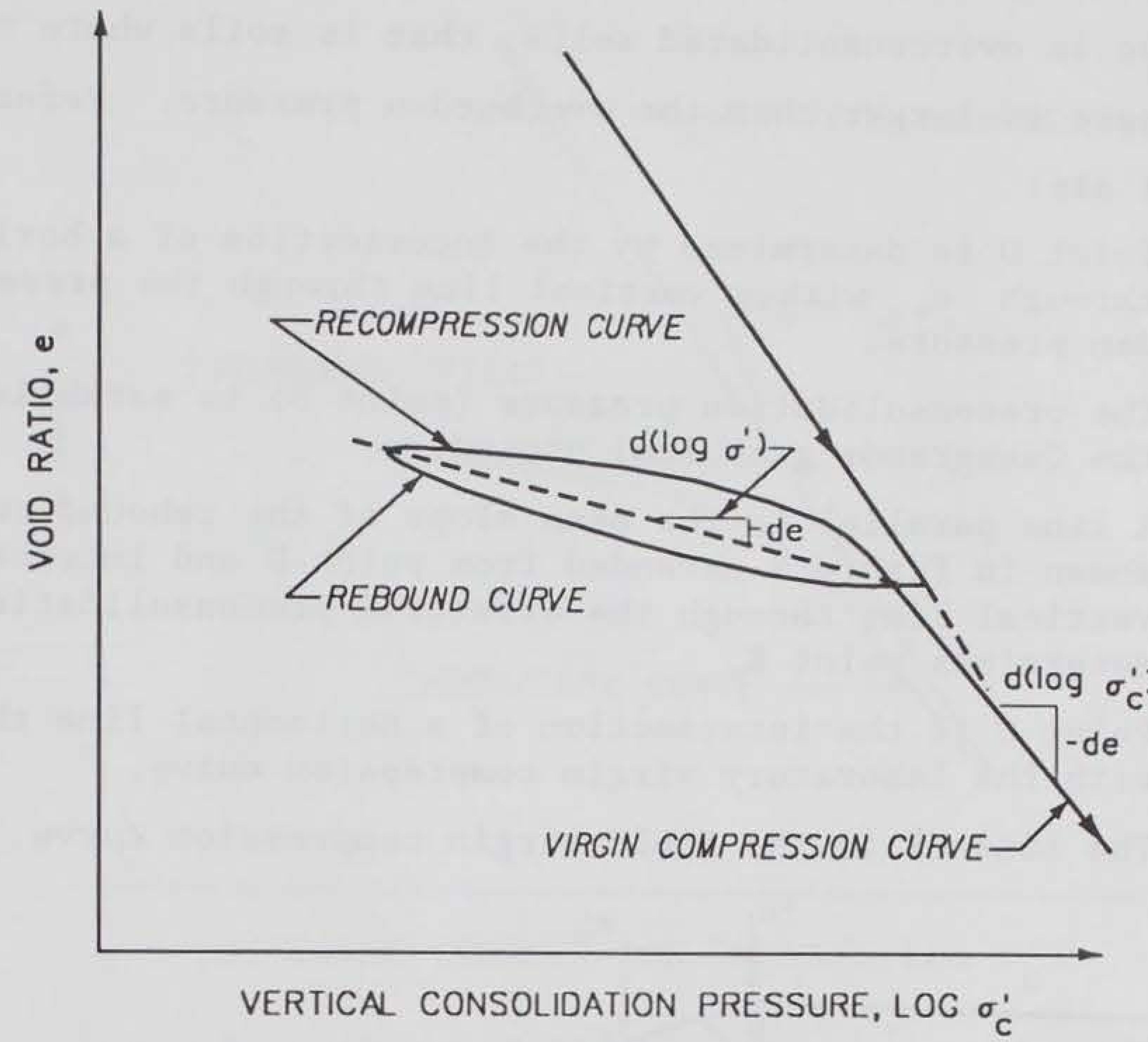


Figure 7. Laboratory void-ratio-effective-stress curve for one-dimensional compression

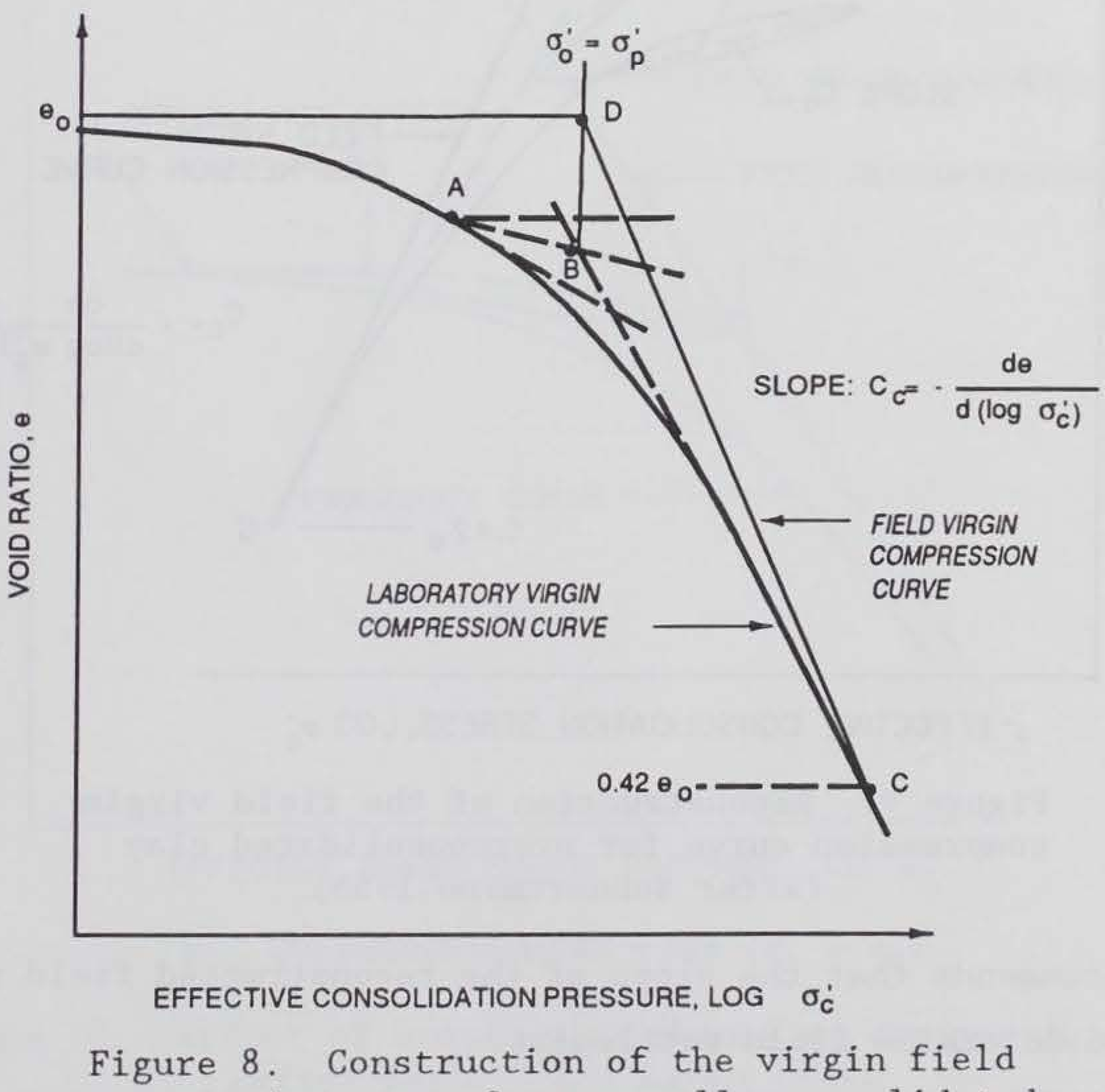


Figure 8. Construction of the virgin field compression curve from normally consolidated cohesive soil (after Schmertmann 1955)

46. The procedure is slightly different to determine the field virgin compression curve in overconsolidated soils, that is soils where the preconsolidation pressure is larger than the overburden pressure. Referring to Figure 9, the steps are:

a. Point D is determined by the intersection of a horizontal line through e_o with a vertical line through the present overburden pressure.

b. The preconsolidation pressure (point B) is established using the Casagrande graphical procedure.

c. A line parallel to the mean slope of the rebound curve, as shown in Figure 9 extended from point D and intersecting a vertical line through the effective preconsolidation pressure determines point E.

d. Point C is the intersection of a horizontal line through $0.42e_o$ with the laboratory virgin compression curve.

e. The line CE is the field virgin compression curve.

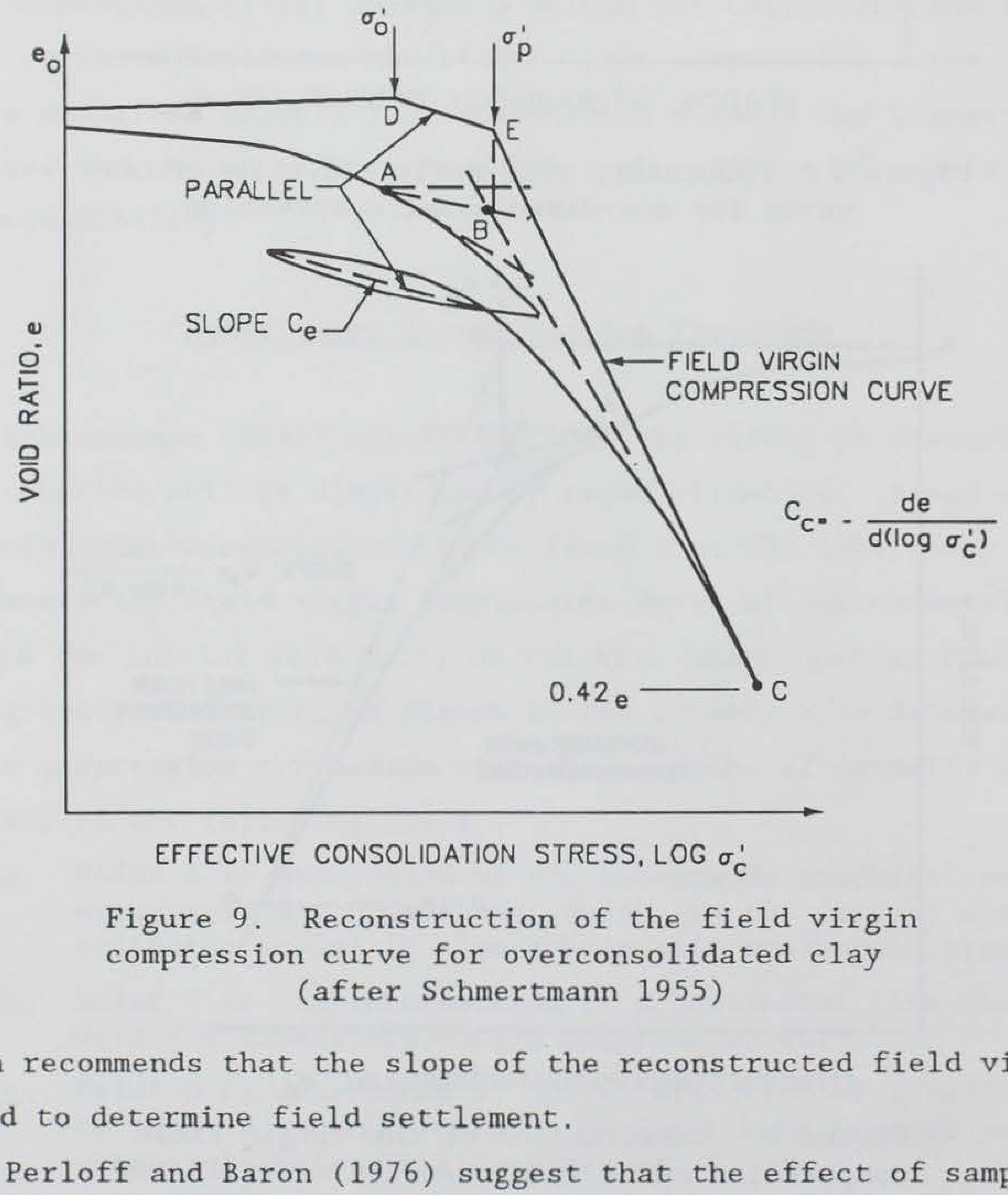
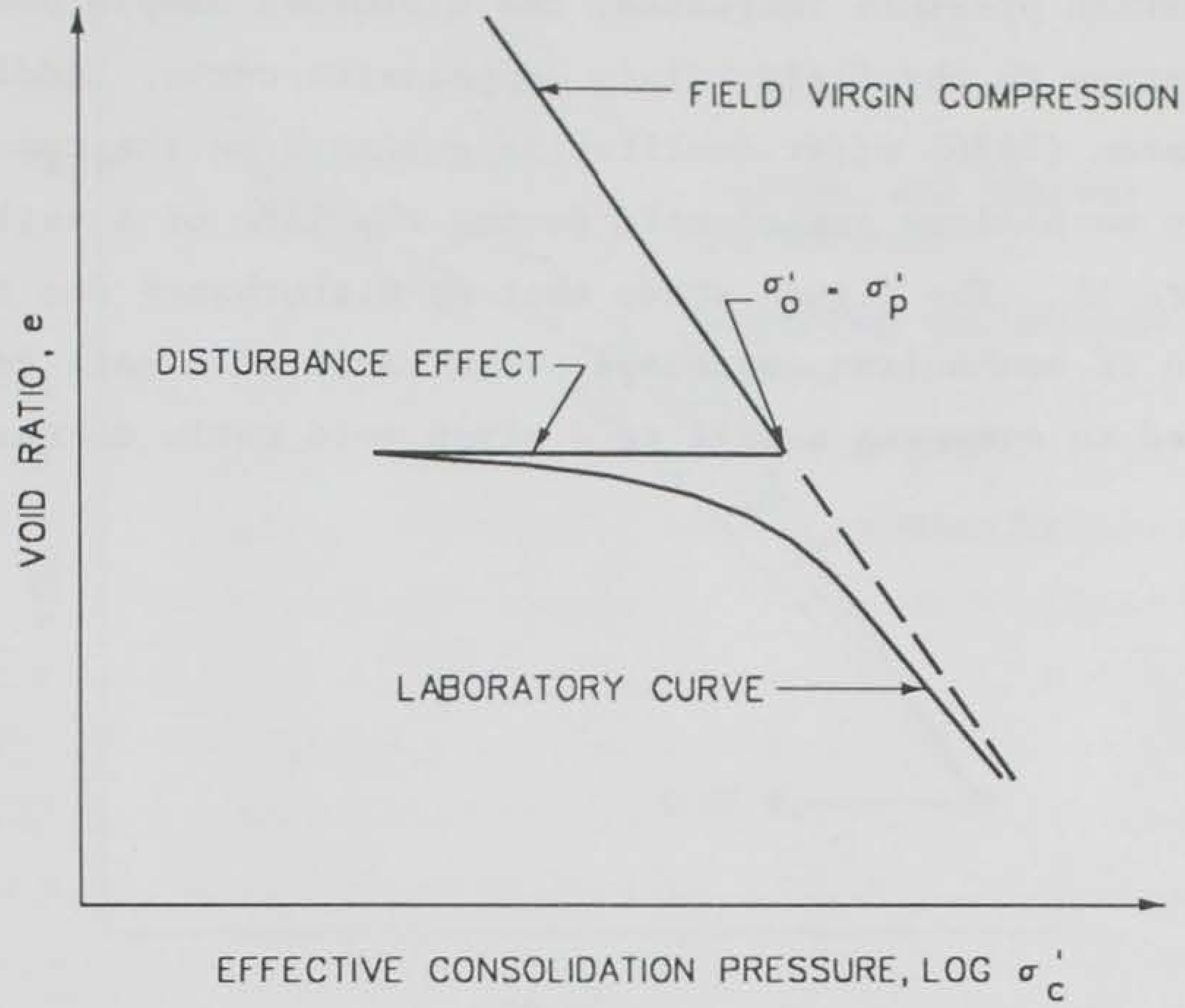


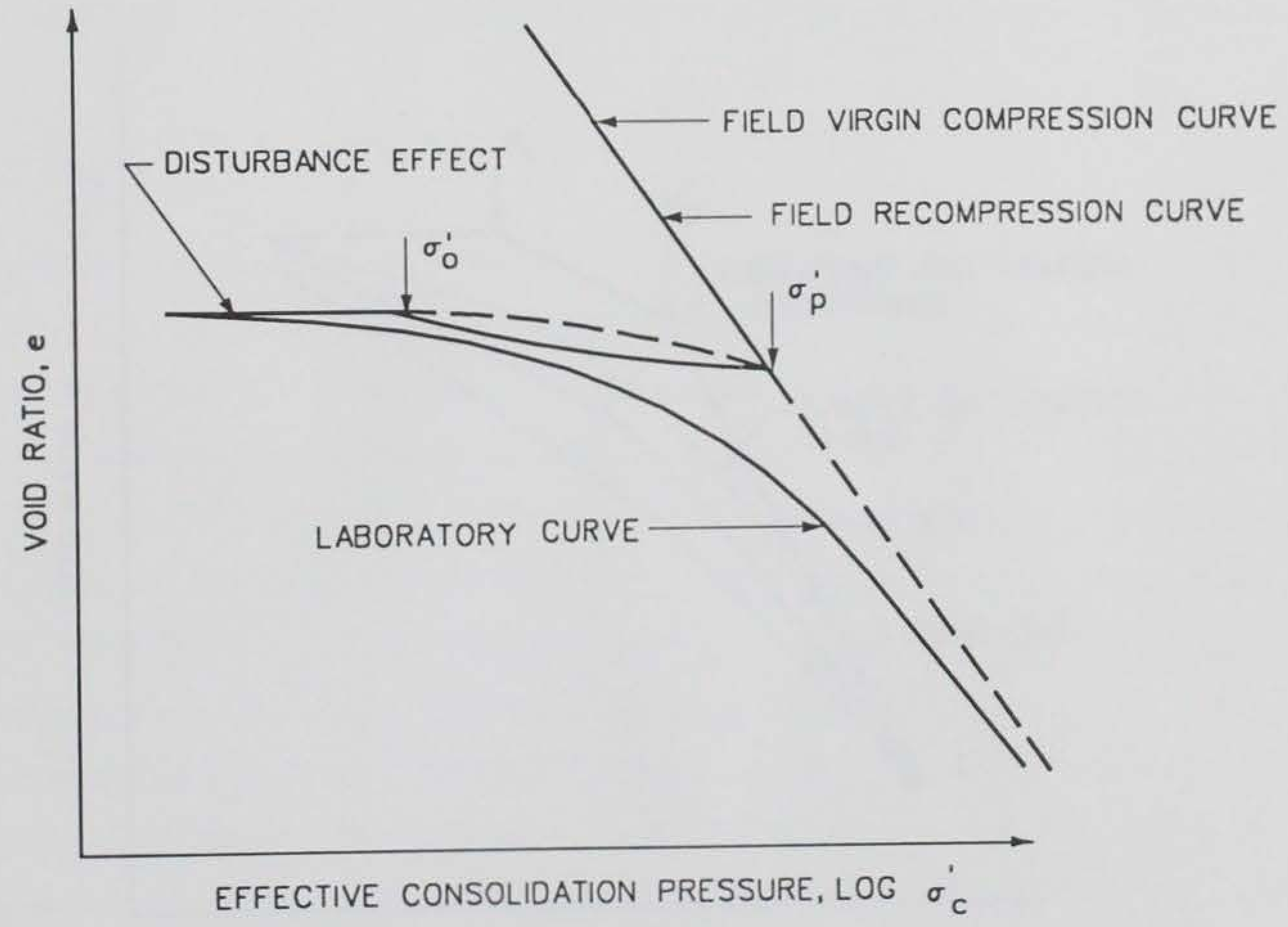
Figure 9. Reconstruction of the field virgin compression curve for overconsolidated clay (after Schmertmann 1955)

Schmertmann recommends that the slope of the reconstructed field virgin curve, C_c , be used to determine field settlement.

47. Perloff and Baron (1976) suggest that the effect of sample disturbance on compressibility is shown in Figure 10 for normally consolidated as



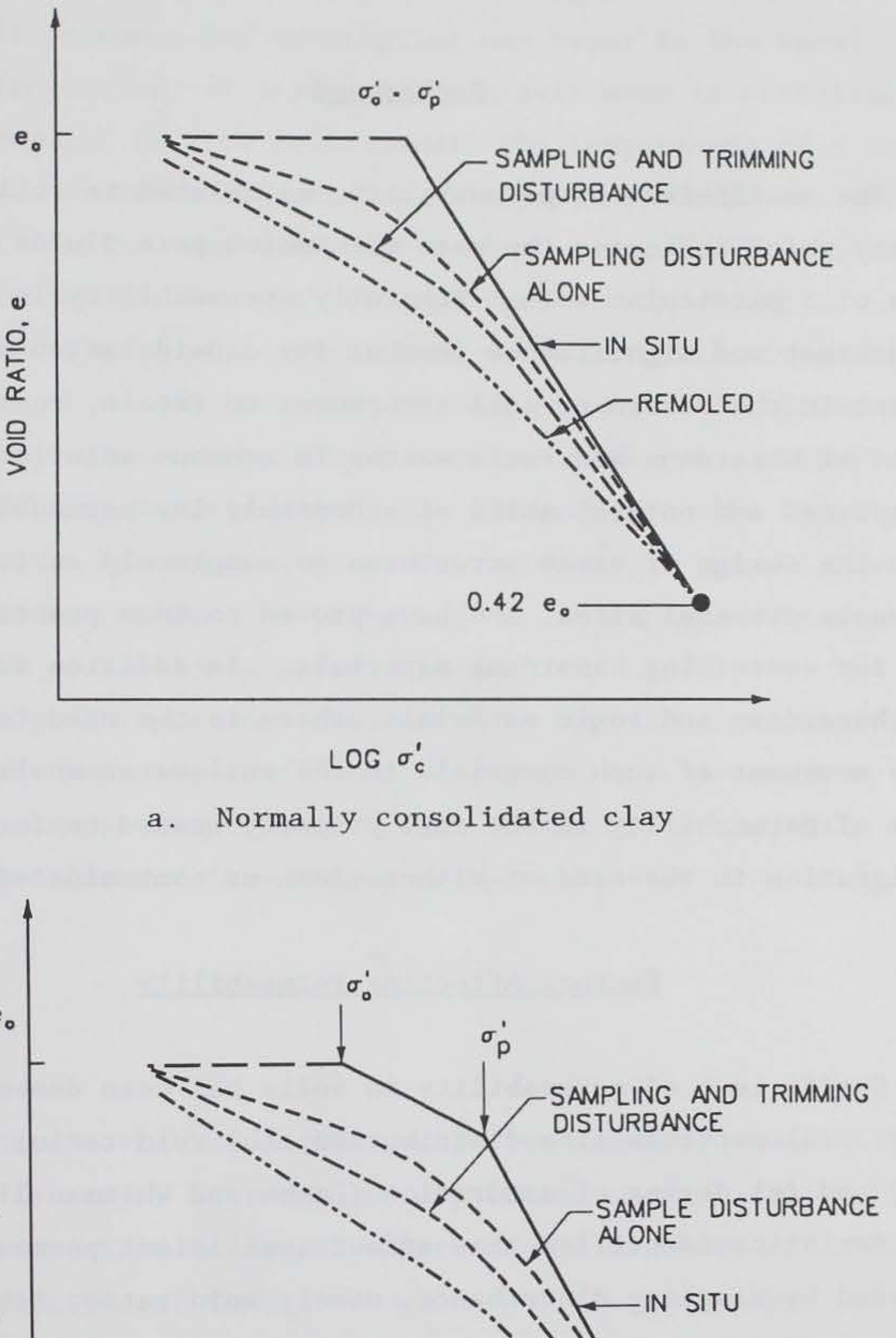
a. Normally consolidated clay ($\sigma'_p = \sigma'_o$)



b. Overconsolidated clay ($\sigma'_p > \sigma'_o$)

Figure 10. Effect of sample disturbance on compressibility (after Perloff and Baron 1976)

well as overconsolidated soils. These relationships also infer that as effective consolidation pressure increases, the disturbed sample compression relationship converges to the field virgin compression curve. Additionally, Perloff and Baron (1976) offer qualitative guidance on the cumulative effects of disturbance mechanisms experienced during the life of a soil sample as shown in Figure 11. The figure shows that as disturbance due to any mechanism or combination of mechanisms increases, void ratio of a soil decreases and the stress required to compress a soil to a given void ratio decreases.



a. Normally consolidated clay

b. Overconsolidated clay

Figure 11. Effect of sampling and specimen preparation on the laboratory e-log σ_c' curve (after Perloff and Baron 1976)

PART VIII: PERMEABILITY

Background

48. The coefficient of permeability, as related to soil mechanics, is that property which indicates the ease with which pore fluids move through the interstices of a particular soil. Presently, permeability is a soil property of great interest and significance in that its consideration and manipulation are important in the design of soil structures to retain, contain, and control the movement of hazardous and toxic wastes in aqueous solution in ground water. Compacted and natural soils of acceptably low permeability are employed in the design of earth structures to completely surround/envelope hazardous waste disposal sites, and have proved to be a practical and effective means for containing hazardous materials. In addition to measures for containing hazardous and toxic materials, there is the need to monitor and predict the movement of such materials in the soil-water environment. Coefficient of permeability is the soil property needed to forecast soil-water flow and migration in the case of either clean or contaminated ground water.

Factors Affecting Permeability

49. Coefficient of permeability in soils has been demonstrated to be affected by: (a) particle size distribution; (b) void ratio; (c) composition; (d) fabric; and (e) degree of saturation (Lambe and Whitman 1969). Of the five characteristics identified that affect coefficient permeability, three may be altered by sampling disturbance, namely void ratio, fabric, and degree of saturation. Even in the case of soils which are sampled with a "perfect sampling" technique, stresses are released which may cause remolding, void ratio changes, and pore water migration within the sample (Bjerrum 1973, La Rochelle et al. 1981). Such changes in the soil sample not only affect the average void ratio, as indicated by the redistribution of water content in the specimen (Bjerrum 1973), but the structure and fabric of the subject clay are changed as well. Removing a soil from its in situ ground-water environment can result in gases which are dissolved in pore fluid under pressure coming out of solution as a specimen is brought to the surface and fluid pressure is released. Whether partially or completely water-saturated, degree of saturation of a soil sample will decrease as the result of dissolution of pore water

and dissolved gas during and after soil sampling (Okumura 1971). It must also be noted that disturbance due to sampling can occur as the result of temperature change. Temperature at depth within a soil mass is essentially constant. Once removed from its in situ environment, the temperature of a soil sample is subject to increase and to fluctuation. Temperature increase, for example, can cause volatilization and dissolution of pore gases within a soil sample. Because gases in the small voids of a soil/water mass are held there tightly enough by surface tension forces that the voids are effectively blocked, the effective shape, size, and character of void space in a soil through which water flow can occur are changed by the presence of gas. Therefore coefficient of permeability as measured in the laboratory can be significantly changed from the in situ value because gas content of the soil voids and, hence, degree of saturation are influenced by sampling as the result of temperature as well as (fluid) pressure changes.

50. Harr (1987) presents (see Table 5) typical coefficients of variation for various laboratory and field determined soil properties, including coefficient of permeability. Coefficient of variation, V , is a measure of scatter or dispersion and is mathematically defined as

$$V = S/A \tag{10}$$

where

S = standard deviation of the population under consideration

A = mean (average) value of the population under consideration

Coefficient of variation is usually expressed in percent.

51. Examination of Table 5 reveals that the coefficient of variation is greater for coefficient of permeability than in any other laboratory (or field) measured parameter. Additionally, it is seen from examination of data presented in the table that coefficient of variation (scatter) in values of coefficient of permeability significantly increases as degree of saturation decreases. Since coefficient of permeability is affected by several soil parameters as well as environmental conditions, all of which are subject to be affected by sampling disturbance, it is not surprising that variation in coefficient of permeability is greater than that of other listed soil parameters.

52. Okumura (1971) presents data from laboratory tests on a plastic (LL = 93 percent, PI = 54 percent) marine clay which show that as disturbance increases, laboratory-measured coefficient of permeability decreases. Data

Table 5

Representative Coefficients of Variation (After Harr 1987)

Parameter	Coefficient of Variation, %	Source
Porosity	10	Schultze (1972)
Specific gravity	2	Padilla and Vanmarcke (1974)
Water content		
Silty clay	20	Padilla and Vanmarcke (1974)
Clay	13	Fredlund and Dahlman (1972)
Degree of saturation	10	Fredlund and Dahlman (1972)
Unit weight	3	Hammitt (1966)
Coefficient of permeability	(240 at 80% saturation to 90 at 100% saturation)	Nielsen et al. (1973)
Compressibility factor	16	Padilla and Vanmarcke (1974)
Preconsolidation pressure	19	Padilla and Vanmarcke (1974)
Compression index		
Sandy clay	26	Lumb (1966)
Clay	30	Fredlund and Dahlman (1972)
Standard penetration test	26	Schultze (1975)
Standard cone test	37	Schultze (1975)
Friction angle ϕ		
Gravel	7	Schultze (1972)
Sand	12	Schultze (1972)
c, strength parameter (cohesion)	40	Fredlund and Dahlman (1972)

presented by Okumura show, generally, that as effective vertical pressure increases, differences between coefficients of permeability in disturbed and undisturbed specimens become smaller.

53. Several aspects of the measurement of coefficient of permeability are disturbing, especially in light of the significance of this parameter in the design of landfills for the containment of hazardous and toxic wastes:

- a. Greater sampling disturbance appears to produce lower (less conservative) laboratory-measured values of coefficient of permeability.
- b. Variation and uncertainty in laboratory-measured values of coefficient of permeability are substantial.
- c. Degree of saturation in soil samples is unavoidably and, perhaps, irreversibly disturbed by removal from the in situ fluid pressure and temperature environment.
- d. The influence of degree of saturation on scatter in measured values of coefficient of permeability is poorly understood.
- e. The influence of the effect of structural disturbance on measurement of the coefficient of permeability in soil is poorly understood.

PART IX: SUMMARY

Factors Affecting Disturbance

54. Soil sampling for the purpose of determining in situ characteristics is a case where the presence of an observer changes the nature of the experiment. Taking a soil out of its in situ environment, no matter how carefully executed, cannot be done without releasing in situ stresses, disturbing the material, and altering its mechanical properties. Soil properties are irreversibly changed to a greater or lesser extent by sampling. The amount of disturbance occurring is determined by:

- a. Soil type, whether cohesive or granular.
- b. Material characteristics such as density, sensitivity, water content, overconsolidation ratio, etc.
- c. Size of specimen.
- d. Sample type (block samples or tube samples).
- e. Geometric design of the sampler and sampling tube used.
- f. Sampling depth.
- g. Rate of sample advance.
- h. Method used to advance, stabilize, and clean the borehole.

55. Each soil type, whether predominantly cohesive or granular, has its own particular problems with respect to sampling disturbance. Clays/cohesive soils experience structural disturbance as the result of sampling, and the susceptibility to disturbance increases as material sensitivity increases. Some investigators have suggested that strength in sensitive clays is a time dependent function of a process involving progressive stress changes and water content distribution. Therefore, strength measurement is a function of the time lapse since sampling. Other investigators could not confirm this time dependency.

56. Granular materials can experience density changes during sampling which affect laboratory strength determination. The consensus of all investigators addressing the topic is that (all other factors equal) the larger a sample, the smaller the disturbance it will suffer from sampling. It was also the consensus that block samples which are hand cut suffer less disturbance than tube samples; however, block samples can be taken only by excavating down to the desired level of interest. This can be expensive and difficult, especially if the desired level is below the water table.

57. Geometric design and mechanical configuration of a perfect sampling tube is not a matter on which all experts agree. All investigators reviewed for this report agree that the area ratio as defined by Hvorslev (1949) should be kept as small as possible, typically between 10 and 15 percent. There was also agreement on the need to keep the tube as strong and round (perfectly cylindrical) as possible, but this is difficult to do if the section of the tube (which is a direct function of area ratio) becomes too small. There is disagreement among investigators reviewed concerning the value of inside clearance ratio (also defined by Hvorslev 1949). Some investigators contend, and present data to support their contention, that inside tube relief from the provision of a nonzero inside clearance ratio allows and aggravates material disturbance during sampling. Additionally, there is disagreement regarding the value of the piston sampler. Terzaghi and Peck (1968) and Hvorslev (1949) present arguments to show that the use of a thin-walled sampler with a fixed piston minimizes internal movement of soil inside the tube during sampling and therefore (minimizes) sampling disturbance. However, other investigators (La Rochelle et al. 1981) present an argument that a fixed piston produces vacuum within the sampling tube which causes substantial sampling disturbance. The opinion of the Laval research team that vacuum/suction is very damaging to a soil sample is shared by researchers at NGI (Andresen and Kolstad 1979).

58. Finally, research indicates that sampling depth influences disturbance as the result of sampling. The greater the depth of sampling, the greater the probability of sampling disturbance. Logic would support this inference simply because the difficulty and effort involved in performing all activities associated with sampling increase as depth increases.

Sources of Disturbance

59. Several mechanisms identified above result in disturbance during soil sampling. The most important of these mechanisms may be:

- a. Compression, extension, shear, and vibration due to the intrusion of the sampling tube.
- b. Tension and shear as the tube is extracted.
- c. The release of fluid and earth pressure as the result of removing the soil from its in situ stress environment.
- d. Changes in pore structure and state of saturation which result from removing the soil from its in situ temperature environment.

e. Shock and structural disturbance during transportation and storage.

f. Compression, extension, shear, and vibration during extrusion and trimming operations.

Properties Change Due to Sampling Disturbance

60. The primary purpose in acquiring undisturbed samples from a soil mass is to perform laboratory tests to determine specific mechanical properties which are characteristic of the mass. Properties of interest include stress-strain, strength, compressibility, and permeability characteristics. The influence of sampling disturbance in the case of most of these characteristics is to cause deviations in measured behavior which are unconservative.

61. Research reviewed for this report indicates that stress-strain characteristics measured on samples of "undisturbed" clay soil with varying amounts of disturbance yield strain levels at maximum shear stress which can be several times greater than those of the highest quality sample. Young's modulus is less in such disturbed samples by a factor of up to 5. Undrained shear strength also increases as disturbance in soil samples decreases. Depending on the sensitivity and degree of disturbance in the clay involved, undrained shear strength can be 50 percent less in disturbed tube samples. Unlike other measured soil properties, the effect of disturbance on undrained strength is to make a design based on use of a laboratory-measured value more conservative. However, caution must be used because the effect of disturbance in loose sands is to densify the sand, increase the laboratory-measured strength, and render a design based on this laboratory-measured value less conservative.

62. Compressibility in disturbed tube samples as measured in laboratory tests decreases as sample disturbance increases. Additionally the slope of the drained compression curve measured in the oedometer test (C_c) decreases with sample disturbance (making analyses and designs based on laboratory-measured values less conservative). A procedure for determining the in situ virgin/undisturbed drained compression curve (C_c) has been devised by Schmertmann (1955). The reconstruction procedure is uncomplicated and is based on the notion that compression of soil under successively higher pressures erases/removes the effects of sampling disturbance. Recent research confirms Schmertmann's early assertion that compression of soil to high

pressures decreases the effects of sampling disturbance. Ladd and Foote (1974) and Baligh, Azzouz, and Chin (1987) determined that compression diminished the influence of sampling disturbance on undrained shear strength as measured in the triaxial test; however, it was acknowledged that no amount of compression will restore disturbed soils to their pristine states of structure and mechanical behavior. Schmertmann* suggests that the way to avoid sampling disturbance to the maximum possible extent is to perform in situ tests, although some disturbance occurs in preparing to perform in situ tests, and in situ tests must be interpreted properly for use in design.

63. Coefficient of permeability is a soil property which is substantially influenced by other properties. Therefore it is not surprising that considerable scatter and uncertainty are normally found in the laboratory measurement of this property. Coefficient of permeability is affected by disturbance because strength, density, (soil) structure, and degree of saturation are affected by disturbance during sampling, storage, and sample preparation. Change in degree of saturation as the result of sampling disturbance, particularly, influences laboratory-measured coefficient of permeability because the area through which flow can occur in a soil specimen is irreversibly and unpredictably changed by changes in degree of saturation, which may be caused by changes in temperature as well as changes in fluid pressure. The NGI considers disturbance effects due to changes in temperature important enough to require that soil samples be stored in a temperature-controlled environment where temperature is maintained at the average ground temperature around Oslo, Norway. Limited research on how disturbance affects coefficient of permeability suggests that coefficient of permeability is decreased by disturbance. This result leads to unconservative estimates of performance if the ability of a soil cover or liner is being evaluated for its ability to retard water flow. However, what often occurs is that steps are taken to saturate a permeability test specimen in the laboratory. Complete water saturation results in a limiting worst-case condition with respect to permeability in that saturation will permit flow through all the void space in a soil specimen. However, disturbance in attaining saturation results in a change in the void structure of a soil. The combined effect of disturbance and

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* Personal communication (11 March 1992).

saturation may, to an extent, be compensating, but the net influence on the true coefficient of permeability is unknown.

REFERENCES

Andresen, A. AA., and Kolstad, P. 1979. "The NGI 54-mm Sampler for Undisturbed Sampling of Clays and Representative Sampling of Coarser Materials," Proceedings of the International Symposium of Soil Sampling, Singapore, pp 13-21.

Baligh, M. M. 1975 (Sep). "Theory of Deep Site Static Cone Penetration Resistance," Research Report No. R75-56, Order No. 517, Dept. of Civil Engineering, MIT, Cambridge, MA, 133 pp.

__________. 1985. "Strain Path Method," Journal of Geotechnical Engineering, ASCE, Vol 111, No. 9, pp 1108-1136.

Baligh, M. M., Azzouz, A. S., and Chin, C. T. 1987. "Disturbances Due to 'Ideal' Tube Sampling," Journal of Geotechnical Engineering, ASCE, Vol 113, No. 7, pp 739-757.

Bjerrum, L. 1973. "Problems of Soil Mechanics and Construction of Soft Clays, State-of-the-Art Report to Session 4," 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol 3, pp 111-159.

Bowles, J. E. 1974. Analytical and Computer Methods in Foundation Engineering, McGraw-Hill Book Company, New York, pp 44-48.

Chantawong, S. 1973. "Probabilistic Definition of Soil Strength from Different Types of Samplers," M. Eng. Thesis No. 519, Asian Institute of Technology, Bangkok.

Fredlund, D. G., and Dahlman, A. E. 1972. "Statistical Geotechnical Properties of Glacial Lake Edmonton Sediments," Statistics and Probability in Civil Engineering, Hong Kong International Conference, Hong Kong University Press, distributed by Oxford University Press, London.

Geotechnical Engineers, Inc. 1976. "Report on Test Pit Investigation and Laboratory Testing, Preliminary Safety Analysis Report, Pilgrim Station, Appendix 2A.2-4, Pilgrim Nuclear Station No 600, Unit 2, Boston Edison Co.," submitted to Bechtel Corporation.

Gilbert, P. A. 1984. "Investigation of Density Variation in Triaxial Test Specimens of Cohesionless Soil Subjected to Cyclic and Monotonic Loading," Technical Report GL-84-10, US Army Engineer Waterways Experiment Station, Vicksburg, MS, pp 57-58.

Hammitt, G. M. 1966. "Statistical Analysis of Data from a Comparative Laboratory Test Program Sponsored by ACIL," Miscellaneous Paper 4-785, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Harr, M. E. 1987. Reliability Based Design in Civil Engineering, McGraw-Hill Book Company, New York, pp 30-31.

Hvorslev, M. J. 1949. "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes," US Army Engineer Waterways Experiment Station, Vicksburg, MS, 521 pp.

Jaky, J. 1948. "Pressure in Silos," Proceedings, 2nd Int. Conf. on Soil Mechanics and Foundation Engineering, Vol I, pp 103-107.

Lacasse, S., Berre, T., and Lefebvre, G. 1985. "Block Sampling of Sensitive Clays," Proceedings of the 11th international Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA, pp 887-892.

Ladd, C. C., and Foote, R. 1974. "New Design Procedure for Stability of Soft Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol 100, GT7, pp 763-786.

Ladd, C. C., and Lambe, T. W. 1964. "The Strength of 'Undisturbed' Clay Determined from Undrained Tests," Laboratory Shear Testing of Soils, ASTM STP 361, American Society for Testing and Materials, Philadelphia, PA, pp 342-371.

Lambe, T. W., and Whitman, R. V. 1969. Soil Mechanics, John Wiley and Sons, New York.

La Rochelle, P., and Lefebvre, G. 1971. "Sampling Disturbance in Champlain Clays," Sampling of Soil and Rock, ASTM STP 483, American Society for Testing and Materials, Philadelphia, PA, pp 143-163.

La Rochelle, P., Sarrailh, J., Roy, M., and Tavenas, F. 1976. "Effect of Storage and Reconsolidation on the Properties of Champlain Clays," Special Technical Publication 599, American Society for Testing and Materials, Philadelphia, PA, pp 126-146.

La Rochelle, P., Sarrailh, J., Tavenas, F., and Leroueil, S. 1981. "Causes of Sampling Disturbances and Design of a New Sampler for Sensitive Soils," Canadian Geotechnical Journal, Vol 18, No 1, pp 52-66.

Lefebvre, G. 1970. "Contribution à l'étude de la stabilité des pentes dans les argiles cimentées.," D. Sc. thesis, Département of Génie civil, Université Laval, Québec, P. Q.

Lefebvre, G., and Poulin, C. 1979. "A New Method of Sampling in Sensitive Clays," Canadian Geotechnical Journal, Vol 16, pp 226-233.

Lumb, P. 1966. "Variability of Natural Soils," Canadian Geotechnical Journal, Vol 3, No 2.

Marcuson, W. F., III. 1978. "Determination of In Situ Strength of Sands," Dynamic Geotechnical Testing, ASTM STP 654, American Society for Testing and Materials, Philadelphia, PA, pp 318-340.

Marcuson, W. F., III, and Franklin, A. G. 1979. "State of the Art of Undisturbed Sampling of Cohesionless Soils," Proceedings of the International Symposium of Soil Sampling, Singapore, pp 57-71.

McCoy, F. W, Jr. 1972. "An Analysis of Piston Coring Through Corehead Camera Photography," Underwater Soil Sampling, Testing, and Construction Control, STP 501, American Society for Testing and Materials, Philadelphia, PA, pp 90-105.

Nelson, J. D., Brand, E. W., Moh, Z. C., and Mason, I. D. 1971. "The Use of Residual Stress to Define Sample Quality," Proceedings, Specialty Session on Quality in Soil Sampling, Fourth Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, Vol 1, pp 82-87.

Nielsen, D. R., Biggar, J. W., and Erh, K. T. 1973. "Spatial Variability of Field-Measured Soil-Water Properties," Hilgardia, J. Agr. Sci. (Calif. Agr. Experiment Sta.), Vol 42, pp 215-260.

Noorany, I. 1972. "Underwater Soil Sampling and Testing--A State-of-the-Art Review," Underwater Soil Sampling, Testing and Construction Control, ASTM STP 501, American Society for Testing and Materials, Philadelphia, PA, pp 3-41.

Okumura, T. 1971. "The Variation of Mechanical Properties of Clay Samples Depending on Its Degree of Disturbance," <u>Proceedings, Specialty Session on Quality in Soil Sampling</u>, Fourth Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, Vol 1, pp 73-81.

Osterberg, J. O. 1952. "New Piston-Type Soil Sampler," Engineering News-Record, Vol 148, pp 77-78.

__________. 1973. "An Improved Hydraulic Piston Sampler," <u>Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering,</u> Moscow, USSR, Vol 1.2, pp 317-321.

Padilla, J. D., and Vanmarcke, E. H. 1974. "Settlement of Structures on Shallow Foundations: A Probabilistic Analysis," Research Report R74-9, MIT, Cambridge, MA.

Perloff, W. H., and Baron, W. 1976. "Soil Mechanics Principles and Applications," The Ronald Press Company, New York, pp 217-245, 278-280.

Schmertmann, J. H. 1955. "The Undisturbed Consolidation Behavior of Clay," <u>Transactions ASCE,</u> Vol 120, p 1201.

Schultze, E. 1972. "Frequency Distribution and Correlations of Soil Properties," <u>Statistics and Probability in Civil Engineering, Hong Kong International Conference</u>, Hong Kong University Press, distributed by Oxford University Press, London.

__________. 1975 (Sep). "The General Significance of Statistics for the Civil Engineer," <u>Proc. 2d Int. Conf. Application of Statistics and Probability in Soil and Structural Engineering</u> (Aachen, Germany).

Tavenas, F., and Leroueil, S. 1977. "Effects of Stresses and Time on Yielding of Clays," <u>Proceedings, 9th International Conference on Soil Mechanics and Foundation Engineering,</u> Tokyo, Vol 1, pp 319-326.

Terzaghi, K., and Peck, R. R. 1968. <u>Soil Mechanics in Engineering Practice,</u> 2nd Edition, John Wiley and Sons, Inc., New York.

Tirey, G. B. 1972. "Recent Trends in Underwater Soil Sampling Methods," <u>Underwater Soil Sampling, Testing and Construction Control,</u> ASTM STP 501, American Society for Testing and Materials, Philadelphia, PA, pp 42-54.

US Army, Office of the Chief of Engineers. 1972. <u>Soil Sampling</u>, EM 1110-2-1907, Washington, D.C.

US Bureau of Reclamation. 1960. <u>Earth Manual,</u> 1st Edition, Department of the Interior, Bureau of Reclamation, Denver, CO.