Pile Buck Guide to Soil Mechanics and Testing

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Preface

This book is designed to be a comprehensive reference book on both soil mechanics and soil testing. It is primarily intended for the active practitioner in the field, although it is certainly a useful reference for the student.

This book is a "compilation," as it is freely taken from a wide variety of sources of information. Many of these sources are good in and of themselves but both the passage of time and the specific demands of the organization that put them out diminish their usefulness for others who might need the information there. It is our objective to take the best of these publications and other sources of information and to put them into one set of books. A list of these source publications – along with a bibliography on soil mechanics and foundations – can be found in § 11.

There are some distinctive features of this book that need to be noted:

- The footnotes are in fact footnotes and not endnotes, to make them simpler to find. They also constitute a source list for this book as well.
- The nomenclature for the formulas is given with the formulas themselves. The composite nature of the text made harmonizing the various systems of nomenclature a daunting task; we have attempted to do this as much as possible.
- We have included in their entirety the most common laboratory procedures for soils testing. Understanding these is essential to understanding the nature of soils, but including laboratory procedures in soil mechanics textbooks is rare. We have taken the procedures from U.S. Army Corps of Engineers' EM 1110-2-1906, *Laboratory Soils Testing*, and not the ASTM and AASHTO specifications in common use in the U.S.. There are some variations among these specifications and the reader needs to be aware of this. We have not modified the procedures themselves; however, we have rearranged some of the text to properly integrate the procedures with the text, in addition to making a few much-needed corrections.
- All of the laboratory data and reporting forms are in the back of the book, for convenience.

We appreciate any errors or omissions to be brought to our attention.

With this, we commend this work to our readers, while awaiting that last great "fill and grade" job:

> "Let every valley be lifted up, and every mountain and hill be made low; and let the rough ground become a plain, and the rugged terrain a broad valley; Then the glory of the LORD will be revealed, and all flesh will see {it} together; for the mouth of the LORD has spoken." (Isaiah 40:4-5 NAS)

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§ 1. Introduction

Man's earliest attempts at construction probably involved soil. As civilization developed through many centuries, man learned by trial and error about soil as a foundation material. Since World War I, much understanding of soil behavior has been achieved by applying the principles of physics, mechanics, hydraulics, strength of materials, and structural engineering. This approach to analyzing soils problems is called "soil mechanics."

Analysis of soil is more complex than the analysis of other construction materials. Steel and concrete are relatively uniform solids that have predictable strength properties within the elastic range of loading. The strength may be "ordered" in the manufacture of steel and in the making of a concrete mix. This strength will be constant under all climatic conditions. Structures can then be built of these materials with confidence in their strength.

On the other hand, because soil is a very complex medium, a workable solution to soil problems cannot be achieved by a linear, theoretical solution only; it can be reached by a combination of the following sources of information.

1. Experience obtained by trial and error in the past; this developed into the empirical or "rule of thumb" procedures for today. The weakness of this approach is not recognizing differences in the engineering properties of soils. What works well at one location may not succeed with the same type of soil at another location.
2. Information on the properties of soils; generally obtained by field explorations and laboratory tests. Subsequent, theoretical analysis results will only be as good as the soils data used as input.
3. Scientific principles from various fields of engineering and science; used to explain or predict the behavior of soils under various conditions.

Once the soil is analyzed, the foundation must be designed so that the foundation carries the load while not exceeding the capacity of the material either of the foundation itself or the soil under or around it. This having been done, the design is implemented in the construction of the site.

The successful transfer of design objectives into construction is accomplished by consideration is accomplished by consideration of construction operations during the design phase. In recent years, the amount of coordination between design and construction has steadily decreased; primarily due to graduate engineers who specialize in design and who are never exposed to construction operations. In past years, engineers either began their careers in construction and advanced into design, or were assigned the design and construction responsibilities for projects. Present lack of coordination stemming from inexperience with field operations can result in a technically superior set of construction plans and specifications that cannot be built. Rational construction control is vital to assure a safe, cost-effective foundation and to avoid unnecessary court of claims actions.

§ 2. Identification and Classification of Soil and Rock

2.1. Introduction

To perform properly, a structure must interact favorably with the soil on which it rests. The modern foundation engineer, who often must build in areas that were considered too poor to build upon a few years past, must be well versed in the fundamentals of soil mechanics. This knowledge will be used in the design of structural foundations and earthworks to answer questions such as:

- Will settlements be excessive?
- Can the structure tolerate settlements?
- Will the proposed cut or fill slopes have adequate stability?

It is imperative, therefore, that the engineer have adequate knowledge of the soil conditions at a site before he or she can attempt to answer these questions. Investing a few thousand dollars into an adequate boring and testing program may prevent costly failures or overconservative design, resulting in design and construction savings of hundreds of thousands of dollars.

This section presents criteria for soil and rock identification and classification plus information on their physical engineering properties. Common soils and rock are discussed as well as special materials such as submarine soils and coral, saprolitic soils, lateritic soils, expansive and collapsing soils, cavernous limestone, quick clay, permafrost and hydraulically placed fills.

For engineering purposes, we shall consider the earth to be made up of rock and soil.

- Soil will be defined as naturally occurring mineral particles which are readily separated into relatively small pieces, and in which the mass may contain air, water, or organic materials (derived from decay of vegetation).
- Rock is that naturally occurring material composed of mineral particles so firmly bonded together that relatively great effort is required to separate the particles (i.e., blasting or heavy crushing forces).

The mineral particles of the soil mass are formed from decomposition of the rock by weathering (by air, ice, wind, and water) and chemical processes.

2.2. Soils

2.2.1. Overview of Soil Types

See Table 2-1 for principal soil deposits grouped in terms of origin (e.g., residual, colluvial, etc.) and mode of occurrence (e.g., fluvial, lacustrine, etc.).

Table 2-1 Principal Soil Deposits

Major Division	*Principal Soil Deposits*	*Pertinent Engineering Characteristics*
	SEDIMENTARY SOILS	
Residual ***Material formed by disintegration of underlying parent rock or partially indurated material.***	Residual sands and fragments of gravel size formed by solution and leaching of cementing material, leaving the more resistant particles; commonly quartz.	Generally favorable foundation conditions.

	Residual clays formed by decomposition of silicate rocks, disintegration of shales, and solution of carbonates in limestone. With few exceptions becomes more compact, rockier, and less weathered with increasing depth. At intermediate stage may reflect composition, structure, and stratification of parent rock.	Variable properties requiring detailed investigation. Deposits present favorable foundation conditions except in humid and tropical climates, where depth and rate of weathering are very great.
Organic ***Accumulation of highly organic material formed in place by the growth and subsequent decay of plant life.***	Peat. A somewhat fibrous aggregate of decayed and decaying vegetation matter having a dark color and odor of decay.	Very compressible. Entirely unsuitable for supporting building foundations.
	Muck. Peat deposits which have advanced in stage of decomposition to such extent that the botanical character is no longer evident.	
	TRANSPORTED SOILS	
Alluvial ***Material transported and deposited by running water.***	Floodplain deposits. Deposits laid down by a stream within that portion of its valley subject to inundation by floodwaters.	
	Point bar. Alternating deposits of arcuate ridges and swales (lows formed on the inside or convex bank of mitigating river bends.) Ridge deposits consist primarily of silt and sand, swales are clay-filled.	Generally favorable foundation conditions; however, detailed investigations are necessary to locate discontinuities. Flow slides may be a problem along riverbanks. Soils are quite pervious.
	Channel fill. Deposits laid down in abandoned meander loops isolated when rivers shorten their courses. Composed primarily of clay; however, silty and sandy soils are found at the upstream and downstream ends.	Fine-grained soils are usually compressible. Portions may be very heterogeneous. Silty soils generally present favorable foundation conditions.
	Backswamp. The prolonged accumulation of floodwater sediments in flood basins bordering a river. Materials are generally clays but tend become siltier near riverbank.	Relatively uniform in a horizontal direction. Clays are usually subjected to seasonal volume changes.
	Alluvial Terrace deposits. Relatively narrow, flat-surfaced, river-flanking remnants of floodplain deposits formed by entrenchment of rivers and associated processes.	Usually drained, oxidized. Generally favorable foundation conditions.
	Estuarine deposits. Mixed deposits of marine and alluvial origin laid down in widened channels at mouths of rivers and influenced by tide of body of water into which they are deposited.	Generally fine-grained and compressible. Many local variations in soil conditions.
	Alluvial-Lacustrine deposits. Material deposited within lakes (other than those associated with glaciation by waves, currents, and organo-chemical processes. Deposits consist of unstratified organic clay or clay in central portions of the lake and typically grade to stratified silts and sands in peripheral zones.	Usually very uniform in horizontal direction. Fine-grained soils generally compressible.

	Deltaic deposits. Deposits formed at the mouths of rivers that result in extension of the shoreline.	Generally fine-grained and compressible. Many local variations in soil condition.
	Piedmont deposits. Alluvial deposits at foot of hills or mountains. Extensive plains or alluvial fans.	Generally favorable foundation conditions.
Aeolian ***Material transported and deposited by wind.***	Loess. A calcareous, unstratified deposit of silts or sandy or clayey silt traversed by a network of tubes formed by root fibers now decayed.	Relatively uniform deposits characterized by ability to stand in vertical cuts. Collapsible structure. Deep weathering or saturation can modify characteristics.
	Dune sands. Mounds, ridges, and hills of uniform fine sand characteristically exhibiting rounded grains.	Very uniform grain size; may exist in relatively loose condition.
Glacial ***Material transported and deposited by glaciers, or by meltwater from the glacier.***	Glacial till. An accumulation of debris, deposited beneath, at the side (lateral moraines, or at the lower limit of a glacier (terminal moraine. Material lowered to ground surface in an irregular sheet by a melting glacier is known as a ground moraine.	Consists of material of all sizes in various proportions from boulder and gravel to clay. Deposits are unstratified. Generally present favorable foundation conditions; however, rapid changes in conditions are common.
	Glacio-Fluvial deposits. Coarse and fine-grained materials deposited by streams of melt water from glaciers. Material deposited on ground surface beyond terminal of glacier is known as an outwash plain. Gravel ridges known as kames and eskers.	Many local variations. Generally, these present favorable foundation conditions.
	Glacio-Lacustrine deposits. Materials deposited within lakes by melt water from glaciers. Consisting of clay in central portions of lake and alternate layers of silty clay or silt and clay (varved clay in peripheral zones.	Very uniform in a horizontal direction.
Marine ***Material transported and deposited by ocean waves and currents in shore and offshore areas.***	Shore deposits. Deposits of sands and/or gravels formed by the transporting, destructive, and sorting action of waves on the shoreline.	Relatively uniform and of moderate to high density.
	Marine clays. Organic and inorganic deposits of fine-grained material.	Generally very uniform in composition. Compressible and usually very sensitive to remolding.
Colluvial ***Material transported and deposited by gravity.***	Talus. Deposits created by gradual accumulation of unsorted rock fragments and debris at base of cliffs.	Previous movement indicates possible future difficulties. Generally unstable foundation conditions.
	Hillwash. Fine colluvium consisting of clayey sand, sand silt, or clay.	
	Landslide deposits. Considerable masses of soil or rock that have slipped down, more or less as units, from their former position on steep slopes.	

Pyroclastic Material ejected from volcanoes and transported by gravity, wind and air.	Ejecta. Loose deposits of volcanic ash, lapilli, bombs, etc.	Typically shardlike particles of silt size with larger volcanic debris. Weathering and redeposition produce highly plastic, compressible clay. Unusual and difficult foundation conditions.
	Pumice. Frequently associated with lava flows and mudflows, or may be mixed with nonvolcanic sediments.	

Beyond these general geologic classifications, geotechnical engineers further classify soils to enable quantification of their engineering properties. Two systems are discussed in this book: the Unified Soil Classification System (USCS) and the Modified Unified System (MUD). The laboratory and field tests used with both of these systems, such as tests for grain size, Atterberg limits, etc., are discussed elsewhere in this book.

2.2.2. Unified Soil Classification System (USCS)

This system is used primarily for engineering purposes and is particularly useful to the geotechnical engineer. Therefore, they should be used for all structural-related projects; such as bridges, retaining walls, buildings, etc. Precise classification requires that a grain size analysis and Atterberg Limits tests be performed on the sample. The method is discussed in detail in ASTM D 2487 and a summary is reprinted in Figure 2-1 and Figure 2-2 for convenience. Tests for grain size, quantities D_{10}, D_{30} and D_{60}, and coefficients of uniformity and curvature are discussed in detail in 3.6.

Figure 2-1 Unified Soil Classification System, Cohesionless Soils[1]

Primary Divisions for Field and Laboratory Identification			Group Symbol	Typical Names	Laboratory Classification Criteria		Supplementary Criteria For Visual Identification
Coarse-grained soils. (More than half of material finer than 3-inch sieve is larger than No. 200 sieve size.)	Gravel. (More than half of the coarse fraction is larger than No. 4 sieve size about 1/4 inch.)	Clean gravels. (Less than 5% of material smaller than No. 200 sieve size.)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.*	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4. $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3.		Wide range in grain size and substantial amounts of all intermediate particle size.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.*	Not meeting both criteria for GW.		Predominantly one size (uniformly graded) or a range of sizes with some intermediate sizes missing (gap graded).
.....do....	.....do....	Gravels with fines. (More than 12% of material smaller than No. 200 sieve size.)*	GM	Silty gravels, and gravel-sand-silt mixtures.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 & 7 is borderline case GM-GC	Nonplastic fines or fines of low plasticity.
			GC	Clayey gravels, and gravel-sand-clay mixtures.	Atterberg limits above "A" line, and PI greater than 7.		Plastic fines.
....do.....	Sands. (More than half of the coarse fraction is smaller than No. 4 sieve size.)	Clean sands. (Less than 5% of material smaller than No. 200 sieve size.)	SW	Well graded sands, gravelly sands, little or no fines.*	$C_u = \frac{D_{60}}{D_{10}}$ greater than 6. $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3.		Wide range in grain sizes and substantial amounts of all intermediate particle sizes.
			SP	Poorly graded sands and gravelly sands, little or no fines.*	Not meeting both criteria for SW.		Predominately one size (uniformly graded) or a range of sizes with some intermediate sizes missing (gap graded).
.....do....	.....do....	Sands with fines. (More than 12% of material smaller than No. 200 sieve size.)*	SM	Silty sands, sand-silt mixtures.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 and 7 is borderline case SM-SC.	Nonplastic fines or fines of low plasticity.
			SC	Clayey sands, sand-clay mixtures.	Atterberg limits above "A" line with PI greater than 7.		Plastic fines.

[1] Cohesionless materials with 5-12% smaller than No. 200 sieve are borderline cases, designated GW-GM, SW-SC, etc.

Figure 2-2 Unified Soil Classification System, Cohesive Soils

Primary Divisions for Field and Laboratory Identification		Group Symbol	Typical Names	Laboratory Classification Criteria		Supplementary Criteria For Visual Identification: Dry Strength	Reaction to Shaking	Toughness Near Plastic Limit
Fine-grained soils. (More than half of material is smaller than No. 200 sieve size.) (Visual: more than half of particles are so fine that they cannot be seen by naked eye.)	Silts and clays. (Liquid limit less than 50.)	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 and 7 is border-line case ML-CL.	None to slight	Quick to slow	None
	........do........	CL	Inorganic clays of low to medium plasticity; gravelly clays, silty clays, sandy clays, lean clays.	Atterberg limits above "A" line, with PI greater than 7.		Medium to high	None to very slow	Medium
	........do........	OL	Organic silts and organic silt-clays of low plasticity.	Atterberg limits below "A" line.		Slight to medium	Slow	Slight

Primary Divisions for Field and Laboratory Identification		Group Symbol	Typical Names	Laboratory Classification Criteria	Supplementary Criteria For Visual Identification: Dry Strength	Reaction to Shaking	Toughness Near Plastic Limit
.....do....	Silts and clays. (Liquid limit greater than 50.)	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.	Atterberg limits below "A" line.	Slight to medium	Slow to none	Slight to medium
	........do........	CH	Inorganic clays of high plasticity, fat clays.	Atterberg limits above "A" line.	High to very high	None	High
	........do........	OH	Organic clays of medium to high plasticity.	Atterberg limit below "A" line	Medium to high	None to very slow	Slight to medium
.....do....	Highly organic soils............	Pt	Peat, muck and other highly organic soils.	High ignition loss, LL and PI decrease after drying.	Organic color and odor, spongy feel, frequently fibrous texture.		

2.2.2.1. Coarse-Grained (Cohesionless or Granular) Soils

Coarse-grained soils are those soils where more than half of particles finer than 3" size can be distinguished by the naked eye. The smallest particle that is large enough to be visible corresponds approximately to the size of the opening of No. 200 sieve used for laboratory identification. Sands are divided from gravels on the No. 4 sieve size, and gravels from cobbles on the 3" size. The division between fine and medium sands is at the No. 40 sieve, and between medium and coarse sand at the No. 10 sieve. Generally, the engineering properties of cohesionless or granular soils are as follows:

1. Excellent foundation material for supporting structures and roads.
2. The best embankment material.
3. The best backfill material for retaining walls.
4. Might settle under vibratory loads or blasts.
5. Dewatering can be difficult due to high permeability.
6. If free draining not frost susceptible.

2.2.2.2. Fine-Grained (Cohesive or Organic) Soils

Soils are identified as fine-grained when more than half of the particles are finer than No. 200 sieve (as a field guide, such particles cannot be seen by the naked eye). Fine-grained soils are classified according to plasticity characteristics determined in Atterberg limit tests. Categories are illustrated on the plasticity chart in Figure 11-6, where the classification procedure is discussed in more detail.

In general, the engineering properties of cohesive soils are as follows:

1. Very often, possess low shear strength.
2. Plastic and compressible.
3. Loses part of shear strength upon wetting.
4. Loses part of shear strength upon disturbance.
5. Shrinks upon drying and expands upon wetting.
6. Very poor material for backfill.
7. Poor material for embankments.
8. Practically impervious.
9. Clay slopes are prone to landslides.

Differing from clays are silts; some characteristics of silts are as follows:

1. Relatively low shear strength
2. High Capillarity and frost susceptibility
3. Relatively low permeability
4. Difficult to compact

Compared to clay, silts exhibit the following characteristics:

5. Better load sustaining qualities
6. Less compressible
7. More permeable

8. Exhibit less volume change

2.2.2.3. Organic Soils

Although their grain size can classify organic soils with silts and clays, they are in fact very different. Any soil containing a sufficient amount of organic matter to influence its engineering properties is called an organic soil. The term organic designates those soils containing an appreciable amount of decayed animal and/or vegetative matter in various states of decomposition.

Materials containing vegetable matter are characterized by relatively low specific gravity, high water content, high ignition loss, and high gas content. Decrease in liquid limit after oven drying to a value less than three-quarters of the original liquid limit is a definite indication of an organic soil. The Unified Soil Classification categorizes organic soils based on the plotted position on the A-line chart as shown in Figure 11-6. However, this does not describe organic soils completely. Therefore, Table 2-2[2] is provided for a more useful classification of organic soils.

Table 2-2 Soil Classifications for Organic Soils

Category	Name	Organic Content (% by wt.)	Group Symbols (See Table 3)	Distinguishing Characteristics For Visual Identification	Range of Laboratory Test Values
ORGANIC MATTER	FIBROUS PEAT (woody, mats, etc.)	75 to 100% Organics either visible or inferred	Pt	Light weight, spongy and often elastic at w_n--shrinks considerably on air drying. Much water squeezes from sample.	w_n--500 to 1200% γ--60 to 70 pcf G--1.2 to 1.8 $C_c/(1+e_o)$=.4+
	FINE GRAINED PEAT (amorphous)			Light weight, spongy but not often elastic at w_n--shrinks considerably on air drying. Much water squeezes from sample.	w_n--400 to 800% LL--400 to 900% PI--200 to 500 γ--60 to 70 pcf G--1.2 to 1.8 $C_c/(1+e_o)$=.35 to .4+
HIGHLY ORGANIC SOILS	Silty Peat	30 to 75% Organics either visible or inferred	Pt	Relatively light weight, spongy. Thread usually weak and spongy near PL Shrinks on air drying; medium dry strength. Usually can squeeze water from sample readily--slow dilatency.	w_n--250 to 500% LL--250 to 600% PI--150 to 350 γ--65 to 90 pcf G--1.8 to 2.3 $C_c/(1+e_o)$=.3 to .4
	Sandy Peat			Sand fraction visible. Thread weak and friable near PL; shrinks on air drying; low dry strength. Usually can squeeze water from sample readily--high dilatency--"gritty."	w_n--100 to 400% LL--150 to 300% (plot below A line) PI--50 to 150 γ--70 to 100 pcf G--1.8 to 2.4 $C_c/(1+e_o)$=.2 to .3

[2] Ayers J., and Plum, R., Unpublished work.

Category	Name	Organic Content (% by wt.)	Group Symbols (See Table 3)	Distinguishing Characteristics For Visual Identification	Range of Laboratory Test Values
ORGANIC SOILS	Clayey ORGANIC SILT	5 to 30% Organics either visible or inferred	OH	Often has strong H_2S odor. Thread may be tough depending on clay fraction. Medium dry strength, slow dilatency.	w_n--65 to 200% LL--65 to 150% (usually plot at or near A line) PI-- 50 to 150 γ --70 to 100 pcf G--2.3 to 2.6 $C_c/(1+e_o)$=.20 to .35
	Organic SAND or SILT		OL	Threads weak and friable near PL--or may not roll at all. Low dry strength; medium to high dilatency.	w_n--30 to 125% LL--30 to 100% (usually plot well below A line) PI--non-plastic to 40 γ --90 to 110 pcf G--2.4 to 2.6 $C_c/(1+e_o)$=.1 to .25
SLIGHTLY ORGANIC SOILS	SOIL FRACTION add slightly Organic	Less than 5% Organics combined visible and inferred	Depend upon inorganic fraction	Depend upon the characteristics of the inorganic fraction.	Depend upon inorganic fractions.

The organic matter is objectionable for three main reasons:

1. Reduces load-carrying capacity of soil.
2. Increases compressibility considerably.
3. Frequently contains toxic gasses that are released during the excavation process.

All organic soils, whether they are peat, organic clays, organic silts, or even organic sands, should be viewed with suspicion as foundation and construction materials.

2.2.2.4. Typical Properties

One of the main uses of soil classification systems is the determination of various soil properties based on the classification of the soil. Soils can be classified by the Unified System using visual inspection, Atterberg Limits and sieve analysis, tests which a) are relatively easy to perform and b) make greater allowance for sample disturbance than, say, consolidation or triaxial tests. Many correlations exist that enable soil properties to be estimated using the soil classification and in some cases other relatively simple tests. These correlations enable the geotechnical engineer to make estimates – in some cases the only estimates possible – of soil properties for preliminary design purposes.

Many of the “typical properties” of soils are based on SPT results; these are discussed with the SPT test in 4.9.1. Those discussed in this section are derived from other tests. It should be noted that these “typical” properties and correlations are of varying quality, expressed by standard deviation, which is the range above and below the average trend, within which about two-thirds of all values occur. These relationships are useful in preliminary analyses but must not supplant careful tests of structural properties. The relationships should never be applied in final analyses without verification by tests of the particular material concerned.

2.2.2.4.1. Index Properties

Some typical index properties of both cohesionless and cohesive soils classified by the Unified System are provided in Table 2-3[3].

Table 2-3 Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids (1)					Unit Weight (2) (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	–	–	–	1.0	0.92	–	0.35	47.6	26	–	–	–	–	–	–	–
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	–	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	–	–	–	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	–	0.40	52	29	80	–	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	–	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	–	–	–	–	1.2	–	0.40	55	29	76	–	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	–	0.14	46	12	89	–	146(3)	90	155(3)	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	–	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	–	–	1.0	–	0.20	50	17	84	–	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	–	0.13	41	11	100	140	148(4)	125	156(4)	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5µ	0.001	–	2.4	–	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	–	–	12	–	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	–	–	–	–	3.0	–	0.55	75	35	40	–	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	–	–	–	–	4.4	–	0.70	81	41	30		100	81	125	18	62

2.2.2.4.2. Internal Friction and Relative Density of Granular Soils

Correlations between friction angle, relative density and Unified classifications are shown in Figure 2-3 and Figure 2-4.

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[3] Hough, B.K., Basic Soils Engineering, Ronald Press, New York, 1969.

Figure 2-3 Correlations of Strength Characteristics for Granular Soils

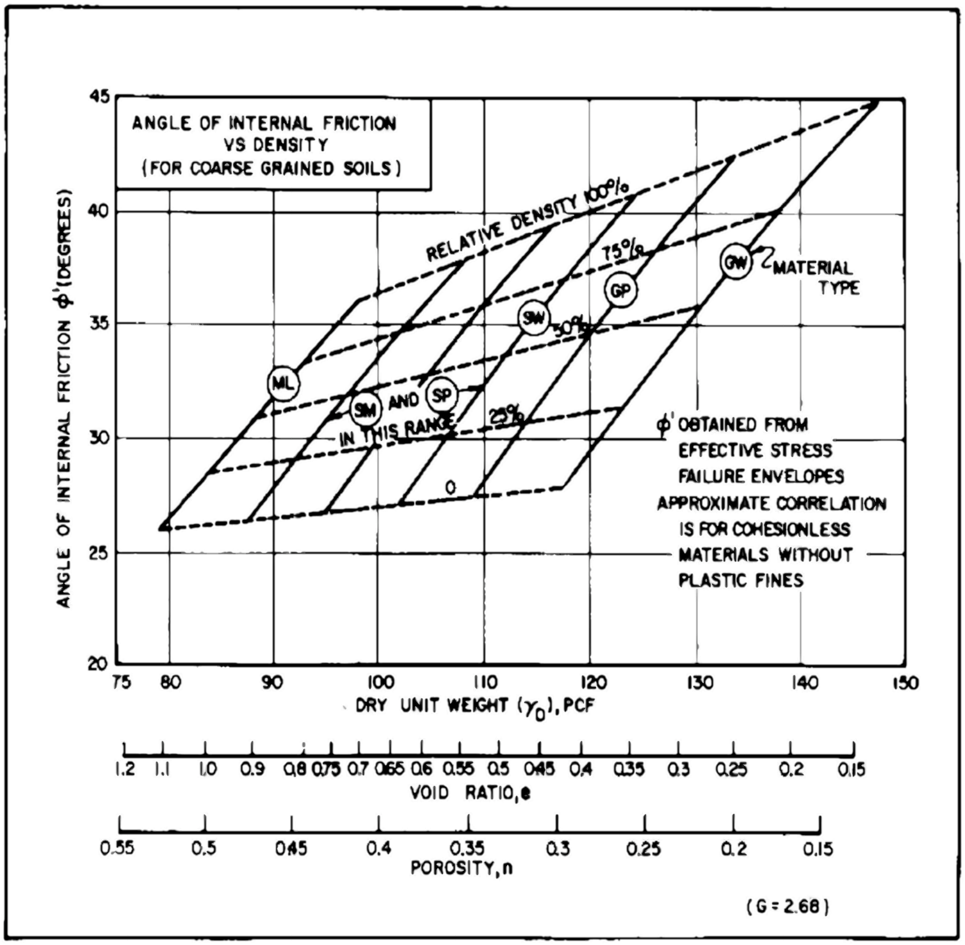
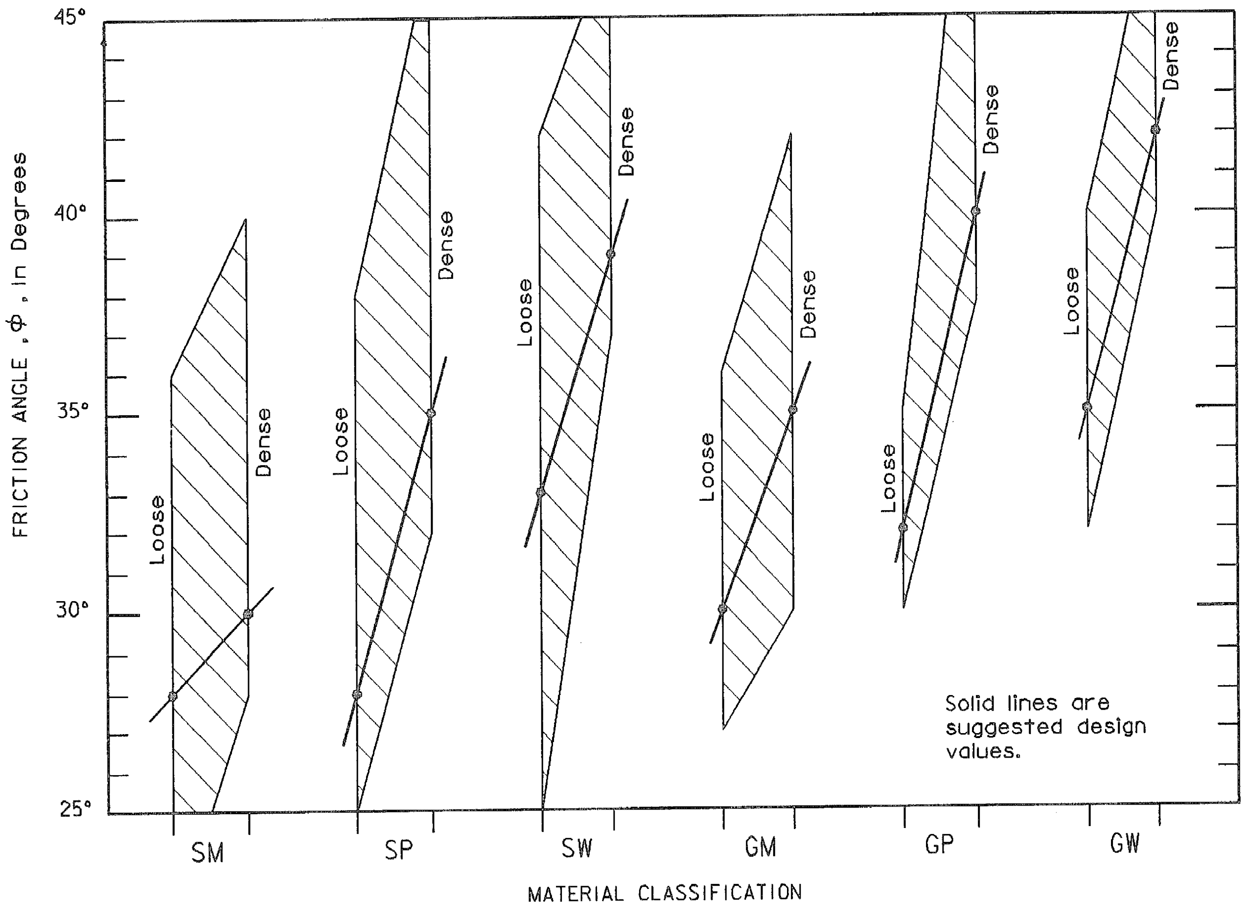


Figure 2-4 Friction Angle of Granular Backfills



2.2.2.5. Examples of Sample Descriptions

- Granular soils:
  - Medium dense, gray coarse to fine SAND, trace silt, trace fine gravel (SW).
  - Dry, dense, light brown coarse to fine SAND, some silt (SM).
- Fine grained soils:
  - Very stiff brown silty CLAY (CL), wet
  - Stiff brown clayey SILT (ML), moist
  - Soft dark brown organic CLAY (OH), wet.

2.2.3. Modified Unified System (MUD)

For many years, soils engineers have successfully used the Unified Soil Classification System to categorize soil samples. The major advantage of this system is the easily understood word picture used to describe the soil samples after classification. The major disadvantage is the number of time-consuming classification tests that must be done to develop the word picture.

At present, numerous private firms and State agencies are using the nomenclature of the Unified System but without the classification testing. This process of visually identifying soil samples as known as the Modified Unified Description (MUD).

The procedure involves visually and manually examining soil samples with respect to texture, plasticity and color. A method is presented for preparing a "word picture" of a sample for entering on a subsurface exploration log or other appropriate data sheet. The procedure applies to soil descriptions made in the field or laboratory.

It should be understood that the soil descriptions are based upon the judgment of the individual making the description. Classification tests are not intended to be used to verify the description, but to provide further information for analysis of soil design problems or for possible use of the soil as a construction material.

It is the intent of this system to describe only the constituent soil sizes that have a significant influence on the visual appearance and behavior of the soil. This description system is intended to provide the best word description of the sample to those involved in the planning, design, construction, and maintenance processes.

2.2.3.1. Definition of Terms

- Boulder - A rock fragment, usually rounded by weathering or abrasion, with an average dimension of 12 inches or more.
- Cobble - A rock fragment, usually rounded or subrounded, with an average dimension between 3 and 12 inches.
- Gravel[4] - Rounded, subrounded, or angular particles of rock that will pass a 3 inch square opening sieve (76.2 mm) and be retained on a Number 10 U.S. standard sieve (2.0 mm).
- Sand - Particles that will pass the Number 10 U.S. standard sieve and be retained on the Number 200 U.S. standard sieve (0.074 mm).
- Silt - Material passing the Number 200 U.S. standard sieve that is nonplastic and exhibits little or no strength when dried.[5]
- Clay - Material passing the Number 200 U.S. standard sieve that can be made to exhibit plasticity (putty like property) within a wide range of water contents and exhibits considerable dry strength.[6]
- Fines - The portion of a soil passing a Number 200 U.S. standard sieve.
- Marl - Unconsolidated white or dark gray calcium carbonate deposit.
- Muck - Finely divided organic material containing various amounts of mineral soil.
- Pest - Organic material in various stages of decomposition.
- Organic Clay - Clay containing microscopic size organic matter. May contain shells and/or fibres.
- Organic Silt - Silt containing microscopic size organic matter. May contain shells and/or fibres.
- Coarse-Grained Soil - Soil having a predominance of gravel and/or sand.
- Fine-Grained Soil - Soil having a predominance of silt and/or clay.

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[4] The term "gravel" in this system denotes a particle size range and should not be confused with "gravel" used to describe a type of geological deposit or a construction material.

[5] New York State Soil Mechanics Bureau STP-2 - Issuance No. 7.41-5/75 "Soil Description Procedure"

[6] When applied to gradation test results, silt size is defined as that portion of the soil finer than the No. 200 U.S. standard sieve and coarser than 0.002 mm. Clay size is that portion of soil finer than 0.002 mm. For the visual-manual procedure, the identification will be based on plasticity characteristics.

- Mixed-Grained Soil - Soil having significant proportions of both fine-grained and coarse-grained sizes.

2.2.3.2. Visual - Manual Identification

Constituents are identified considering grain size distribution and the results of the manual tests. In addition to the principal constituent, other constituents that may affect the engineering properties of the soil should be identified. Secondary constituents are generally indicated as modifiers to the principal constituent (i.e., sandy clay or silty gravel). Other constituents can be included in the description through the use of terms such as with, some and trace. Details of visual identification of samples can be found in Table 2-4.

Table 2-4 Visual Identification of Samples

Definitions of Soil Components and Fractions		
1. Grain Size		
Material	Fraction	Sieve Size
Boulders		12"+
Cobbles		3" - 12"
Gravel: The particles may have an angular, rounded, or subrounded shape. Gravel size particles usually occur in varying combinations with other particle sizes.	Coarse Fine	3/4" - 3" No. 4 to 3/4"
Sand: Gritty grains that can easily be seen and felt. No plasticity or cohesion.	Coarse Medium Fine	No. 10 to No. 4 No. 40 to No. 10 No. 200 to No. 40
Fines (Silt & Clay):		Passing No. 200
∞ Silt - Identified by behavior. Fines that have no plasticity. May be rolled into a thread but will easily crumble. Has no cohesion. When dry, can be easily broken by hand into powdery form.		
∞ Clay - Identified by behavior. Fines are plastic and cohesive when in a moist or wet state. Can be rolled into a thin thread that will not crumble. When dry, forms hard lumps that cannot be readily broken by hand.		
∞ Clay is often encountered in combination with other soil		

sizes. If a sample exhibits plasticity or cohesion, it contains clay. The amount of clay can be related to the degree of plasticity or cohesiveness; the higher the clay content the greater the plasticity.		
2. Grading of Coarse Soils [7]		
	Well-Graded	Soil contains a good representation of all particle sizes from largest to smallest.
	Poorly-Graded	Soil contains particles about the same size. A soil of this type is sometimes described as being uniform.
	Gap-Graded	Soil does not contain one or more intermediate particles sizes. A soil consisting of gravel and fine sand would be gap graded because of the absence of medium and coarse sand sizes.
2. Coarse- and Fine-Grained Soils		
	Descriptive Adjective	Percentage Requirement
	Trace	1 - 10%
	Little	10 - 20%
	Some	20 - 35%
	And	35 - 50%
3. Fine-Grained Soils. Identify in accordance with plasticity characteristics, dry strength, and toughness.		
	Descriptive Term	Thickness
	Alternating	
	Thick	
Stratified Soils	Thin With	
	Parting	0 to 1/16" thickness
	Seam	1/16 to 1/2" thickness
	Layer	1/2 to 12" thickness
	Stratum	Greater than 12" thickness

[7] Descriptions of fine-grained soils should not include a grading.

	Varved Clay	Alternating seams or layers of sand, silt and clay
	Pocket	Small, erratic deposit, usually less than 1 foot
	Lens	Lenticular deposit
	Occasional	One or less per foot of thickness
	Frequent	More than one per foot of thickness

Other terms that might be used include the following:

- Marl - A white or grey calcium carbonate paste. May contain granular spheres, shells, organic material or inorganic soils. Reacts with weak hydrochloric acid.
- Muck - Black or dark brown finely divided organic material mixed with various proportions of sand, silt, and clay. May contain minor amounts of fibrous material such as roots, leaves, and sedges.
- Pest - Black of dark brown plant remains. The visible plan remains range from coarse fibers to finely divided organic material.
- Organic Clay - Dark grey clay with microscopic size organic material dispersed throughout. May contain shells and/or fibers. Has weak structure that exhibits little resistance to kneading.
- Organic Silt - Dark grey silt with microscopic size organic material dispersed throughout. May contain shells and/or fibers. Has weak structure that exhibits little resistance to kneading.
- Fill - Man-made deposits of natural soils and/or waste materials. Document the components carefully since presence and depth of fill are important engineering considerations.

2.2.3.3. Soil Sample Identification Procedure

1. First Decision -
   a. Is sample coarse-grained, fine-grained, mixed-grained or organic?
   b. If mixed-grained, decide whether coarse-grained or fine-grained predominates.
2. 2nd Decision -
   a. What is principal component?
   b. Use as noun in soil description. Example: Silty Sand
3. 3rd Decision -
   a. What is secondary component?
   b. Use as adjective in soil description. Example: Silty Sand
4. 4th Decision -
   a. Are there additional components?
   b. Use as additional adjective. Example: Silty Sand, Gravelly

2.2.3.4. Examples Of Descriptions Of The Soil Components

- Sand - Describes a sample that consists of both fine and coarse sand particles.
- Gravel - Describes a sample that consists of both fine and coarse gravel particles.

- Silty fine Sand - Major component fine sand, with nonplastic fines.
- Sandy Gravel - Major component gravel size, with fine and coarse sand. May contain small amount of fines.
- Gravelly Sand - Major component sand, with gravel. May contain small amount of fines.
- Gravelly Sand, Silty - Major component sand, with gravel and nonplastic fines.
- Gravelly Sand, Clayey - Major component sand, with gravel and plastic fines.
- Sandy Gravel, Silty - Major component gravel size, with sand and nonplastic fines.
- Gravelly Sand, Clayey - Major component gravel size, with sand and plastic fines.
- Silty Gravel - Major component gravel size, with nonplastic fines. May contain sand.
- Clayey Gravel - Major component gravel size, with plastic fines. May contain sand and silt.
- Clayey Silt - Major component silt size, with sufficient clay to impart plasticity and considerable strength when dry.
- Silty Clay - Major component clay, with silt size. Higher degree of plasticity and higher dry strength than clayey silt.

The above system may be expanded where necessary to provide meaningful descriptions of the sample. Examples: Shale fragments - Cobble and gravel size, silty. Decomposed rock - Gravel size

2.2.3.5. Other Information for Describing Soils

1. Color Of The Sample - Brown, Grey Red, Black, etc. The color description is restricted to two colors. If more than two colors exist, the soil should be described as multi-colored or mottled and the two predominant colors given.
2. Moisture condition. Judge by appearance of sample before manipulating. The in-situ moisture content of a soil should be described as dry, moist, or wet.
3. Plasticity - Plastic, Low Plastic, Nonplastic. Note: Sample must be in moist or wet condition for plasticity determination. For dry samples requiring wetting make note in description. Example - "plastic (low or nonplastic) when wet." Plasticity not required for marl, muck and peat.
4. Structure - Fissured, Blocky, Varved, Layered. (Indicate approximate thickness of layers). The description of layering for coarse-grained soils must be made from field observations before sample is removed from sampler.
5. Particle shape. Coarse-grained soils are described as angular, sub-angular, sub-rounded, or rounded. Gravel, cobbles, and boulders can be described as flat, elongated, or flat and elongated. Descriptions of fine-grained soils will not include a particle angularity or shape.
6. Any additional descriptive terms considered helpful in identifying the soil should be included. Examples of such terms include calcareous, cemented, and gritty. Material origins or local names should be included in parentheses (i.e., fill, iron rock)

2.2.3.6. Preparing the Word Picture

The word-picture is the description of the soil sample as determined by the visual-manual procedure. Where applicable, the following are to be included in the word-picture (*a sample of this appears also*):

1. Colour of the sample: *Brown*
2. Description of Soil Components: *Silty Gravel*

3. Moisture Condition: *moist*
4. Plasticity: *nonplastic*
5. Structure
6. Particle shape: *angular*
7. Other: *cemented*

The written description for the given example is: Brown Silty angular Gravel, moist, nonplastic, cemented.

2.2.3.7. Examples Of Complete Soil Descriptions

∞ Light Grey Silty Clay, moist, plastic, with 1/2 inch layers of wet, grey Silt, nonplastic

∞ Red brown Clayey Silt with 1/4 inch layers of Silty Clay, moist, plastic

∞ Brown Silty fine Sand, wet, nonplastic

∞ Grey Sandy rounded Gravel, dry, nonplastic

∞ Grey Sandy angular Gravel, Clayey, moist, low plastic

∞ Dark Brown Silty Sand, wet, nonplastic

∞ Red Brown Sand, dry, nonplastic, with roots

∞ Fill - Brown Sandy subrounded Gravel, with pieces of brick and cinders, wet, nonplastic

∞ Fill containing cinders, paper, garbage, and glass, wet

∞ Dark Grey Organic Clay, with shells and roots, moist, plastic

2.2.4. Relative Density or Consistency

Relative density refers to the degree of compactness of a coarse-grained soil. Consistency refers to the stiffness of a fine-grained soil. Standard Penetration Test N-values are usually used to define the relative density and consistency. Both the tests and the correlations are discussed in more detail in 4.9.1.

If static cone penetration resistance q_c and N values are measured during the field exploration, a q_c-N correlation could be made, and Table 4-12 is used to describe compactness. If N is not measured, but q_c is measured, then use $N = q_c/4$ for sand and fine to medium gravel and $N = (q_c/5)$ for sand, and use Table 4-12 for describing compactness.[8]

If SPT data is not available, consistency can be estimated based on visual-manual examination of the material. Refer to ASTM D 2488 for consistency criteria.

2.3. Logging

Sample completed logs, the form of which is useful for borings and test pits alike, are shown in Figure 2-5 or Figure 2-6. The majority of information to be included on this form is self-explanatory.

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[8] Mitchell, J.K. and Lunne, T.A., Cone Resistance as Measure of Sand Strength, Journal of the geotechnical engineering Division, ASCE, Vol. 104, No. GT7, 1978.

Figure 2-5 English Units Typical Boring Log

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

FIELD BORING LOG

FORM 675-020-12
MATERIALS - 05/94

SHEET [illegible] OF 3

PROJECT NO. 79100-1523 NAME SR-40 over Tomoka River COUNTY Volusia DISTRICT 5

LOCATION STA 14+80, 25 ft RT CL Survey TOWNSHIP 14S RANGE 31E SECTION 25

ROAD NUMBER SR-40 SURFACE ELEVATION +22.6 ft, NGVD

EQUIPMENT TYPE CME-45, Automatic Hammer RIG NO. 7476 BORING NO. 4

DATE STARTED 8/27/90 COMPLETED 8/28/90 DRILLED BY Jenkins

LOGGED BY Dawson BORING TYPE: AUGER, WASHED, PERCUSSION, ROTARY, CASED, UNCASED, DRILLING MUD, To 14.5 ft

WATER TABLE: 0 HR. 4.2 ft 24 HRS. 4.2 ft HRS. ______

SAMPLE CONDITIONS: DISTURBED, GOOD, LOST, CORE SAMPLE

SAMPLE TYPES: A: AUGER; SB: SPLIT BARREL; S: SHELBY TUBE; RC: ROCK CORE NX SIZE

TESTS: W.C.: WATER CONTENT (%); T: TORVANE (TSF); V: IN-SITU VANE TEST (TSF)

ELEV. (FT.) 22.6	DEPTH (FT.)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES CON.	NO. TYPE	REC. (%)	TESTS	REMARKS
		2, 3, 5	Light Brown Fine SAND, Poorly Graded, Loose to Compact, Moist to Wet, Sub-Angular (SP)		SB-1	50		
		5, 7, 8			SB-2	60		
17.6	5							
		0, 0, 1	Dark Brown Sandy SILT, some Wood, Very Loose, Wet, Fibrous (Muck) (ML)		SB-3	20		Advanced 12" Under Weight of Hammer
		1, 1, 2	Reddish-Brown Silty CLAY, Trace Sand and Shell, Soft to Firm, Wet (CL)		SB-4	80		
12.6	10							
					S-1	100	T = 0.4	
		3, 3, 4			SB-5	100		
7.6	15	12, 16, 25	Tan LIMESTONE, Highly to Moderately Weathered, Soft		SB-6	40		
					RC-1	75		RQD = 38% Loss of Water at 17.8 ft
2.6	20	46, 50/3"			SB-7	30		Boring Terminated at 20.5 ft Backfilled 8/28/90

Figure 2-6 Metric Units Typical Boring Log

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
FIELD BORING LOG

FORM 675-020-13
MATERIALS - 05/94

SHEET 1 OF 5

PROJECT NO. 79100-1523 NAME S.R. 40 / Tomoka River COUNTY Volusia DISTRICT 5

LOCATION STA 14+80, 27.5 m RT CL Survey TOWNSHIP 14S RANGE 31E SECTION 25

ROAD NUMBER State Road # 40 SURFACE ELEVATION +0.68 m, NGVD

EQUIPMENT TYPE CME 45 RIG NO. 7476 BORING NO. 4

DATE STARTED 8/27/90 COMPLETED 8/28/90 DRILLED BY Jenkins

LOGGED BY Dawson BORING TYPE: AUGER, WASHED, PERCUSSION, ROTARY, Rotary
CASED, UNCASED, DRILLING MUD, Cased/Uncased

WATER TABLE: 0 HR. 0.46 m 24 HRS. 0.46 m HRS. ______

SAMPLE CONDITIONS: DISTURBED, GOOD, LOST, CORE SAMPLE

SAMPLE TYPES: A: AUGER; SB: SPLIT BARREL; S: SHELBY TUBE; RC: ROCK CORE ______ SIZE

TESTS: W.C.: WATER CONTENT (%); T: TORVANE (kPa); V: IN-SITU VANE TEST (kPa)

ELEV. (M)	DEPTH (M)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES CON.	SAMPLES NO. TYPE	SAMPLES REC. (%)	TESTS	REMARKS
		2						
		4	Dark brown fine SAND, trace peat (SP)					
		4		G	S-1	100		
		4						
		2	Light grey to dark brown fine SAND (SP)					
		3		G	S-2	20		
		2						
		2					WC = 20	
		2		G	S-3	50	-200 = 3	
-0.82	1.5	2						
		3						
		3		G	S-4	100		
		2						
		3						
		4		G	S-5	100		
		3						
		4					WC = 20	
		3		G	S-6	100	-200 = 3	
		5						
-2.32	3.0	6						
		7		G	S-7	100		
		3						
		9						
		13		G	S-8	100		
		8						
		7	Greenish-grey silty fine SAND,				WC = 29	
		5	few shell fragments (SM)	G	S-9	100	-200 = 18	
		6						
		9						
-3.82	4.5	4		G	S-10	100		
			Light to dark grey fine SAND with silt,					
			trace to few shell (SP-SM)					
		7						
		8						
		11		G	S-11	75		
-5.32	6.0	7						
		2						

RECYCLED PAPER

2.3.1. Comments on Drilling Procedures and/or Problems

Any occurrences, which may indicate characteristics of the in-situ material, should be reported. Such occurrences include obstructions; difficulties in drilling such as caving, surging sands, or caverns; loss of drilling fluid; change in drilling method; and termination of boring above planned depth.

2.3.2. Test Results

Results of tests performed on samples in the field, such as pocket penetrometer or torvane tests should be noted. Results of tests on in-situ materials, such as field vane tests, should also be recorded.

2.4. Rocks

Rock is classified with respect to its geological origin as follows:

- Igneous rocks, such as granite, diorite and basalt, are those formed by the solidification of molten material, either by intrusion at depth in the earth's crust or by extrusion at the earth's surface.
- Sedimentary rocks, such as sandstone, limestone and shale, are those rocks formed by deposition, usually under water, of products derived by the disaggregation of pre-existing rocks.
- Metamorphic rocks, such as quartzite, schist and gneiss, may be either igneous or sedimentary rocks that have been altered physically and sometimes chemically by the application of intense heat and pressure at some time in their geological history

A preliminary estimate of the physical and engineering properties can be made based on the classification criteria given together with published charts, tables and correlations interpreted by experienced engineering geologists[9].

2.4.1. Visual Classification

Geological structures generally have a significant influence on the rock mass properties. Some of the important features are described as follows:

- Rock mass means an aggregate of blocks of solid rock material containing structural features that constitute mechanical discontinuities. Rock mass refers to any *in situ* rock with all inherent geomechanical discontinuities.
- Rock material or intact rock means the consolidated aggregate of mineral particles forming solid material between structural discontinuities. Properties attributed to it refer to rock material free of geomechanical discontinuities.
- Geomechanical or structural discontinuities means all geological features which separate solid blocks of the rock mass, such as joints, faults, bedding planes, cleavage planes, shear zones, and solution cavities. These features constitute planes of weakness that reduce the strength of the rock mass appreciably.

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[9] Deere, D.U. and Patton, F.D., Slope Stability in Residual Soils, Proceedings of the Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering, San Juan, Volume 1, pp 87-100, 1971, and Deere, D.U., Geologic Considerations, Rock Mechanics in Engineering Practice, Stagg, K.G. and Zienkiewicz, O.C., ed., John Wiley and Sons, New York, Chapter 1, 1969, and Farmer, I.W., Engineering Properties of Rocks, E&FN Spon LTP, London, 1968. Guidance is provided in these works for a) for description of weathered igneous and metamorphic rock (residual soil, b) transition from residual to saprolite, etc.) in terms of RQD, c) percent core recovery, d) relative permeability, e) strength, and f) typical strength parameters for weathered igneous and metamorphic rocks.

- Major discontinuities or major structures means those geological features constituting structural discontinuities which are sufficiently well developed and continuous that shear failure along them would involve little or no shearing of intact and rock material.

With this in mind, we can describe a rock sample in the following sequence:

- Weathering Classification. Describe as fresh, slightly weathered, etc. in accordance with Table 2-5[10].

Table 2-5 Weathering Classification

Grade	*Symbol*	*Diagnostic Procedures*
Fresh	F	No visible sign of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	WS	Slight discoloration inwards from open fractures, otherwise similar to F.
Moderately Weathered	WM	Discoloration throughout. Weaker minerals such as feldspar decomposed. Strength somewhat less than fresh rock but cores cannot be broken by hand or scraped by knife. Texture preserved.
Highly Weathered	WH	Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with knife. Core stones present in rock mass. Texture becoming indistinct but fabric preserved.
Completely Weathered	WC	Minerals decomposed to soil but fabric and structure preserved (Saprolite). Specimens easily crumbled or penetrated.
Residual Soil	RS	Advanced state of decomposition resulting in plastic soils. Rock fabric and structure destroyed. Large volume change.

- Discontinuity Classification. For foundation purposes, the nature of rock discontinuities may be expressed in terms of their width, the degree of weathering of rock contact faces, and the character of infilling materials. Describe spacing of discontinuities as close, wide, etc., in accordance with Table 2-6. In describing structural features, describe rock mass as thickly bedded or thinly bedded, in accordance with Table 2-6. Depending on project requirements, identify the form of joint (stepped, smooth, undulating, planar, etc.), its dip (in degrees), its surface (rough, smooth, slickensided), its opening (giving width), and its filling (none, sand, clay, breccia, etc.).

Table 2-6 Discontinuity Spacing

Description for Structural Features: Bedding, Foliation, or Flow Banding	*Spacing*	*Description for Joints, Faults or Other Fractures*
Very thickly (bedded, foliated, or banded)	More than 6 feet	Very widely (fractured or jointed)
Thickly	2 - 6 feet	Widely
Medium	8 - 24 inches	Medium

[10] ISRM Working Party, Suggested Methods of the Description of Rock Masses, Joints and Discontinuities, International Society of Rock Mechanics Second Draft of Working Party, Lisbon, 1975.

Thinly	2 1/2 - 8 inches	Closely
Very thinly	3/4 – 2 1/2 inches	Very closely
Description for Micro-Structural Features: Lamination, Foliation, or Cleavage	***Spacing***	***Description for Joints, Faults or Other Fractures***
Intensely (laminated, foliated, or cleaved)	1/4 - 3/4 inch	Very Close
Very intensely	Less than 1/4 inch	

In addition to the strength of rock material, and the spacing and nature of discontinuities, the quality of a rock mass for foundation purposes is affected by the orientation of discontinuities with respect to the applied load. A rock mass is said to contain adversely oriented discontinuities, if under the action of the resultant foundation load the minimum resistance to sliding occurs when the sliding surface is considered to be along these discontinuities.

- Color and Grain Size. Describe with respect to basic colors on rock color chart[11]. Use the following term to describe grain size:
  - For Igneous and Metamorphic Rocks:
    - Coarse-grained - grain diameter >> 5mm
    - Medium-grained - grain diameter 1 - 5mm
    - Fine-grained - grain diameter << 1mm
    - Aphanitic - grain size is too small to be perceived by unaided eye
    - Glassy - no grain form can be distinguished.
  - For Sedimentary Rocks
    - Coarse-grained - grain diameter >> 2mm
    - Medium-grained - grain diameter = 0.06 - 2mm
    - Fine-grained - grain diameter = 0.002 - 0.06mm
    - Very fine-grained - grain diameter << 0.002mm
  - Use 10X hand lens if necessary to examine rock sample.
- Hardness Classification. Describe as very soft, soft, etc. in accordance with Table 2-7, which shows range of strength values of intact rock associated with hardness classes.

Table 2-7 Hardness Classification of Intact Rock

****CLASS****	***HARDNESS***	***FIELD TEST***	***APPROXIMATE RANGE OF UNIAXIAL COMPRESSION STRENGTH kg/cm.^{2} (tons/ft.^{2})***
I	Extremely hard	Many blows with geologic hammer required in breaking an intact specimen.	>2000

[11] Geological Society of America, Rock Color Chart.

II	Very hard	Hand held specimen breaks with hammer end of pick under more than one blow.	2000- 1000
III	Hard	Cannot be scraped or peeled with knife, hand held specimen can be broken with single moderate blow with pick.	1000 - 500
IV	Soft	Can just be scraped or peeled with knife. Indentations 1 mm to 3 mm show in specimen with moderate blow with pick.	500 - 250
V	Very soft	Material crumbles under moderate blow with sharp end of pick and can be peeled with a knife, but is too hard to hand-trim for triaxial test specimen	250 - 10

∞ Geological Classification. Identify the rock by geologic name and local name (if any). A simplified classification is given in Table 2-8. Identify subordinate constituents in rock sample such as seams or bands of other type of minerals, e.g., dolomitic limestone, calcareous sandstone, sandy limestone, mica schist.

Table 2-8 Simplified Rock Classification

Texture	Group	Hardness	Characteristics	Further characteristics	Rock
Foliated			Very fine-grained; splits along thin planes		Slate
Foliated			Metallic reflection; splits into slabs and flakes or thin semitransparent sheets		Schist
Foliated			Contains streaks or bands of light and dark minerals; breaks to bulky, angular fragments		Gneiss
Very Fine-Grained	Frothy		Light colored; lightweight; easily crushed		Pumice
Very Fine-Grained	Frothy		Dark colored; cindery		Scoria
Very Fine-Grained	Glassy		Light colored; massive; extremely hard		Quartz
Very Fine-Grained	Glassy		Dark colored; may have some gas bubbles		Obsidian
Very Fine-Grained	Stony	Soft	No acid reaction	Earthy; clay odor; platy	Shale
Very Fine-Grained	Stony	Soft	No acid reaction	May have small pieces of glass; low density	Tuff
Very Fine-Grained	Stony	Soft	Acid reaction	Sugary appearance	Marble
Very Fine-Grained	Stony	Soft	Acid reaction	Dull and massive	Limestone
Very Fine-Grained	Stony	Hard	Waxy; very hard; weathers to soft white		Chert
Very Fine-Grained	Stony	Hard	Dull; may contain some gas bubbles or visible crystals	Light colored	Felsite
Very Fine-Grained	Stony	Hard	Dull; may contain some gas bubbles or visible crystals	Dark colored	Basalt
Very Fine-Grained	Stony	Hard	Sandy; mostly one mineral (quartz)	Gritty sandpaper feel	Sandstone
Very Fine-Grained	Stony	Hard	Sandy; mostly one mineral (quartz)	Sugary; not gritty	Quartzite
Coarse-Grained		Hard	Sandy; mostly one mineral (quartz)	Gritty sandpaper feel	Sandstone
Coarse-Grained		Hard	Sandy; mostly one mineral (quartz)	Sugary; not gritty	Quartzite
Coarse-Grained		Hard	Mixed minerals; salt-and-pepper appearance	Light colored	Granite
Coarse-Grained		Hard	Mixed minerals; salt-and-pepper appearance	Dark colored	Gabbro-diorite
Coarse-Grained		Hard	Fragmental; appearance of broken concrete		Conglomerate
Coarse-Grained		Soft	Fragmental; may contain small pieces of glass	Low density	Tuff
Coarse-Grained		Soft	Acid reaction	Sugary appearance	Marble
Coarse-Grained		Soft	Acid reaction	Shell fragments	Limestone

2.4.2. Classification by Field Measurements and Strength Tests

2.4.2.1. Classification by Rock Quality Designation and Velocity Index

The Rock Quality Designation (RQD) describes the quality of rock based on the degree and amount of natural fracturing. In addition to the percent recovery, the RQD should be recorded for each core run. RQD is a modified core recovery, which is best used on NX size core or larger. Determine the RQD by summing the lengths of all core pieces equal to or longer than 4 inches (100 mm) (ignoring fresh irregular breaks caused by drilling) and dividing that sum by the total length of the core run.

Mathematically this is expressed as:

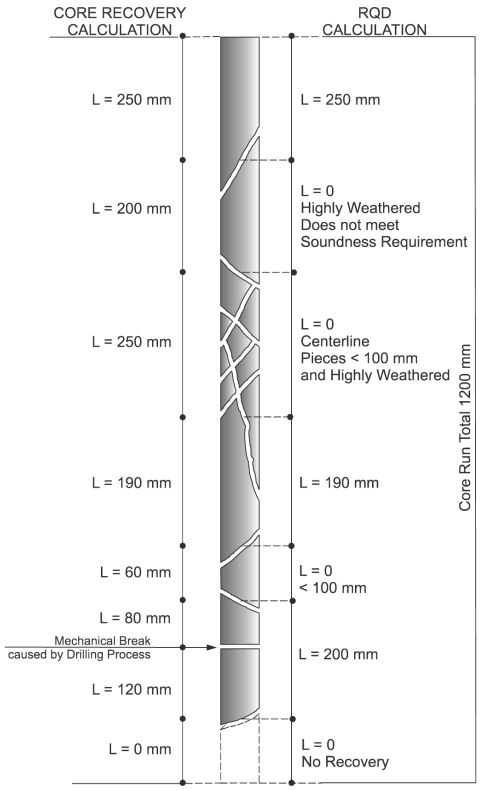
Equation 2-1: $$RQD = \frac{\sum_{i=1}^{n} L_i}{L_t}$$

Where

- L_i = length of a given core sample recovered piece greater than or equal to 4" (101.6 mm) in length
- n = total number of recovered pieces with length greater than or equal to 4".
- L_t = total length of core sample, in. or mm

The resultant is multiplied by 100 to get RQD in percent. It is necessary to distinguish between natural fractures and those caused by the drilling or recovery operations. The fresh, irregular breaks should be ignored and the pieces counted as intact lengths. Depending on the engineering requirements of the project, breaks induced along highly anisotropic planes, such as foliation or bedding, may be counted as natural fractures. An example of an actual RQD run, along with the calculations, is shown in Figure 2-7.

Figure 2-7 Sample Core Runs and Calculations



$$Core\ Recovery, CR = \frac{Total\ length\ of\ rock\ recovered}{Total\ core\ run\ length}$$

$$CR = \frac{(250+200+250+190+60+80+120)\,mm}{1{,}200\,mm}$$

$$CR = 96\%$$

$$RQD = \frac{\sum Length\ of\ sound\ pieces > 100\,mm}{Total\ core\ run\ length}$$

$$RQD = \frac{(250+190+200)\,mm}{1{,}200\,mm} * 100\%$$

$$RQD = 53\%$$

A qualitative relationship between RQD, velocity index and rock mass quality is presented in Table 2-9[12].

Table 2-9 Engineering Classification for *In situ* Rock Quality

RQD %	***VELOCITY INDEX***	***ROCK MASS QUALITY***
90 - 100	0.80 - 1.00	Excellent
75 - 90	0.60 - 0.80	Good
50 - 75	0.40 - 0.60	Fair
25 - 50	0.20 - 0.40	Poor
0 - 25	0 - 0.20	Very Poor

The velocity index is defined as the square of the ratio of the field compressional wave velocity to the laboratory compressional wave velocity. The velocity index is typically used to determine rock quality using geophysical surveys.[13]

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[12] Coon, J.H. and Merritt, A.H., Predicting *In situ* Modulus of Deformation Using Rock Quality Indexes, Determination of the *In situ* Modulus of Deformation of Rock, STP 457, ASTM 1970.

[13] For further information see Deere, D.U., Hendron A.J. Jr., Patton, F.D. and Cording, E.J., Design of Surface and Near Surface Construction in Rock, Proceedings, Eighth Symposium on Rock Mechanics, MN., 1966.

2.4.2.2. Classification by Strength and Structural Characteristics

Standard methods of testing rock in the laboratory for structural characteristics are only for intact rock. See Table 2-10 for testing procedures. Behavior of *in situ* rock, which typically has bedding planes, joints, etc., and may contain discontinuities filled with weaker material, is found to be very different from that of intact rock. *In situ* tests of joint strengths and compressibility are, therefore, more appropriate. Factors, which correlate intact rock sample parameters to realistic field parameters, are RQD (Rock Quality Designation) or the ratios of field values to laboratory values of compression or shear wave velocities.

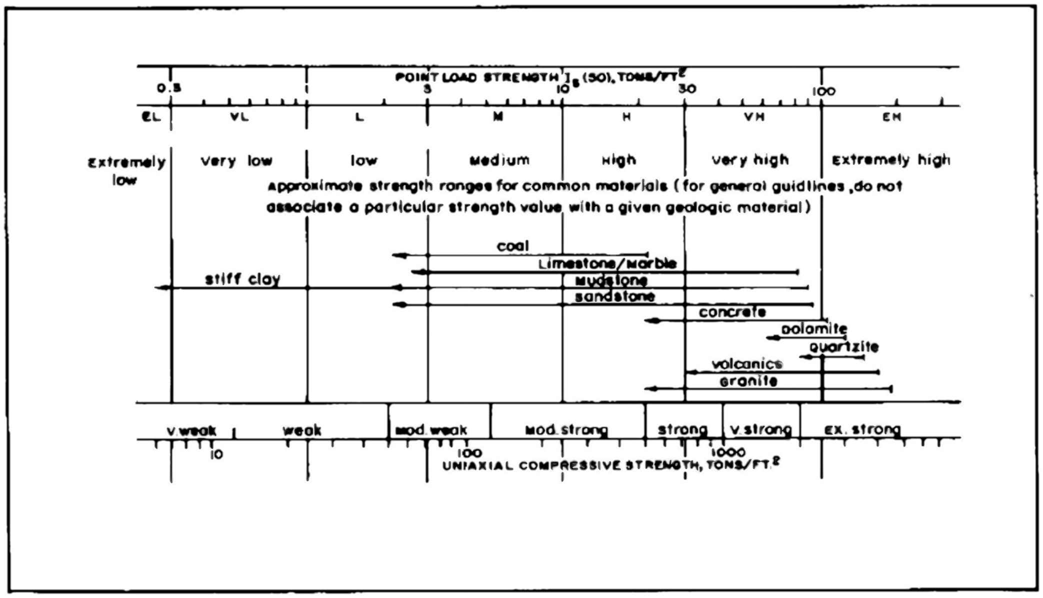
Table 2-10 Test Procedures for Intact Rock

Test	*Size of Sample and Other Notes for Test*
Unconfined compressive strength of core specimen (ASTM D2938)	Right circular cylinder with length to diameter ratio of 2 to 2.5, and a diameter not less than 2 inches. Describe the strength of intact sample tested as weak, strong, etc., in accordance with Figure 2-8[14].
Elastic constants of core specimen (ASTM D3148)	Right circular cylinder with length to diameter ratio of 2 to 2.5.
Direct tensile strength of intact rock core specimen (ASTM D2936)	Right circular cylinder with length to diameter ratio of 2 to 2.5.
Triaxial strength of core specimen (ASTM D2664)	Right circular cylinder with length to diameter ratio of 2 to 2.5.
Dynamic properties of core specimen at small strains (ASTM D2845)	Variable, dependent on properties of specimen and test apparatus.
Point Load Strength	Describe the point load strength of specimen tested as low, medium, etc. in accordance with Figure 2-8. Point load strength tests are sometimes performed in the field for larger projects where rippability and rock strength are critical design factors. This simple field test can be performed on core samples and irregular rock specimens. The point load strength index is defined as the ratio of the applied force at failure to the squared distance between loaded points. This index is related to the direct tensile strength of the rock by a proportionality constant of 0.7 to 1.0 depending on the size of sample.[15]

[14] Broch, E. and Franklin, J.A., The Point Load Strength Test, International Journal of Rock Mechanics and Mining Science, Pergamon Press, Vol. 9, pp 669 - 697, 1972.

[15] Useful relationships of point load tensile strength index to other parameters such as specific gravity, seismic velocity, elastic modulus, and compressive strength are given in DiAndrea, D.V., Fischer, R.L., and Fogelson, D.E., Prediction of Compressive Strength from Other Rock Properties, U.S. Bureau of Mines, Report Investigation 6702, p 23, 1967.

Figure 2-8 Strength Classification



2.4.2.3. Classification by Durability

Short-term weathering of rocks, particularly shakes and mudstones, can have a considerable effect on their engineering performance. The weatherability of these materials is extremely variable, and rocks that are likely to degrade on exposure should be further characterized by use of tests for durability under standard drying and wetting cycle[16]. If, for example, wetting and drying cycles reduce shale to grain size, then rapid slaking and erosion in the field is probable when rock is exposed[17].

2.4.2.4. Aggregate Tests

While intended for roadway construction, asphalt, and concrete aggregates, there are several standard tests that provide methods for measuring certain aspects of rock quality (see Table 2-11).

Table 2-11 Test Procedures for Aggregate

Test	*Applicability to Rock Cores*
Weathering resistance. (ASTM C88)	Applicable in principle, can be used directly by fracturing core.
Visual evaluation of rock quality. (ASTM C295)	Direct.
Resistance to freezing. (ASTM C666)	Applicable in principle; but only with significant procedure changes.
Hardness. (ASTM C851)	Direct.

[16] Franklin, J.A., Broch, E., and Walton, G., Logging Mechanical Character of Rock, Transactions, Institution of Mining and Metallurgy, A 80, A1-A9, 1971.

[17] Underwood, L.B., Classification and Identification of Shales, Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM6, 1962.

2.5. Special Materials and Difficult Soils

Many types of soils create special problems in design and construction, especially those that experience large changes in volume. An overview of some of these soils is shown in Table 2-12. Other types of special materials are shown below.

Table 2-12 Problem Soils and Conditions[18]

Problem Soils	
Soil	Description
Organic	Colloids or fibrous materials such as peats, organic silts, and clays of many estuarine, lacustrine, or fluvial environments are generally weak and will deform excessively under load. These soils are usually not satisfactory for supporting even very light structures because of excessive settlements.
Normally consolidated clays	Additional loads imposed on soil consolidated only under the weight of the existing environment will cause significant long-term settlements, particularly in soft and organic clays. These clays can be penetrated several centimeters by the thumb. The magnitude and approximate rate of settlement should be determined by methods described in Section III, Chapter 3, in order to determine acceptability of settlements for the function and characteristics of the structure. Bottoms of excavations may heave and adjoining areas settle unless precautions are taken to prevent such movement.
Sensitive clays	The ratio of undisturbed to remolded strength is the sensitivity of clay. Clays having remolded strengths 25% or less of the undisturbed strength are considered sensitive and subject to excessive settlement and possible catastrophic failure. Such clays preconsolidated by partial desiccation or erosion of overlying soil may support shear stresses caused by foundation loads if these loads are well within the shear strength of the clay.
Swelling and shrinking clays and shales	Clays, especially those containing montmorillonite or smectite, expand or contract from changes in water content and are widely distributed throughout the United States and the world. Clay shales may swell significantly following stress relief as in a cut or excavation and following exposure to air. Foundations in these soils may have excessive movements unless the foundation soil is treated or provisions are made in the design to account for these movements or swell pressures developed in the soil on contact with moisture. Refer to 7.5, for details on analysis of heave and shrinkage.
Collapsible soils	The open, porous structure of loosely deposited soil such as silty clays and sands with particles bonded with soluble salts may collapse following saturation. These soils are often strong and stable when dry. Undisturbed samples should be taken to accurately determine the *in situ* density. Refer to 7.5, for details on settlement analysis.
Loose granular soils	All granular soils are subject to some densification from vibration, which may cause significant settlement and liquefaction of soil below the water table; however, minor vibration, pile driving, blasting, and earthquake motion in loose to very loose sands may induce significant settlement. Limits to potential settlement and applicable densification techniques should be determined. Refer to 7.4 for analysis of dynamic

[18] Based on information from the Canadian Foundation Engineering Manual, 2nd edition.

	settlements in these soils.
Glacial tills	Till is usually a good foundation soil except boulders and soft layers may cause problems if undetected during the field investigation.
Fills	Unspecified fills placed randomly with poor compaction control can settle significantly and provide unsuitable foundation soil. Fills should usually be engineered granular, cohesive materials of low plasticity index < 12 and liquid limit < 35. Suitable materials of the Unified Soil Classification System include GW, GM, GC, GP, SW, SP, SM, SC, and CL soils. Compaction beneath structures to ≥ 92% of optimum density for cohesive fill or 95% for cohesionless fill using ASTM Standard Test Methods D 1557 has provided highly successful constructability and in-service performance.
Problem Conditions	
Condition	Description
Meander loops	Soils that fill abandoned waterways are usually weak and cut-offs highly compressible. The depth of these soils should be determined and estimates made of potential settlement early in design to allow time for development of suitable measures for treating the soil or accommodating settlement.
Landslides	Potential landslides are not easily detected, but evidence of displacement such as bowed trees and tilted or warped strata should be noted. Sensitive clays and cutting action of eroding rivers significantly increase the risk of landslides. Slopes and excavations should be minimized, seasonal variations in the local water table considered in the design, and suitable arrangements for drainage provided at the top and toe of slopes.
Kettle holes	The retreating continental ice sheet left large blocks of ice that melted and left depressions, which eventually filled with peat or with soft organic soils. Lateral dimensions can vary from a few to several hundred feet. Depths of kettle holes usually do not exceed 40% of lateral dimensions and can sometimes be identified as shallow surface depressions.
Mined areas and sinkholes	Voids beneath the surface soil may lead to severe ground movements and differential settlement from subsidence or caving. Sinkholes are deep depressions formed by the collapse of the roofs of underground caverns such as in limestone. Maps of previous mined areas are helpful when available. Published geological data, nondestructive *in situ* tests and past experience help indicate the existence of subsurface cavities. Investigations should be thorough to accurately determine the existence and location of any subsurface voids.
Lateral soil distortions	Lateral distortions are usually not significant, but can occur in highly plastic soils near the edge of surface loads. These distortions can adversely affect the performance of foundations of structures and embankments. Driven piles can cause large lateral displacements and excessive pressures on retaining walls.
Downdrag	Compression of fills or consolidation of soft soil adjacent to wall footings or piles cause downdrag on the footing or pile. This leads to substantial loads at the base of the foundation that can exceed the bearing capacity of the underlying soil supporting the footing or pile. Failure of the foundation can occur with gross distortion.
Vibrations	Cohesionless soil, especially loose sands and gravels, can densify and settle when subject to machine vibration, blasts, and earthquakes. Distortion with negligible

volume change can occur in loose, saturated sands due to liquefaction. Low-level sustained vibration can densify saturated sands.

2.5.1. Expansive Soils

2.5.1.1. General

These soils have great potential for volume change with change in water content. Clay soils with high colloidal contents, such as montmorillonite, found in regions where high rainfalls are followed by long periods of little or no rainfall, exhibit high volume increases and decreases. The geographic distribution and methods of identification and classification are given in § 2. Laboratory test procedures for determining the swell potential are described in 7.7.10.

2.5.1.2. Foundation Problems

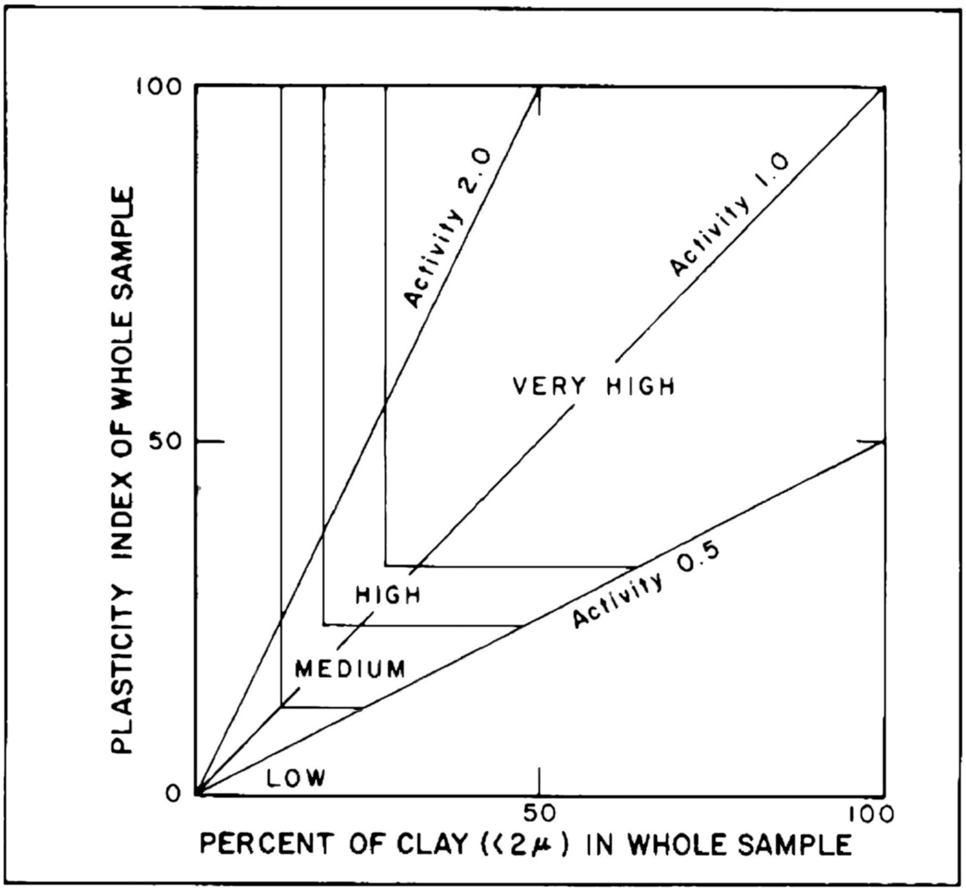
Problems associated with swelling and shrinking soils are total and differential settlement or heave, excessive pressures on retaining structures, and cracking of embankments.

2.5.1.3. Identification and Classification

Figure 2-9[19] shows a method based on Atterberg limits and grain size for classifying expansive soils. Activity of clay is defined as the ratio of plasticity index and the percent by weight finer than two microns (2μ). The swell test 7.8 is used for estimating the swell potential.

[19] The Canadian Geotechnical Society, Shallow Foundations, Part 2, Canadian Foundation Engineering Manual, 1978.

Figure 2-9 Volume Change Potential for Clay Soils



2.5.1.4. Calculation of Soil Movements

Guidance for calculation of soil movements in expansive soil is given in 7.5.

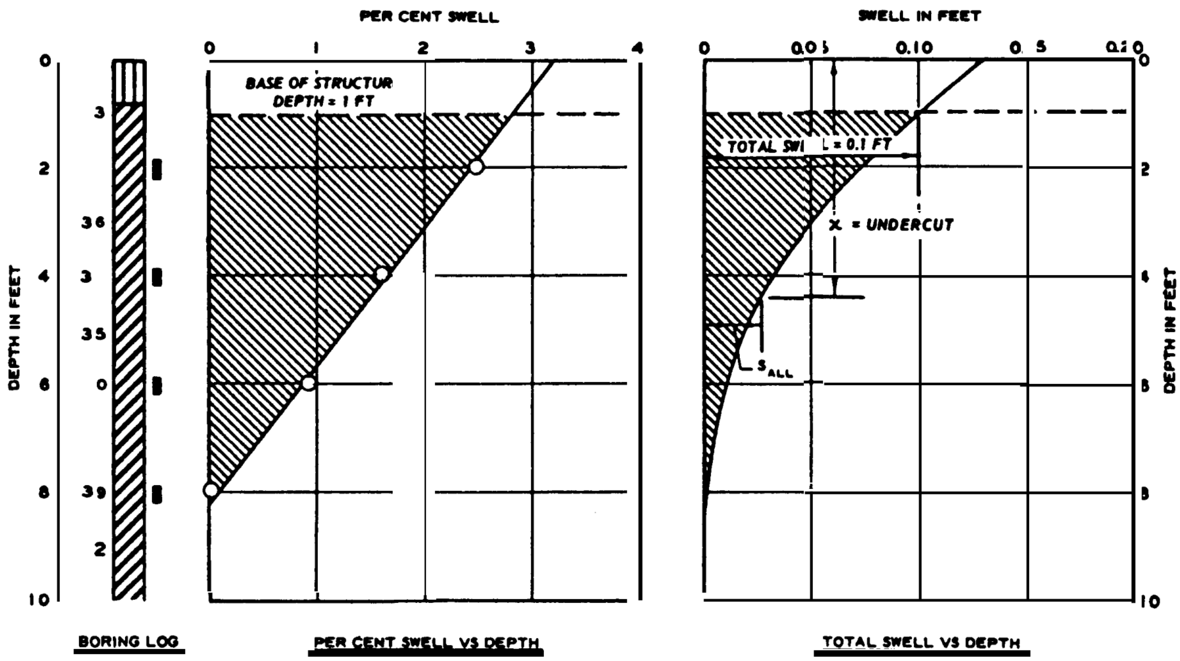
2.5.1.5. Suitable Foundations

Suitable foundations can be provided by removing and replacing the undesirable soil, isolating the structural element of foundation from the soil, designing a structure capable of resisting heave pressures, or preventing heave from occurring by prewetting. Membranes and surficial grading can accomplish prevention of water access.

Methods of estimating heave and procedures for treatment of heave are given in 5.4. Estimating swell using the South African method is illustrated in Figure 2-10. The method is usually conservative. Swelling pressures are usually relieved with little displacement. It is advantageous to isolate the floor from the soil by using collapsible cardboard forms or leaving a similar void space. Lubricating deep foundation shafts or installing them in pre-bored holes filled with vermiculite or bentonite achieves further isolation. For backfill of retaining structures, swelling soils should not be used. See Table 2-13 for recommended foundation systems and Table 2-14 for remedial measures for existing foundations on swelling soils[20].

[20] For further guidance, see Chen, F.H., Foundations on Expansive Soils, Elsevier Scientific Publishing Co., New York, 1975.

Figure 2-10 Estimating Swell Using the South African Method[21]



NOTE: FIGURES TO LEFT OF BORING LOG ARE NATURAL WATER CONTENTS.

▮ DENOTES LOCATION OF SAMPLE FOR SWELL TESTS.

(A) PROCEDURE FOR ESTIMATING TOTAL SWELL

1. ON BASIS OF BORING LOG PROFILE SELECT SAMPLES AT INTERVALS FOR SWELL TESTS.
2. LOAD SPECIMENS IN CONSOLIDOMETER TO OVERBURDEN PRESSURE PLUS WEIGHT OF STRUCTURE; ADD WATER AND OBSERVE SWELL.
3. COMPUTE SWELL IN TERMS OF PER CENT OF ORIGINAL SPECIMEN HEIGHT AND PLOT VS DEPTH.
4. COMPUTE TOTAL SWELL WHICH IS EQUAL TO AREA ENCOMPASSED BY PER CENT SWELL VS DEPTH CURVE. FOR EXAMPLE, USING CURVES SHOWN ABOVE:

$$\text{TOTAL SWELL} = 1/2 \times (8.2 - 1.0) \times 2.8/100 = 0.10 \text{ FT}$$

(B) PROCEDURE FOR ESTIMATING AMOUNT OF UNDERCUT (x) NECESSARY TO REDUCE TOTAL SWELL TO AN ALLOWABLE VALUE (S_{ALL})

1. FROM PER CENT SWELL VS DEPTH RELATIONSHIP, COMPUTE AND PLOT TOTAL SWELL VS DEPTH RELATIONSHIP.
2. FOR A GIVEN VALUE OF S_{ALL}, THE AMOUNT OF UNDERCUT IS READ DIRECTLY OFF THE TOTAL SWELL-DEPTH CURVE.

NOTE: UNDERCUT MATERIAL SHOULD BE REPLACED BY INERT MATERIAL OR ELSE THE BASE OF THE STRUCTURE SHOULD BE LOWERED TO THE DEPTH OF THE REQUIRED UNDERCUT.

[21] Van der Merwe, The Prediction of Heave from the Plasticity Index and Percentage of Clay Fraction, The Civil Engineer in South Africa, June, 1964.

Table 2-13 Recommended Foundations for Expansive Soils[22]

Predicted Differential Movement, inches	Effective Plasticity Index, $\overline{PI}$	Foundation System	Remarks		
1/2	<15	Shallow individual Continuous wall Strip	Lightly loaded buildings and residences.		
		Reinforced and stiffened thin mat	Residences and lightly loaded structures; on-grade 4- to 5-in. reinforced concrete slab with stiffening beams; maximum free area between beams 400 ft^2; 1/2 percent reinforcing steel; 10- to 12-in.-thick beams; external beams thickened or deepened, and extra steel stirrups added to tolerate high edge forces as needed; dimensions adjusted to resist loading. Beams positioned beneath corners to reduce slab distortion.		
			Type of Mat	Beam Depth, in.	Beam Spacing, ft
1/2 to 1	15 to 25		Light	16 to 20	20 to 15
1 to 2	26 to 40		Medium	20 to 25	15 to 12
2 to 4	>41		Heavy	25 to 30	15 to 12
No limit		Thick, reinforced mat	Large, heavy structures; mats usually 2 ft or more in thickness.		
No limit		Deep foundations, pile or drilled shaft	Foundations for any light or heavy structure; grade beams span between piles or shafts 6 to 12 in. above ground level; suspended floors or on-grade slabs isolated from grade beams and walls. Concrete drilled shafts may be underreamed or straight, reinforced, and cast in place with 3000-psi concrete of 6-in. slump.		

[22] Departments of the Army and Air Force, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), TM5-818-1/AFM88-3. Chapter 7, Washington, D.C., 1979.

Table 2-14 Remedial Measures for Existing Foundations on Swelling Soils

Measure	Description
Drainage	Slope ground surface (positive drainage) from structure; add drains for downspouts and outdoor faucets in areas of poor drainage, and discharge away from foundation soil; provide subdrains if perched water tables or free flow of subsurface water are problems; provide flexible, watertight utility connections.
Moisture stabilization (maintenance of constant moisture whether at high or low levels)	Remove natural swelling soil and recompact with impervious, nonswelling backfill; install vertical and/or horizontal membranes around the perimeter; locate deep-rooted vegetation outside of moisture barriers; avoid automatic sprinkling systems in areas protected with moisture barriers; provide a constant source of moisture if a combination of swelling/shrinking soils is occurring; thoroughly mix 4 to 8 percent lime into soil to reduce potential for swell or pressure-inject line slurry around the perimeter of the structure.
Superstructure adjustments	Free slabs from foundation by cutting along foundation walls; provide slip joints in interior walls and door frames; reinforce masonry and concrete block walls with horizontal and vertical tie bars or reinforced concrete beams; provide fanlights over doors extended to the ceiling.
Spread footings and deep foundation adjustments	Decrease footing size; underpin with deep shafts; mudjack using a cement grout; reconstruct void beneath grade beams; eliminate mushrooms at top of shafts; adjust elevation by cutting the top of the shaft or by adding shims; increase footing or shaft spacing to concentrate loading forces and to reduce angular distortion from differential heave between adjacent footings and shafts.
Continuous wall foundation adjustments	Provide voids beneath portions of wall foundation; posttension; reinforce with horizontal and vertical tie bars or reinforced concrete beams.
Reinforced and stiffened slab-on-grade adjustments	Mudjack using a cement grout; underpin with spread footings or shafts to jack up the edge of slabs,

2.5.1.6. Design guidelines

Mat foundations are usually appropriate if expansive soil extends to great depths that preclude economic use of drilled piers founded in a constant moisture zone. In cases where the potential heave is estimated at one inch or less, continuous wall footings and individual spread footings may be used in conjunction with a slab on grade. Ribbed mats (slab on grade with thickened edge and integral interior beams) may be used instead of continuous wall footings.

Deep foundations (e.g., drilled piers) should extend below the active zone of swelling (typically 20 feet). Drilled piers are belled to provide anchorage to resist uplift forces, and reinforcement is provided to carry uplift tensile force. Using the smallest appropriate shaft diameter can minimize uplift forces. In computing magnitude of uplift use an adhesive factor of 1.0 (i.e. $C_a = c$).

2.5.2. Collapsing Soils

2.5.2.1. Characteristics

Collapsible soil usually consists of cohesive silty sands with a loose structure or large void ratio. A common feature of these soils is that these loose bulky grains are held together by capillary stresses. The chemical bonding of particles with soluble compounds such as calcareous or ferrous salts usually causes the cohesion. Examples of soils exhibiting this behavior are loess, weakly cemented sands and silts where cementing agent is soluble (e.g., soluble gypsum, halite, etc.) and certain granite residual soils. Deposits of collapsible soils are usually associated with regions of moisture deficiency. Collapse occurs when the bonds between particles are dissolved. This occurs when moisture content is increased even without increase in external loads. Guidance for calculation of settlement in collapsible soil is given in 7.5. Other characteristics include:

- Loss of strength when wetted.
- Differential settlement.
- Low density.
- Moisture Sensitive.
- Gypsum/Anhydrite often present.

2.5.2.2. Identification and Classification

Detailed geologic studies could identify potentially collapsible soils. Figure 2-11[23] provides guidance for identifying the potential for collapse for clayey sands and sandy clays found in the western United States. For cemented soils and nonplastic soils, criteria based on consolidometer tests are more applicable as illustrated in Figure 2-12[24]. The potential for collapse is also evaluated in the field by performing standard plate load tests (ASTM D1194, Bearing Capacity of Soil for Static Load on Spread Footings) under varied moisture environments.[25]

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[23] Holtz, W.G. and Gibbs, H.J., Research Related to Soil Problems of the Arid Western United States, Proceedings of the Third Panamerican Conference on Soil Mechanics and Foundation Engineering, Caracas, 1967.

[24] Jennings, J.E. and Knights, K., A Guide to Construction on or With Materials Exhibiting Additional Settlements Due to Collapse of Grain Structure, Proceedings of the Sixth Regional Conference for Africa on Soil Mechanics and Foundation Engineering, pp 99-105, 1975, and Knight, K., The Origin and Occurrence of Collapsing Soil, Proceedings of the Third Regional Conference of Africa on Soil Mechanics and Foundation Engineering, Vol. 1, pp 127-130, 1963.

[25] Beckwith, G.H., Experience with Collapsible Soil in the Southwest, ASCE Conference, Arizona Section, 1979.

Figure 2-11 Criterion for Collapse Potential (USBR)

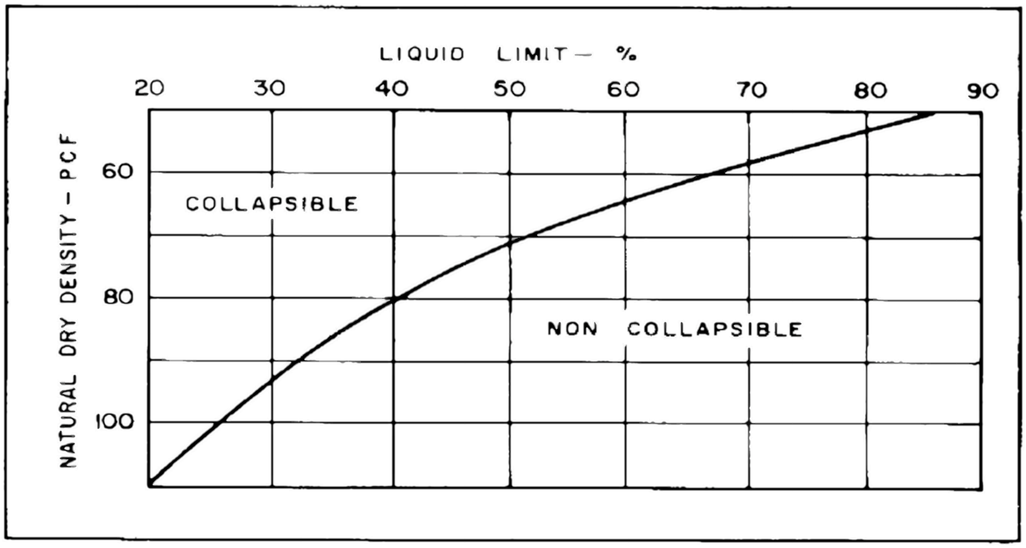
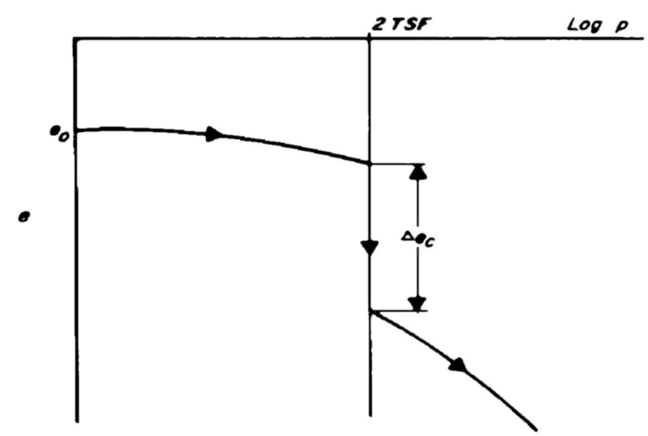


Figure 2-12 Typical Collapse Potential Test Results



Collapse Potential (CP) is defined as:

$$CP = \frac{\Delta e_c}{1 + e_o} \quad \text{or} \quad CP = \frac{\Delta Hc}{Ho}$$

Δe_c = Change in void ratio upon wetting

e_o = Natural void ratio

ΔHc = Change in height upon wetting

Ho = Initial height

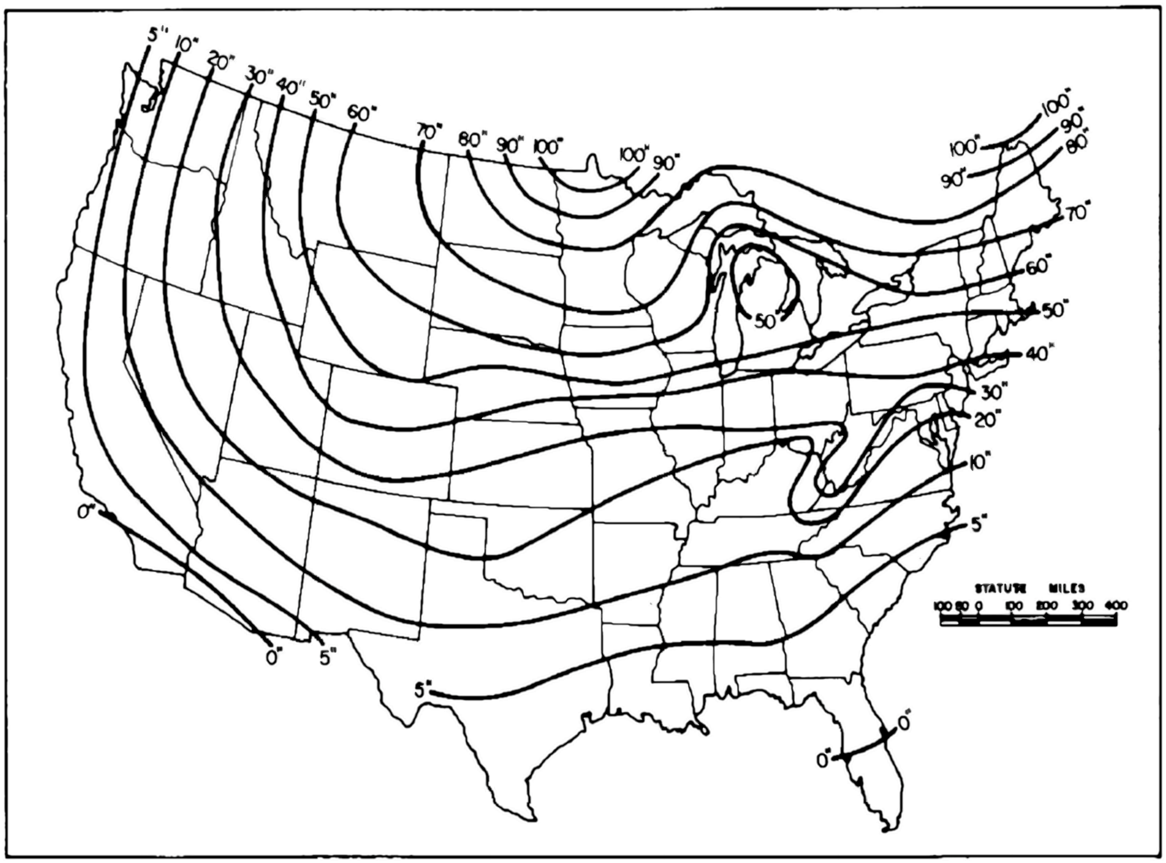
Collapse Potential Values

CP	Severity of Problem
0-1%	No problem
1-5%	Moderate trouble
5-10%	Trouble
10-20%	Severe Trouble
20%	Very Severe Trouble

2.5.3. Permafrost and Frost Penetration

- Characteristics. In non-frost susceptible soil, volume increase is typically 4% (porosity 40%, water volume increase in turning to ice = 10%, total heave = 40% x 10% = 4%). In susceptible soil heave is much greater as water flows to colder zones (forming ice lenses). The associated loss of support upon thaw can be more detrimental to structure than the heave itself.
- Classification. Silts are the most susceptible to frost heave. Soils of types SM, ML, GM, SC, GC, and CL are classified as having frost heave potential.
- Geography. Figure 2-13[26] may be used as a guide for estimating extreme depth of frost penetration in the United States.

Figure 2-13 Extreme Frost Penetration (in inches) Based upon State Average



2.5.4. Limestone and Related Materials

- Characteristics. Limestone, dolomite, gypsum and anhydrite are characterized by their solubility and thus the potential for cavity presence and cavity development. Limestones are defined as those rocks composed of more than 50% carbonate minerals of which 50% or more consist of calcite and/or aragonite. Some near shore carbonate sediments (also called limestone, marl, chalk) could fit this description. Such sediments are noted for erratic degrees of induration, and thus variability in load supporting capacity and uncertainty in their long-term performance under sustained loads. The most significant limestone feature is its solubility. An extremely soluble one can be riddled with solution caves, channels, or other open, water, or clay filled features.

[26] National Oceanic and Atmospheric Administration, Environmental Data and Information Service, Asheville, NC.

- Identification. Geological reconnaissance, drilling, and other forms of bedrock verification may check presence of solution features. Geophyical techniques, including shallow seismic refraction, resistivity and gravimetry are often found to be valuable supplements.
- Karst Topography. In places such as Kentucky, Virginia, Pennsylvania, Tennessee, Indiana, California, Texas, and New Mexico, limestone is prone to being cavernous. Such leads to the following:
  - Uneven underground erosion leads to erratic depth and quality of "bedrock".
  - Erosion also leads to underground caverns and water flows.
  - Expansion of underground voids can lead to sinkholes.
- Calcareous Soils
  - Overview. Calcareous soils are some of the most challenging types of soils for the design and installation of piling. Because they frequently appear in areas where offshore oil is found (i.e., southeast Asia, the Persian Gulf, Australia, etc.), a great deal of research has been done on these soils. Because of the complex nature of these soils and the variable way in which they are formulated, their properties are complex and not as well quantified as other types of soils.
    - Definition and Origin. Living coral and coralline debris is generally found in tropical regions where the water temperature exceeds 20º C. Coral is a term commonly used for the group of animals which secrete an outer skeleton composed of calcium carbonate, and which generally grow in colonies. The term "coral reef" is often applied to large concentrations of such colonies that form extensive submerged tracts around tropical coasts and islands. In general, coralline soils deposited after the breakdown of the reef, typically by wave action, are thin (a few meters thick) and form a veneer upon cemented materials (limestones, sandstones, etc.). Calcareous soils are those that are composed of primarily sand size particles of calcium carbonate, which may be indurated to varying degrees. They can originate from biological processes such as sedimentation of skeletal debris and coral reef formation. They can also occur because of chemical precipitation of particles such as oolites. Because of their association with coral reefs, these soils appear mostly between the latitudes of 30EN and 30ES.
    - Geological Classification. Because the granular coralline and algal materials are derived from organisms which vary in size from microscopic shells to large coralheads several meters in diameter, the fragments are broadly graded and range in size from boulders to fine-grained muds. Similarly, the shape of these materials varies from sharp, irregular fragments to well-rounded particles. Geologists generally refer to corralline deposits as "biogenic materials". When cemented, they may be termed "reefrock," or "beachrock," or other names that imply an origin through cementation of particles into a hard, coherent material.
  - Important Properties of Calcareous Soils. Coralline deposits are generally poor foundation materials in their natural state because of their variability and susceptibility to solution by percolating waters, and their generally brittle nature. Coralline materials are often used for compacted fill for roads and light structures. Under loads, compaction occurs as the brittle carbonate grains fracture and consolidate. They can provide a firm support for mats or spread footings bearing light loads, but it is necessary to thoroughly compact the material before using it as a supporting surface. Heavy structures in coral

areas are generally supported on pile foundations because of the erratic induration. Predrilling frequently is required. The brittle, crushable nature of calcareous sands complicates the site investigation. This makes both the site investigation itself and a meaningful correlation of test data to actual soil properties difficult. However, there are some important soil properties to watch for.

- Carbonate Content. By definition, these soils have higher than average carbonate content. The calcareous soils most prone to difficulties have a carbonate content by weight above 50 percent. Problems are especially pronounced above 80 percent, where many pile driven into these soils have abnormally low capacities.
- Degree of Cementation and Grain Structure. The grain structure of these soils is highly variable due to the diverse nature of the soils. This variability is one of the most important factors in the unpredictability of these soils. This variability can manifest itself in the angularity, size, or void structure of the grains or other factors. Light cementation can lead to both low shaft friction and toe capacity.
- Bulk Density. Void ratios for calcareous sands can vary from 0.8 to 1.4 as opposed to 0.4 to 0.9 for noncarbonated sands. The tendency to voids of all sizes is one of the most difficult problems encountered with calcareous sands.
- Specific Gravity. This normally varies from 2.75 to 2.85 with these soils.
- Friction Angle. This is generally greater than 35 degrees and can be greater than 50 degrees. This may decline with increased confining pressure, and the surface friction angle may decrease with surface roughness.
- Because of extreme variability in engineering properties of natural coral formation, it is not prudent to make preliminary engineering decisions based on "typical properties." Unconfined compression strengths of intact specimens may range from 50 tons/ft^2 to 300 tons/ft^2, and porosity may range from less than 40% to over 50%.[27]

- Other characteristics include:
  - Solution cavities.
  - Extreme variations in porosity.
  - Void ratios in coral up to two.
  - Chimney-like sinkholes and collapse structures.
  - Slump failures, ravelling.
  - Rock settlement and consolidation.
  - Piles or bridging often required.
- Loading Response. Calcareous soils are highly compressible under pressure loading and are subject to softening under cyclic loading.
- Prediction of Driven Pile Capacity. Although a great deal of research has been done into the nature and engineering properties of calcareous sands, predicting the capacity of piles

[27] For more information see Sowers, F.G., Failure in Limestone in Humid Subtropics, Journal of the geotechnical engineering Division, ASCE, Vol. 101, No. GR8, 1975, and Way, S.D., Terrain Analysis - A Guide to Site Selection Using Aerial Photographic Interpretation, Dowden, Hutchinson & Ross, Inc., Stroudsburg, PA., 1973.

driven into these soils is highly speculative. Great care must be taken to verify that the soil can in fact support the installed foundation, both in the design process and especially in the field verification process. This should include extensive instrumentation and load testing of both indicator and production piling. With driven piles, a frequent occurrence with these soils is very easy driving; this can be due to temporary conditions or a more permanent condition, in which case remedial action must be taken. Blow count cannot be relied upon as a key jobsite control method.

  - Remediation. Once a situation has been encountered where the piles installed do not have the required capacity, remediation is necessary. One method, of course, is to drive the piles further. This can be uneconomical depending upon the situation. Another potential solution is to use drilled and grouted piles. These can significantly improve pile capacity by pressurizing the surrounding soil. However, as with any pile of this type comprehensive quality control during installation is essential.

2.5.5. Quick Clays

- Characteristics. Quick clays are characterized by their great sensitivity or strength reduction upon disturbance. All quick clays are of marine origin. Because of their brittle nature, collapse occurs at relatively small strains. Slopes in quick clays can fail without large movements[28]. Other characteristics include:
  - Severe loss of strength when disturbed by construction activities of seismic ground shaking.
  - Replacement of formation water containing dissolved salt with fresh water results in strength loss.
  - Produces landslide prone areas (such as Anchorage, Alaska).
- Geography. Generally confined to far north areas such as Eastern Canada, Alaska and Scandinavia.
- Identification. Quick clays are readily recognized by measured sensitivities greater than about 15 and by the distinctive, strain-softening shape of their stress-strain curves from strength or compressibility tests.

2.5.6. Sanitary Landfills

2.5.6.1. Introduction

Sanitary landfills are becoming the major sites for solid waste disposal. The geotechnical engineer's role in solid waste disposal includes:

(1) Evaluation of physical and chemical material properties;

(2) Design and supervision during construction of disposal facilities;

(3) Monitoring of facilities during operation to ensure satisfactory performance; and

(4) Evaluation of potential land uses after completion of disposal operations.

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[28] For further information see Anne, Q.A., Quick Clays and California: No Quick Solutions, Focus on Environmental Geology, Ronald Rark, ed., pp 140-145, 1973.

2.5.6.2. Composition of Material

The engineering properties of sanitary landfill are largely influenced by the composition of the refuse.

2.5.6.3. Settlement Characteristics

2.5.6.3.1. Subsidence of Refuse Fill Under Self-weight

The following mechanisms can lead to surface subsidence:

(1) Movement of particles into large voids;

(2) Biological decomposition of organics;

(3) Chemical reactions, including oxidation and combustion;

(4) Dissolving of soluble substances by percolating groundwater or leachate;

(5) Change in deformation properties with time;

(6) Plastic flow or creep.

The time-settlement relationship of subsidence under self-weight is analogous to the secondary compression of soils after a short period of pseudo-primary (mechanical) settlement typically 1 to 4 months long. Measurements indicate a coefficient of secondary compression ranging from 0.1 to 0.4. Thus, settlement of the fill under its own weight after completion can be estimated by:

Equation 2-2: $$(\Delta H) = HC_{\alpha} \log \frac{t_2}{t_1}$$

where:

- (ΔH) = settlement at time t_2 (length unit)
- H = thickness of fill (length unit)
- t_1 = time pseudo-primary (mechanical settlement) to occur after completion of fill
- t_2 = time after completion of fill
- C_{α} = coefficient of secondary compression (any mathematically compatible units acceptable)

2.5.6.3.2. Subsidence of Refuse Fills under External Loads

The time-settlement behavior of old refuse fills under an applied load is analogous to the behavior of peat. Primary settlements will likely occur as the load is applied. Secondary compression occurs over al long period of time and the amount of long-term settlement is determined by environmental conditions (i.e. humid environment is more conducive to decomposition) as well as the composition of the refuse. Reported primary compression indexes ($C_c/(1+e_o)$) ranged from 0.1 to 0.4 and the coefficient of secondary compression (C_{α}) from 0.02 to 0.07. These values are for fills that have undergone decomposition for some time prior to loading (10 to 15 years, typically). Higher compressibility is usually associated with high organic content and/or advanced degree of decomposition.

2.5.6.4. Construction over Sanitary Landfills

Any foundation investigation for a structure being built over a sanitary landfill should include the evaluation of the following potential problems:

(1) Differential settlement of floor slabs, walls, and utilities;

(2) Irregular subsidence due to highly variable composition;

(3) Corrosion of concrete foundations and pipe utilities;

(4) Generation of methane gas;

(5) Slope stability;

(6) Effect of construction on leachate control.

2.5.6.5. Methods of Treatment for Foundation Support

(1) Control and compaction during placement. Compaction and shredding of refuse as it is being placed in the landfill will greatly increase its suitability for later use.

(2) Proofrolling of fills and replacement of soft pockets with compacted soil will reduce irregular settlements.

(3) Use of surcharge fills where refuse is thick.

(4) Deep foundations founded below the refuse fills. If piles are used provisions must be made for the corrosive environment and possible damage during driving, as well as re-sealing any holes created in leachate cutoffs.

(5) Grouting of refuse fills to stabilize voids.

(6) Use of flexible connections for utilities[29].

2.5.7. Other Materials and Considerations

- Man-Made and Hydraulic Fills. Composition and density are the main concerns. Unless these can be shown to be non-detrimental to the performance of the foundation, bypassing with deep foundations, or removal and replacement are in order. Other characteristics include:
  - Found in coastal facilities, levees, dikes and tailings dams.
  - High void ratio.
  - Uniform gradation but variable grain size within same fill.
  - High liquefaction potential
  - Lateral spreading.
  - Easily eroded.
- Chemically Reactive Soils. For foundation construction, the main concerns usually are corrosion and gas generation. Corrosion potential is determined in terms of pH, resistivity, stray current activity, groundwater position, chemical analysis, etc; and a compatible foundation treatment, e.g., sulfate resistant concrete, lacquers, creosote, cathodic protection, etc., is prescribed. For gas concentration, organic matter content and field-testing for gas are usually performed. If gas generation is expected, some form of venting system is designed. The potential presence of noxious or explosive gases should be considered during the construction excavations and tunneling.

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[29] Further guidance on construction over sanitary landfills is given in Lutton, R.J., Regan, G.LL., and Jones, L.W., Design and Construction of Covers for Solid Waste Landfills, U.S. Army Waterways Experiment Station, August, 1979 and County of Los Angeles, Engineering-Science, Inc., Development of Construction and Use Criteria for Sanitary Landfills, An Interim Report, U.S. Department of Health, Education, and Welfare, October, 1968.

- Lateritic Soils. Lateritic soils are found in tropical climates throughout the world. Typical characteristics are as follows: [30]
    - Found where tropical rain forest and savannas are located.
    - Deep residual soil profile.
    - Shield and sedimentary cover outside shield in Central and South America, Central and West Africa, southeast Asia, and other parts of the world.
    - Loss of soil strength with time.
    - High void ratio and permeability.
    - Aggregate deterioration.
    - Variable moisture content.
    - Shrinkage cracks.
    - Easily compacts.
    - Shear Characteristics somewhere between sand and silt.
    - Landslide prone.
    - Depth of wetting affects slope stability.
    - Varied foundation conditions.
- Submarine Soils. Typical characteristics:[31]
    - Found in continental shelf deposits at water depths up to several hundred feet in places such as submarine canyons, turbidity flow, deltaic deposits and abyssal plain.
    - Distribution and physical properties of sand, silt, and clay may change with time and local geologic conditions.
    - Shelf deposits have few unique characteristics requiring modification of soil mechanics principals.
    - Local areas, such as the Gulf of Mexico, have weak, underconsolidated deposits.
    - Deep sea calcareous deposits have water contents up to 100% and shear strengths up to about 220 psf.
    - Deep-sea silty clays have average water contents of 100-200% and shear strength of 35-75 psf.
    - Deep-sea deposits are normally consolidated but near shelf deposits may be underconsolidated.

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[30] For further information see Gidigasu, M.D., Laterite Soil Engineering, Elsevier Scientific Publishing Co., 1976; Persons, S.B., Laterite Genesis, Location, Use, Plenum Press, 1970; U.S. Agency for International Development, Engineering Study of Laterite and Lateritic Soils in Connection With Construction of Roads, Highways and Airfields, Southeast Asia, 1969; U.S. Agency for International Development, Laterite, Lateritic Soils and Other Problem Soils of Africa, 1971; U.S. Agency for International Development, Laterite and Lateritic Soils and Other Problem Soils of the Tropics, 1975.

[31] For further information see Noorany, I. and Gizienski, S.F., Engineering Properties of Submarine Soils: State-of-the Art Review, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM5, 1970.

§ 3. Laboratory Tests and Index Properties of Soils

3.1. Overview of Laboratory Testing

Laboratory testing is an important element in foundation engineering. The complexity of testing required for a particular project may range from a simple moisture content determination to specialized strength testing. However, testing can be expensive and time consuming. The foundation engineer should recognize the project problems to be solved to optimize testing, particular strength and consolidation testing.

A sample of soil may be composed of soil grains, water and air. The soil grains are irregularly shaped solids, which are in contact with other adjacent soil grains. The weight and volume of a soil sample depends on the specific gravity of the soil grains (solids), the size of the area between soil grains (voids or pores) and the amount of void space filled with water. Common terms associated with weight-volume relationships are shown in Table 3-1. Of particular note is the void ratio (e) that is a general indicator of the relative strength and compressibility of the soil sample, i.e., low void ratios generally indicate strong, incompressible soils, and high void ratios may indicate weak, compressible soils.

Table 3-1 Soil Properties for Analysis and Design

Property	Symbol	Unit[(a)]	How Obtained	Direct Applications
Volume-weight Characteristics[(b)]				
Moisture Content	w	D	Direct from test	Classification and volume-weight relations.
Unit Weights	γ	FL^{-3}	Directly from test or from volume-weight relations	Classification and pressure computations.
Porosity	n	D	Computed from volume-weight relations	Parameters used to represent relative volume of voids with respect to total volume of soil or volume of solids.
Void Ratio	e	D	Computed from volume-weight relations	
Specific Gravity	G	D	Directly from test	Volume computations.
Plasticity Characteristics:				
Liquid Limit	LL	D	Directly from test	Classification and properties correlation.
Plastic Limit	PL	D	Directly from test	
Plasticity Index	PI	D	LL-PL	

Property	Symbol	Unit[(a)]	How Obtained	Direct Applications
Shrinkage limit	SL	D	Directly from test.	Classification and computation of swell.
Shrinkage index	SI	D	PL–SL	
Activity	A_c	D	$\frac{PI}{\%<2\text{ microns}}$	Identification of clay mineral.
Liquidity index	LI	D	$\frac{w - PL}{PI}$	Estimating degree of preconsolidation, and soil consistency.
Gradation Characteristics:				
Effective diameter	D_{10}	L	From grain size curve.	Classification, estimating permeability and unit weight, filter design, grout selection, and evaluating potential frost heave and liquefaction.
Percent grain size	D_{30} D_{60} D_{85}	L	From grain size curve.	
Coefficient of uniformity	C_u	D	$\frac{D_{60}}{D_{10}}$	
Coefficient of curvature	C_z	D	$\frac{(D_{30})^2}{(D_{10})x(D_{60})}$	
Clay size fraction	-	D	From grain size curve, % finer than 0.002 mm.	

Property	Symbol	Unit(a)	How Obtained	Direct Applications
Drainage Characteristics:				
Coefficient of permeability	k	LT^{-1}	Directly from permeability test or computed from consolidation test data.	Drainage, seepage, and consolidation analysis.
Capillary head	h_c	L	Directly from test.	Drainage and drawdown analysis.
Effective porosity	n_c	D	Directly from test for volume of drainable water.	
Consolidation Characteristics:				
Coefficient of compressibility	a_v	L^2F^{-1}	Determined from natural plot of e vs. p curve.	Computation of ultimate settlement or heave in consolidation analysis.
Coefficient of volume compressibility	m_v	L^2F^{-1}	$\frac{a_v}{1 + e}$	
Compression index	C_c	D	Determined from e vs. log p curve.	Computation of ultimate settlement or swell in consolidation analysis.
Recompression index	C_r	D		
Swelling index	C_s	D		

Property	Symbol	Unit(a)	How Obtained	Direct Applications
Coefficient of secondary compression	C	D	Determined from semilog time-consolidation curve.	Computation of time rate of settlement.
Coefficient of consolidation	c_v	L^2T^{-1}		
Preconsolidation pressure	p_c	FL^{-2}	Estimate from e vs. log p curve.	Settlement analysis.
Overconsolidation ratio	OCR	D	$\frac{p_c}{p_o}$	Basis for normalizing behavior of clay.
Shear Strength Characteristics:				
Apparent angle of shearing resistance	ϕ	A	Determined from Mohr circle plot of shear test data for total stress.	
Cohesion intercept	c	FL^{-2}		
Effective angle of shearing resistance	ϕ'	A	Determined from Mohr circle plot of effective stress shear test data (drained tests with pore pressure measurements).	
Effective cohesion	c'	FL^{-2}		

Property	Symbol	Unit[a]	How Obtained	Direct Applications
Unconfined compressive strength	q_u	FL^{-2}	Directly from test.	Analysis of stability, load carrying capacity of foundations, lateral earth load.
Undrained shear strength	s_u	FL^{-2}		
Sensitivity	S_t	D	$\frac{\text{undisturbed strength}}{\text{remolded strength}}$	
Modulus of elasticity or Young's modulus	E_s	FL^{-2}	Determined from stress-strain curve or dynamic test.	Computation of elastic settlement or rebound.
Compaction Characteristics:				
Maximum dry unit weight	γ_{max}	FL^{-3}	Determined from moisture-dry unit weight curve.	Compaction criterion.
Optimum moisture content	OMC	D		
Maximum and minimum density of cohesionless soils	$\gamma_{d\ max}$ $\gamma_{d\ min}$	FL^{-3} FL^{-3}	Directly from test.	
Characteristics of Compacted Samples:				
Percent compaction	-	D	$\frac{\gamma}{\gamma_{max}}$	Compaction control, properties correlation.
Needle penetration resistance	P_r	FL^{-2}	Directly from test.	Moisture control of compaction.

Property	Symbol	Unit(a)	How Obtained	Direct Applications
Relative density	D_r	D	Determined from results of max. and min. density tests.	Compaction control, properties correlation, liquefaction studies.
California Bearing Ratio	CBR	D	Directly from test.	Pavement design, compaction control.
Dynamic Characteristics:				
Shear modulus	G	FL^{-2}	Determined from resonant column, cyclic simple shear, ultrasonic pulse, or dynamic triaxial tests.	Analysis of foundation and soil behavior under dynamic loading.
Damping ratios	–	D	Determined from resonant column test, dynamic triaxial, or cyclic simple shear test.	
Rod (longitudinal)	D_L			
Shear (torsional)	D_T			
Resonant frequency	–	T^{-1}	Determined from resonant column test.	
longitudinal	f_L			
torsional	f_T			

(a) Units F = Force or weight; L = Length; T = Time; D = Dimensionless; A = Angular Measure

(b) For complete list of volume - weight relationships, see Table 6

Symbols and terms relating to tests and soil properties conform, generally, to definitions given in ASTM Standard D653, Standard Definitions of Terms and Symbols Relating to Soil and Rock Mechanics[32].

When a load is applied to a soil sample, the deformation that occurs will depend on the grain-to-grain contacts (intergranular forces) and the amount of water in the voids (pore water). If no pore water exists, the sample deformation will be due to sliding between soil grains and deformation of individual soil grains. Experience has shown that rearrangement of soil grains due to sliding accounts for the most deformation. Adequate deformation is required to increase the grain contact areas to take the applied load. As the amount of pore water in the void increases the pressure it exerts on soil grains will increase and reduce the intergranular contact forces. In fact, tiny clay particles may be forced completely apart by water in the pore space.

Deformation of a saturated soil is more complicated than dry soil as water molecules, which fill the voids, must be squeezed out of the sample before readjustment of soil grains can occur. The more permeable a soil is, the faster the deformation under load will occur. However, when the load on a saturated soil sample is quickly increased, the increase is carried entirely by the pore water until drainage begins. Then more and more load is gradually transferred to the soil grains until the excess pore pressure has dissipated

[32] American Society for Testing and Materials, Annual Book of ASTM Standards, Part 19 - Natural Building Stone, Soil and Rock, Peat, Mosses, and Humus; Part 14 - Concrete and Mineral Aggregates; Part 4 - Structural Steel, ASTM, Philadelphia, Pennsylvania.

and the soil grains readjust to a denser configuration. The process is called consolidation and results in a higher unit weight and a decreased void ratio.

3.2. Soil Properties, Laboratory Tests and their Presentation in this Book

3.2.1. Types of Soil Properties and Their Laboratory Tests

There are four types of soil properties and corresponding laboratory tests that are used for the analysis of soils for their geotechnical properties:

1. Index Tests
2. Structural Property Tests
3. Dynamic Tests
4. Compactive Samples Tests

Many of these tests are presented in their entirety in this book. The index property tests are included in this section. Others are placed with the topics they relate to.

3.2.1.1. Index Properties and Tests

Index properties are used to classify soils, to group soils in major strata, to obtain estimates of structural properties, an to correlate the results of structural properties tests on one portion of a stratum with other portions of that stratum or other similar deposits where only index test data are available. Procedures for most index tests are standardized (Table 3-2). Either a representative disturbed or an undisturbed sample is utilized. Tests are assigned after review of boring data and visual identification of samples recovered. For a simple project with 4 to 6 borings, at least three gradations and/or Atterberg tests should be made per significant stratum (5 to 15 feet thick). For complex soil conditions, thick strata, or larger sites with more borings, additional tests should be made. Moisture content tests should be made liberally on samples of fine-grained soil. In general, the test program should be planned so that soil properties and their variation can be defined adequately for the lateral and vertical extent of the project concerned.

Table 3-2 Index Property Tests

Test	*ASTM or other Test Specification*	*Location in This Book*
Moisture content of soil; dry unit weight	ASTM D 2216 AASHTO T 265	3.3
Moisture, ash, and organic matter of peat materials	ASTM D2974	
Specific gravity: Material smaller than No. 4 sieve size	ASTM D 854 AASHTO T 100	3.4.3
Specific gravity: Material larger than No. 4 sieve size	ASTM C 127 AASHTO T 85	3.4.3
Unit Weights, Void Ratio, Porosity and Degree of Saturation		3.4.2
Atterberg Limits:		

Liquid limit	ASTM D 4318 AASHTO T 89	3.4.3
Plastic limit	ASTM D 4318 AASHTO T 90	3.4.3
Shrinkage limit	ASTM D 427 AASHTO T 92	
Gradation:		
Sieve analysis	ASTM C 136	3.6
Hydrometer analysis	ASTM D 422 AASHTO T 88	3.6
Corrosivity:		
Sulphate content	ASTM D 4130 (using supernatant from soils)	
Chloride content	ASTM D 512	
pH	ASTM G 51	
Resistivity (laboratory)	ASTM D 1125	
Resistivity (field)	ASTM G 57	
Relative Density	ASTM D2049	3.7
Dispersivity		

3.2.1.2. Corrosivity Tests

The likelihood of soil adversely affecting foundation elements or utilities (concrete and metal elements) can be evaluated on a preliminary basis from the results of the tests referenced in Table 3-2. The tests should be run on samples of soil that will be in contact with the foundations and/or utilities in question; typically, these will be only near-surface materials. For a simple project with uniform conditions, three sets of tests may be adequate. Usually the chemical tests are run only if there is reason to suspect the presence of those ions.

3.2.1.3. Structural Property Tests

3.2.1.3.1. Overview of Tests

A list of structural properties tests is given below.

Table 3-3 Structural Properties Tests

Test	***ASTM or other Test Specification***	***Location in This Book (where applicable) and/or Description***
Permeability:		

Constant head procedure for coarse-grained soils	ASTM D2434 AASHTO T 215	6.8. The simplest of all methods for determining the coefficient of permeability is the constant-head type of test illustrated in Figure 6-1. This test is performed by measuring the quantity of water, Q, flowing through the soil specimen, the length of the soil specimen, L, the head of water, h, and the elapsed time, t. The head of water is kept constant throughout the test. For fine-grained soils, Q is small and may be difficult to measure accurately. Therefore, the constant-head test is used principally for coarse-grained soils (clean sands and gravels) with k values greater than about 10 x 10^{-4} cm/sec.
Falling head procedure		6.8. This test is conducted in the same manner as the constant-head test, except that the head of water is not maintained constant but is permitted to fall within the upper part of the specimen container or in a standpipe directly connected to the specimen. The quantity of water flowing through the specimen is determined indirectly by computation. The falling-head test is generally used for less pervious soils (fine sands to fat clays) with k values less than 10 x 10^{-4} cm/sec.
Flexible Wall Permeability	ASTM D 5084	For fine-grained soils, tests performed using a triaxial cell are generally preferred. In-situ conditions can be modeled by application of an appropriate confining pressure. The sample can be saturated using back pressuring techniques. Water is then allowed to flow through the sample and measurements are taken until steady-state conditions occur.
Capillary head		
Consolidation: Consolidation	ASTM D 2435 AASHTO T 216	7.7
Swell	ASTM D4546 AASHTO T258	7.7.10
Collapse Potential	See [33]	
Shear Strength:		

[33] Jennings, J.E. and Knight, K., A Guide to Construction on or with Materials Exhibiting Additional Settlement Due to Collapse of Grain Structures, Sixth Regional Conference for Africa on Soil Mechanics and Foundation Engineering, 1975.

Direct Shear	ASTM D 3080 AASHTO T 236	5.5; 5.6 (repeated test). The direct shear test is a relatively simple test used to measure the shear strength of fine granular soils. To run the test, a thin soil sample is placed in a shear box consisting of two parallel blocks. The lower block is fixed while the upper block is moved parallel to it in a horizontal direction. The soil fails by shearing along a plane assumed horizontal. This test is suitable for either sands or clays if proper equipment used. Stress-strain curves are usually produced for a minimum of three soil samples, each at a different test pressure. The stress-strain measurements are usually extended substantially beyond the peak to determine the residual shear stress, i.e., the stress that remains constant with increasing strain. The peak and residual shear strengths are plotted versus confining pressure to determine rate of increase of strength with applied load.
Drained Repeated Direct Shear Test		5.6. The repeated direct shear test is used to measure the residual shear strength of a soil (primarily clay shale material) under drained conditions. In this test, a square soil specimen acted on by a normal stress is repeatedly sheared by reversal of the direction of shear until a minimum, i.e., residual, shear stress is determined. The concept of the test is that a specimen of material is acted upon by normal and shear stresses until, after large shear deformation, disaggregation and increase in parallel orientation of soil particles in the direction of shearing occur and a surface or a thin zone of remolded material is formed. Thus, a minimum drained shearing resistance is offered. The shear surface or zone can be formed by: a) shearing an intact specimen b) cutting a plane surface through an intact specimen with a fine wire, or c) shearing a completely remolded specimen. Practical laboratory considerations favour the use of a precut shear plane, as it may not be possible to induce a plane surface by shearing an intact stiff-to-hard specimen. Any irregularities of the shear surface would introduce an added resistance that would not be a measure of the shear strength of the material.

Unconfined Compression	ASTM D 2166	5.8. The unconfined compression test is used to measure the unconfined compressive strength of a cohesive soil. The unconfined compression test is applicable only to coherent materials such as saturated clays or cemented soils that retain intrinsic strength after removal of confining pressure; it is not a substitute for the Q test. Dry or crumbly soils, fissured or varved materials, silts, and sands cannot be tested meaningfully in unconfined compression. In this test, a laterally unsupported cylindrical specimen is subjected to a gradually increased axial compression load until failure occurs. The unconfined compression test is a form of triaxial test in which the major principal stress is equal to the applied axial stress, and the intermediate and minor principal stresses are equal to zero. The unconfined compressive strength, q_u is defined as the maximum unit axial compressive stress at failure or at 15% strain, whichever occurs first. The undrained shear strength, s_u, is assumed equal to one-half the unconfined compressive strength. The axial load may be applied to the specimen either by the controlled strain procedure, in which the stress is applied to produce a predetermined rate of strain, or by the controlled stress procedure, in which the stress is applied in predetermined increments of load.
Triaxial Compression:		The triaxial compression test is a strength test where the sample is subjected to confining pressures similar to those that existed in the ground before sampling. In general, triaxial tests must be done on soil samples that have either been consolidated in the lab to the effective overburden pressure before testing or left unconsolidated and tested at the total overburden pressure. In either case, the tests try to produce the *in situ* effective stress condition. Unconsolidated tests must be done soon after sampling to insure no changes have occurred in the amount of pore water in the sample. The consolidated triaxial compression test duplicates as accurately as possible the sample's conditions in the ground and gives an accurate indication of shear strength. A series of tests on samples consolidated under various confining pressures may be run in order to determine the amount of strength increase with consolidation under embankment loads. The triaxial test procedure may be varied to account for short term (undrained) load application or long term (drained) load application.

Unconsolidated-Undrained (Q or UU)	ASTM D 2850 AASHTO T 296	5.7. In the UU test, the initial water content of the test specimen is not permitted to change during shearing of the specimen. Thus, this test is also dependent on the soil sample retaining its original structure until testing occurs. All UU tests should only be done on samples extruded directly from the sampling tube and tested untrimmed at full diameter. The results are the shear strength existing at that depth. Theoretically, the test confining pressure may be varied substantially above total overburden without changing the test results as the pore water carries all increases in effective stress. Practically, the pore water carries slight increases in effective stress, and slight increases in shear strength will be noted due to sample drying (nonsaturation) and disturbance of original structure. These increases should not be interpreted as shear strength gain with increasing load. The UU shear strength may be used in quick loading situations such as rapid construction of a highway embankment. The Q test is usually applicable only to soils that are not free draining, i.e., to soils having permeability less than 10 x 10^{-4} cm/sec.
Consolidated-Undrained (R or CU)[34]	ASTM D 4767 AASHTO T 297	5.7. In the CU test, complete consolidation of the test specimen is permitted under the confining pressure, but no drainage is permitted during shear. Then, with the water content held constant, the specimen is loaded to failure by increasing the deviator stress. Specimens must as a general rule be completely saturated before application of the deviator stress. A minimum of three tests is required to define strength parameters c and Φ, though four test specimens are preferable with one serving as a check. Each sample is consolidated to a different effective stress. The shear strength of each sample is determined and plotted on a Mohr diagram. The result is a line (called an envelope) which intercepts the Y-axis at the cohesion value and has an inclination measured from the X-axis equal to the friction angle. The undrained shear values may be estimated for any loading within the range of test pressures.

[34] Consolidated-undrained tests may run with or without pore pressure measurements, according to basis for design.

Consolidated-Drained (S or CD)		5.7. In the CU test, complete consolidation of the test specimen is permitted under the confining pressure and during the loading of the specimen to failure by increasing the deviator stress. Consequently, no excess pore pressures exist at the time of failure. This test is interpreted similar to the CU test except the envelope will usually have a smaller cohesion (typical value of 100 psf) and a larger friction angle. The results are used to duplicate long term loadings when excess pore pressures do not develop. A minimum of three tests is required for c' and Φ' determination. CD tests are generally performed on well draining soils. For slow draining soils, several weeks may be required to perform a CD test.
Vane Shear	ASTM D 4648	The vane shear test is a field test made in conjunction with drill hole explorations in soft clays; it is discussed in more detail with other field tests. This is a test for determining shear strength rapidly without laboratory testing. The results are accurate in soft silts and clays. Miniature laboratory vane tests are not dependable due to sample confinement in the tube, size effects, and sampling disturbance.
Constant Rate of Strain Test	ASTM D 4186	The sample is subjected to a constantly changing load while maintaining a constant rate of strain; and the single-increment test, sometimes used for organic soils, in which the sample is subjected only to the load expected in the field. A direct analogy is drawn between laboratory consolidation and field settlement amounts and rates.
Organic Content	ASTM D 2974 AASHTO T 267	Organic soils demonstrate very poor engineering characteristics, most notably low strength and high compressibility. In the field, these soils can usually be identified by their dark colour, musty odour and low unit weight. The most used laboratory test for design purposes is the Ignition Loss test, which measures how much of a sample's mass burns off when placed in a muffle furnace. The results are presented as a percentage of the total sample mass.

3.2.1.3.2. Shear Strength Test Selection and Structure

The shear strength of a soil is the maximum shear stress that the soil structure can resist before failure. Shear stresses are carried by the structure of soil grains, as the water filling the pores has no shear strength. However, the shear strength of the soil structure is indirectly dependent on the pressure in the pore water which influences the $\overline{\sigma}_v$ term in Equation 5-10. Foundation designers must consider the effects of expected construction operations on the subsoils when planning a test program.

For example, when a highway embankment or structure footing is suddenly placed on a soft clay deposit, the pore water initially carries the entire load and shear strength does not increase until drainage begins and the pore pressure decreases. In planning a test program for such a situation the designer would request unconsolidated undrained triaxial tests to determine the critical strength values, i.e., undrained shear strength before consolidation begins. Additional consolidated undrained or drained tests would also be used to determine the increase in shear strength as consolidation occurs and pore pressures dissipate. These results can be used to determine alternate methods of safely applying the loads, especially if the critical unconsolidated undrained strength is insufficient to sustain the proposed loading.

In determining the type of test to be employed, considerations must be given to soil type and the applications for which the test data is required.

- Clean Sands and Gravels. Undisturbed samples are very difficult to obtain and test properly; therefore sophisticated shear tests are usually impractical. For simple foundation problems, the angle of internal friction can be satisfactorily approximated by correlation with penetration resistance, relative density, and soil classification such as shown in Figure 2-3. Confirmation of the potential range of the angle of internal friction can be obtained from shear tests on the sample at laboratory densities bracketing conditions anticipated in the field. For earth dam and high embankment work where the soil will be placed under controlled conditions, triaxial compression tests are warranted.
- Clays. For simple total stress applications where the immediate stability of foundations or embankments is of concern, the unconfined compression test or UU triaxial test is often adequate. For very soft or sensitive soils, which are difficult to sample, the field vane test is useful. For long-term stability problems requiring effective stress and analysis, such as landslides, CU triaxial tests with pore pressure measurements should be used. Long-term stability problems in some highly overconsolidated clays may require the CD test[35].
- Silts and Mixed Soils. The choice of test is governed by whether total stress analysis or effective stress analysis is applicable. In cases of very soft silts, such as in marine deposits, the in-place vane shear test is especially helpful in evaluating the shear strength and its increase with depth. For some thinly layered soils, such as varved clay, direct shear tests or simple shear tests are well suited for determining the strength of the individual layers. Where partial drainage is anticipated, use CU tests with pore water pressure measurements to obtain effective strength parameters.
- Overconsolidated Soils. Frequently overconsolidated soils have defects such as jointing, fissures, etc. The laboratory values of strength, which are obtained from a small test specimen, are generally higher than the field strength values that are representative of the entire soil mass.

The release of stress due to excavation and exposure to weathering reduces strength over a long period. This effect cannot be assessed by any of the laboratory tests currently in use. Most overconsolidated clays are anisotropic and the degree of anisotropy may be influenced by their age. Effect of anisotropy can be determined in the laboratory.

In highly overconsolidated soil that may not be fully saturated, unusually high backpressure may be necessary to achieve full saturation, thus making it difficult to perform CU tests. CS tests are more appropriate.

[35] Skempton, A.W. and Hutchinson, J., Stability of Natural Slopes and Embankment Foundations State-of-the-Art Report, Proceedings, Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico, 1969.

3.2.1.4. Dynamic Tests

3.2.1.4.1. Laboratory Tests

Dynamic testing of soil and rock involves three ranges: low frequency (generally less than 10 Hz) cyclic testing, resonant column high frequency testing, and ultrasonic pulse testing. The dynamic tests are used to evaluate foundation support characteristics under repeated loadings such as a drop forge, traffic, or earthquake; a primary concern is often liquefaction. Young's modulus (E_s), shear modulus (G), and damping characteristics are determined by cyclic triaxial and simple shear tests. Resonant column can be used to determine E_s, G, and damping.

From the resonant frequency of the material in longitudinal, transverse, and torsional modes, Poisson's ratio (ν) can be computed from test data. Foundation response to dynamic loading and the effect of wave energy on its surroundings is studied in the light of these test results. The ultrasonic pulse test also evaluates the two moduli and Poisson's ratio, but the test results are more reliable for rocks than for soils.

Dynamic tests can be run on undisturbed on compacted samples, but should be run only if the particular project really requires them. The number of tests depends on project circumstances. Estimates of dynamic parameters can be obtained from correlations with other properties. A list of dynamic test is given in Table 3-4.

Capabilities of dynamic soil testing methods and their suitability for various motion characteristics are shown in Figure 3-1. Dynamic testing is needed for loose granular soils and soft sensitive clays in earthquake areas, for machine foundation design, and for impact loadings.

Various testing and material factors that may affect cyclic strength as determined in the laboratory are method of specimen preparation, difference between reconstituted and intact specimens, prestressing, loading wave form, grain size and gradation, etc.[36]

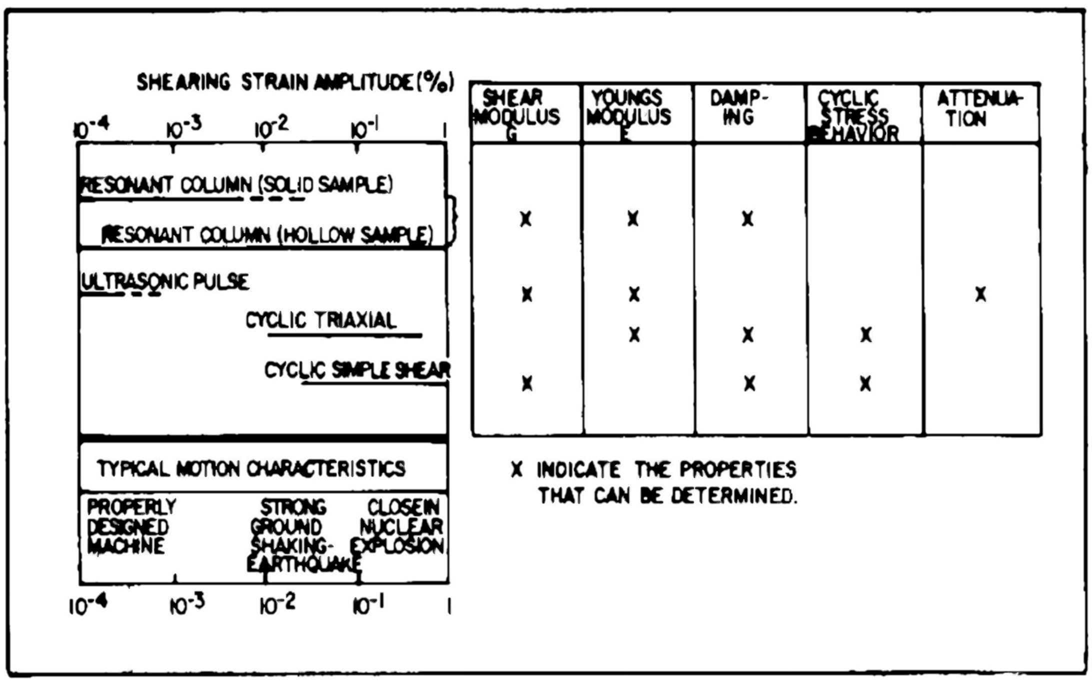
Table 3-4 Dynamic Tests

Test	***ASTM or other Test Specification***	***Location in This Book (where applicable) or information on the test***
Cyclic Triaxial Compression	ASTM D 3999	10.4.3.
Cyclic Simple Shear Test		Simple shear equipment has also found wide use in cyclic testing. The non-uniform stress conditions in simple shear may cause failure at a lower stress than that which would cause failure *in situ*. Measurement or control of lateral pressure is difficult in simple shear tests.
Cyclic Torsional Simple ShearTests	See [37]	Cyclic torsional simple shear tests on hollow samples offer the capability of measuring lateral confining pressure. In hollow cylinders, stresses within the specimen are uniform, though the specimens are difficult to produce. In addition, tapered hollow cylinders have been used in torsional cyclic tests.

[36] For details on cyclic testing, see Townsend, F.L., A Review of Factors Affecting Cyclic Triaxial Tests, Dynamic Geotechnical Testing, ASTM, STP 654, pp 356-383, 1978. For the nature of soil behaviours under various types of dynamic testing see Hardin, B.O., The Nature of Stress-Strain Behavior for Soils, Earthquake Engineering and Soil Dynamics, Proceedings of the ASCE geotechnical engineering Division Specialty Conference, Pasadena, California, pp. 3-90, 1978.

Resonant Column	See [38]	The resonant column test consists of the application of sinusoidal vibration to one end (termed the active end) of a solid or hollow cylindrical soil specimen. The other end is known as the passive end. Compression waves or shear waves are propagated through the soil specimen to determine either Young's (E_s) or shear modulus (G). Moduli are computed from the resonant frequency of the cylinder. For example, in the case where passive end platen is fixed, the lowest frequency for which the excitation force is in phase with the velocity at the active end is termed the resonant frequency. Damping is determined by turning off the excitation at resonant frequency and recording the decaying vibration.
Ultrasonic Pulse:		
Soil	See [39]	
Rock	**ASTM D2845**	
Critical Void Ratio		

Figure 3-1 Capabilities of Dynamic Testing Apparatus



[37] Wood, Richard D., Measurement of Dynamic Soil Properties, ASCE Geotechnical Division Special Conference on Earthquake Engineering and Soil Dynamics, 1978.

[38] Drnevich, V.P., Hardin, B.O., and Shippy, D.J., Modulus and Sampling of Soils by the Resonant Column Method, ASTM, STP 654, 1978.

[39] Stephenson, R.W., Ultrasonic Testing for Determining Dynamic Soil Modulus, ASTM, STP 654, 1978.

3.2.1.4.2. Empirical Indicators

The empirical relationships given here are to be used only as indicators and not in final design. Design involving dynamic properties of soil must be done only under the direction of experienced personnel.

- Shear Modulus. In the absence of dynamic tests initial estimates of shear modulus, G, may be made using relationships found in the literature.[40]
- Poisson's Ratio. Values of Poisson's ratio (ν) are generally difficult to establish accurately. For most projects, the value does not affect the response of the structure sufficiently to warrant a great deal of effort in their determination. For cohesionless soils, $\nu = 0.25$ and for cohesive soils $\nu = 0.33$ are considered reasonable assumptions.[41]
- Liquefaction of Coarse-Grained Soils. Liquefaction has usually occurred in relatively uniform material with D_{10} ranging between 0.01 and 0.25 mm, C_u between 2 and 10, and standard penetration resistance less than 25 blows per foot. Liquefaction is more likely to be triggered by higher velocity than by higher acceleration. These characteristics may be used as a guide in determining the need for dynamic testing. The potential influence of local soil conditions on shaking (depth of stratum, depth of groundwater table, variation in soil density, etc.) and damage intensity must be carefully evaluated.[42] A surcharge reduces the tendency of a deposit to liquefy.

3.2.1.5. Compactive Samples Tests

In prospecting for borrow materials, index tests or compaction tests may be required in a number proportional to the volume of borrow involved or the number of samples obtained. Structural properties tests are assigned after borrow materials have been grouped in major categories by index and compaction properties. Select samples for structural tests to represent the main soil groups and probably compacted condition. At least one compaction or relative density test is required for each significantly different material (based on gradation or plasticity). Numbers of other tests depend on project requirements. A list of these tests is given in Table 3-5.

Table 3-5 Compactive Samples Tests

Test	*ASTM or other Test Specification*	*Location in This Book (where applicable) or Overview*
Moisture-Density Relations:		
Standard Proctor 5.5" lb. Hammer, 12" drop	ASTM D698 AASHTO T 99	9.7; 9.8

[40] Hardin, B.O. and Drnevich, V.P., Shear Modulus and Damping in Soils: Design Equations and Curves, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 98, No. SM7, 1972, and Seed, H.B. and Idriss, I.M., Soil Moduli and Damping Factors for Dynamic Response Analyses, Reprot No. EERC 70-10, University of California, 1970.

[41] Richart, F.E., Foundation Vibration, Foundation Engineering Handbook, H.F. Winterkorn and H.Y. Fang, eds., Van Nostrand Reinhold Company, New York, Chapter 24, 1975.

[42] Seed, H.B., Earthquake Effects on Soil Foundation Systems, Foundation Engineering Handbook, H.F. Winterkorn and H.Y. Fang, eds., Van Nostrand Reinhold Company, New York, Chapter 25, 1975, and Iwasaki, R., Tatsuoka, F., Tokida, K. and Yasuda, S., A Practical Method for Assessing Soil Liquefaction Potential Based on Case Studies at Various Sites in Japan, Proceedings of the Second International Conference on Microzonation for Safer Construction-Research and Application, Vol. II, San Francisco, pp 885-896, 1978.

drop		
Modified Proctor: 10 lb. Hammer, 18" drop	ASTM D1557 AASHTO T 180	9.7; 9.8
Relative Density	ASTM D 4253 (Maximum Density) ASTM D 4254 (Minimum Density)	3.7
Maximum and Minimum Densities of Cohesionless Soils	ASTM D2049	
California Bearing Ratio	ASTM D 1883	
Resistance R-value	ASTM D 2844	
Expansion Pressure	AASHTO T 190	
Permeability and Compression	See 43	
Resilient Modulus	AASHTO T 294	This test is used to determine the dynamic elastic modulus of a base or subgrade soil under conditions that represent a reasonable simulation of the physical conditions and stress states of such materials under flexible pavements subjected to wheel loads. A prepared cylindrical sample is placed in a triaxial chamber and conditioned under static or dynamic stresses. A repeated axial stress is then applied at a fixed magnitude, duration, and frequency. The resilient modulus, M_r, is calculated by dividing the deviator stress by the resilient axial strain. This value is used in the design and evaluation of pavement systems.

3.2.1.5.1. Utilization

Compaction is used to densify soils during placement to minimize post-construction consolidation and to improve strength characteristics. Compaction characteristics are determined by moisture density testing; structural and supporting capabilities are evaluated by appropriate tests on samples of compacted soil.

3.2.1.5.2. Moisture-Density Relationships

The Proctor test or a variation is employed in determining the moisture-density relationship. For cohesionless soils, Relative Density methods may be more appropriate.

- Standard Proctor Test. Use standard Proctor tests for ordinary embankment compaction control. In preparing for control, obtain a family of compaction curves representing principal borrow materials.

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[43] Bureau of Reclamation, Permeability and Settlement of Soils, Earth Manual, Designation E-13, United States Government Printing Office, 1974.

- Modified Proctor Test. Especially applicable to either a heavily compacted base course or a subgrade for airfield pavement and may be used for mass earthwork.
- Relative Density of Cohesionless Soils. Proctor tests are often difficult to control for free-draining cohesionless soils and may give erratic compaction curves or density substantially less than those provided by ordinary compaction in the field[44]. Thus, relative density methods may be preferred. Tests for maximum and minimum densities should be done in accordance with ASTM Standard D2049, Relative Density of Cohesionless Soils.

3.2.1.5.3. California Bearing Ratio (CBR)

This test procedure covers the evaluation of subgrade, subbase, and base course materials for pavement design for highways and airfield. The resistance of a compacted soil to the gradual penetration of a cylindrical piston with 3 square inches in area is measured. The load required causing either 0.1 inch or 0.2 inch penetration of the piston is compared to that established for a standard compacted crushed stone to obtain the bearing ratio.

3.2.2. Laboratory Tests on Rock Cores

Laboratory tests on rock are performed on small samples of intact cores. However, the properties of in-situ rock are often determined by the presence of joints, bedding planes, etc. It is also important that the rock cores come from the zone that the foundations are founded in. Laboratory test results must therefore be considered in conjunction with knowledge of the in-situ characteristics of the rock mass. Some of the more common laboratory tests are:

3.2.2.1. Unconfined Compression Test

This test is performed on intact rock core specimens, which preferably have a length of at least two times the diameter. The specimen is placed in the testing machine and loaded axially at an approximately constant rate such that failure occurs within 2 to 15 minutes. **Note: the testing machine must be of the proper size for the samples being tested.** Tests shall be performed in accordance with ASTM D 2938.

3.2.2.2. Absorption and Bulk Specific Gravity

Absorption is a measure of the amount of water, which an initially dry specimen can absorb during a 48-hour soaking period. It is indicative of the porosity of the sample. Bulk specific gravity is used to calculate the unit weight of the material. Tests shall be performed in accordance with ASTM C 97.

3.2.2.3. Splitting Tensile Strength Test

This test is an indirect tensile strength test similar to the point load test; however, the compressive loads are line loads applied parallel to the core's axis by steel bearing plates between which the specimen is placed horizontally. Loading is applied continuously such that failure occurs within one to ten minutes. The splitting tensile strength of the specimen is calculated from the results. Tests shall be performed in accordance with ASTM D 3967 ***except the*** minimum t/D (length-to-diameter) ratio shall be 1.0 when testing.

3.2.2.4. Triaxial Compression Strength

This test is performed to provide shearing strengths and elastic properties of rock under a confining pressure. It is commonly used to simulate the stress conditions under which the rock exists in the field. Tests shall be performed in accordance with ASTM D 2664.

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

3.2.2.5. Unit Weight of Sample

This is a direct determination of either the moist or total weight of the rock core sample divided by the total cylindrical volume of the intact sample (for the total/moist unit weight), or the oven-dried weight divided by the total volume (for the dry unit weight). This measurement includes any voids or pore spaces in the sample, and therefore can be a relative indicator of the strength of the core sample. Samples should be tested at the moisture content representative of field conditions, and samples should be preserved until time of testing. Moisture contents shall be performed in accordance with ASTM D 2216.

3.2.3. Testing Procedures

Since soils exist in an enormous variety, and since the problems of applied soil mechanics exist in a very great variety, testing procedures for determining the engineering properties of soils (such as strength-deformation relationships) cannot be standardized.

Before any soils testing is requested of a laboratory, the design engineer responsible for formulating the testing program must clearly define the purpose of each test to himself and to the person who will supervise the testing. It is generally necessary to adapt the testing procedures to the specific requirements of an investigation.

For example, the consolidation test can be performed in various ways. What is often called the “standard consolidation test” is performed by always doubling the previous load on the specimen. This procedure will produce time-consolidation curves that usually permit the most precise evaluation of the coefficients of permeability and consolidation. However, these load increments are not always satisfactory for defining the preconsolidation pressure from the shape of the void ratio-pressure curve; for this purpose, a much smaller factor than 2.0 should be used during incremental loading. In addition, the maximum load to which a consolidation test should be continued will depend on the consistency and stress history of the soil and the requirements of the project.

For example, if clay that had been normally consolidated under an effective overburden pressure of 1 ksf is to be loaded by an embankment that will exert an additional pressure of 2 ksf, the consolidation test need not be continued beyond a load of 8 ksf to fulfill the purpose of the test. On the other hand, highly overconsolidated clay that will be loaded by an embankment of substantial height may require that the consolidation test be continued to a loading of 40 ksf or more.

An even greater variety in testing procedures exists for measuring the strength of soils, and the purpose of the tests must be constantly reviewed to insure that the results have meaning with respect to design. Tests that do not measure clearly defined engineering properties such as Atterberg limits, specific gravity, grain-size analyses, and compaction), however, do require adherence to standardized procedures. Even here, the dangers of injudicious testing must be recognized. As an example, compaction test results-must be carefully evaluated if the material coarser than _” (or some other size) has been removed according to the standard method.

Deviations from these procedures may be necessary on occasion, according to the judgment of testing or design engineers, their experience with local soils, or peculiarities of a project. However, to insure that the test methods remain compatible with the purpose of the tests and that the results will be acceptable, every such deviation should be discussed in advance with the design office requesting the tests. In addition, a description of any non-conventional procedure must accompany the test data.

3.2.4. Reliability of Testing Apparatus and Responsibility of Personnel

All who are engaged in soils testing must constantly be aware of the importance of accuracy in measurement. Inaccurate measurements will produce test results that are not only valueless but are misleading.

Each test described below contains a list of the more common possible errors associated with the procedures described in that test. Serious errors can be caused by poorly constructed apparatus (for example, piston friction in triaxial compression chambers or rough finished consolidometer rings), by maladjusted apparatus (liquid limit devices, proving rings, or mechanical compactors), and by worn parts (liquid limit cup or grooving tool or knife edges of lever systems). Regular calibration and inspection must be a standard practice in all laboratories. The personnel performing the tests must be thoroughly familiar with the apparatus, the testing procedures, and good laboratory technique in general.

They must be conscientious in the handling of soils and must appreciate the purpose of each test they perform. Neat, thoughtful work, with the recording of all test data and a continuous watchfulness for irregularities can prevent most errors.

The philosophy should be that one good test is not only far better than many poor tests, but is also less expensive and less likely to permit a misjudgment in design.

3.2.5. Laboratory Facilities

A laboratory preferably should be on a ground floor or basement with a solid floor and should be free of traffic and machinery vibrations.

Separate areas should be designated for dust producing activities such as sieve analyses and sample processing. Temperature control of the entire laboratory is to be preferred. If the temperature-controlled space is limited, this space should be used for triaxial compression, consolidation, and permeability testing.

A humid room large enough to permit the storage of samples and the preparation of test specimens should be available.

3.2.6. Sample Handling and Storage

The identification markings of all samples should be verified immediately upon their receipt at the laboratory, and an inventory of the samples received should be maintained. Samples should be examined and tested as soon as possible after receipt; however, it is often necessary to store samples for several days or even weeks to complete a large testing program.

Every care must be taken to protect undisturbed samples against damage or changes in water content. Such samples should be stored in a humid room and may require rewaxing and relabeling before storage.

Except for special purposes, such as for viewing by designers or contractors or for research, soil samples should not be retained for long periods; even the most careful sealing and storing of undisturbed samples cannot prevent the physical and chemical changes which, in time, would invalidate any subsequent determinations of their engineering properties.

3.2.7. Selection and Preparation of Test Specimens

Under the most favorable circumstances, a laboratory determination of the engineering properties of a small specimen of undisturbed soil gives but an approximate guide to the behavior of an extensive nonhomogeneous geological formation under the complex system of stresses induced by the construction of an embankment or other structure; under the worst circumstances such a determination may have no meaning. In addition, the strength, compressibility, and permeability of a soil in place may vary by several orders of magnitude within a few inches.

No other aspect of laboratory soils testing is as important as the selection of test specimens to best represent those features of a foundation soil that influence the design of a project. The selection cannot be based on boring logs alone, but requires personal inspection of the samples and the closest teamwork of the laboratory personnel and the design engineer.

This cooperation must be continued throughout the testing program since, as quantitative data become available, changes in the initial allocation of samples or the securing of additional samples may be necessary. Second in importance only to the selection of the most representative undisturbed material is the preparation and handling of the test specimens to preserve in every way possible the natural structure and water content of the material.

Indifferent handling of undisturbed soils can result in test data that are erroneous by several times any errors caused by faulty testing apparatus. With but few exceptions, test specimens should always be prepared in a humid room.

Trimming instruments should be sharp and clean and the specimens should be adequately supported at all times; details of the preparation equipment and procedures are presented in the test procedures.

What cannot be gained from any manual, however, is the judgment and awareness necessary to adjust the techniques for each type of material in order to secure the most satisfactory specimens. During the preparation of specimens, the laboratory personnel have the best opportunity to record a complete description of the material and to judge whether the material is truly undisturbed.

The description should include an identification of the material, its color and consistency, the brittleness of the material and the loss of strength upon remolding, and any heterogeneity or unusual characteristics that might prove valuable in analyzing the test results. In addition, any indication of disturbance of boring samples (strata deformed at periphery or distortions concentric with axis of sample) must be noted.

Often these distortions cannot be seen except by slowly drying a slice of the material to water content at which the differences between strata show clearly.[45] Photographs of such partially dried slices may be helpful when evaluating the test data and can contribute to improvements in sampling equipment and techniques.

Disturbed samples should never be used for any tests other than classification, specific gravity or water content.

3.2.8. Data Sheets and Report Forms

Examples of suggested form sheets for recording and computing test data are presented in the tests, and some tests show the forms to be used for reporting test results. These can be found in 11.2. The data sheets shown may be satisfactory in many instances, though each laboratory should adopt whatever data sheets are most suitable for their practices and apparatus.

Well-planned data sheets can improve the efficiency of testing and, by encouraging the recording of data which otherwise might be lost, can lead to better testing. Because they are intended for review purposes, these forms often do not display the test results in sufficient detail for interpretation by the design engineer. Therefore, each laboratory should include with the standard report forms whatever tabulated or plotted data are necessary to satisfy the purpose of a testing program.

Graphs should show all the plotted points, not just smooth curves, and be given scales in easily read units, such as 1, 2, or 5 divisions per unit. The report form should contain a complete description of the material, not just the classification, and sketches to illustrate the mode of failure of strength test specimens.

[45] M. J. Hvorslev, Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes, U. S. Army Engineer Waterways Experiment Station, CE (Vicksburg, Miss., November 1949).

3.3. Elastic Parameters

3.3.1. General

The magnitudes of soil elastic distortion or immediate settlement for practical applications are evaluated from the elastic soil parameters Young's modulus E_s, shear modulus G_s and Poisson's ratio ν_s. For most practical applications the foundation soil is heterogeneous or multilayered in which the elastic parameters can vary significantly from layer to layer.

3.3.2. Elastic Young's Modulus

Young's elastic modulus is commonly used for estimation of settlement from static loads. Suitable values of the elastic modulus E_s as a function of depth may be estimated from empirical correlations, results of laboratory tests on undisturbed specimens and results of field tests.

3.3.2.1. Definition

Materials that are truly elastic obey Hooke's law in which each equal increment of applied uniaxial stress σ_z causes a proportionate increase in strain ε_z

Equation 3-1: $$\varepsilon_z = \frac{1}{E}\sigma_z$$

where E is Young's modulus of elasticity, Table 3-6. Figure 3-2 illustrates the stress path for the uniaxial (UT) and other test methods. An elastic material regains its initial dimensions following removal of the applied stress.

Table 3-6 Laboratory Tests for Evaluation of Elastic Parameters

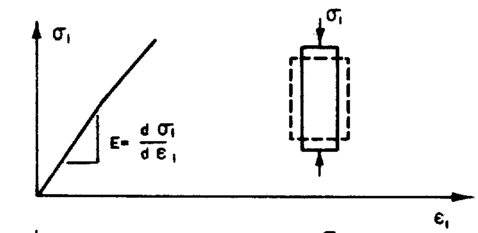
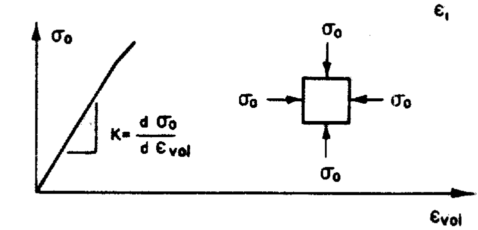
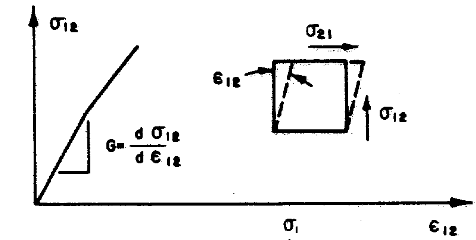
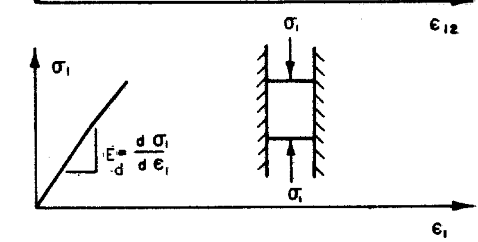
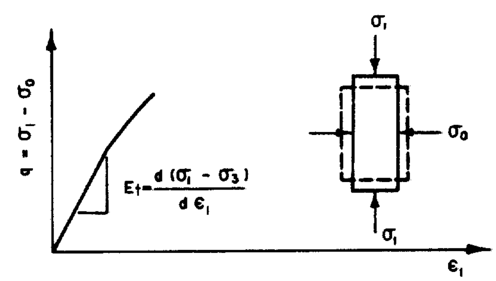
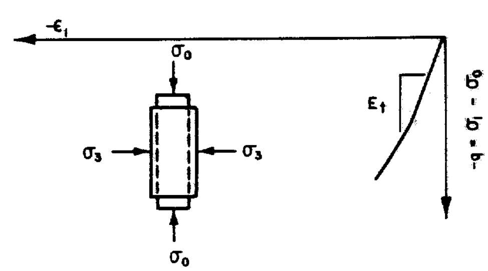
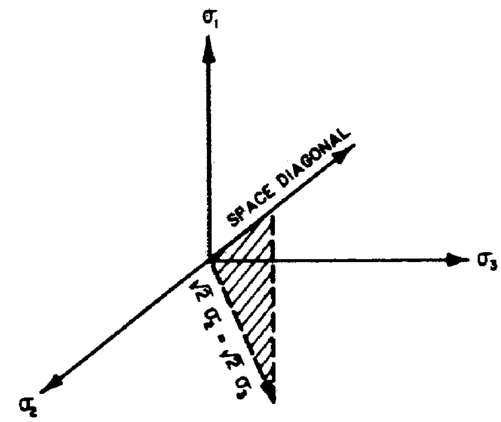
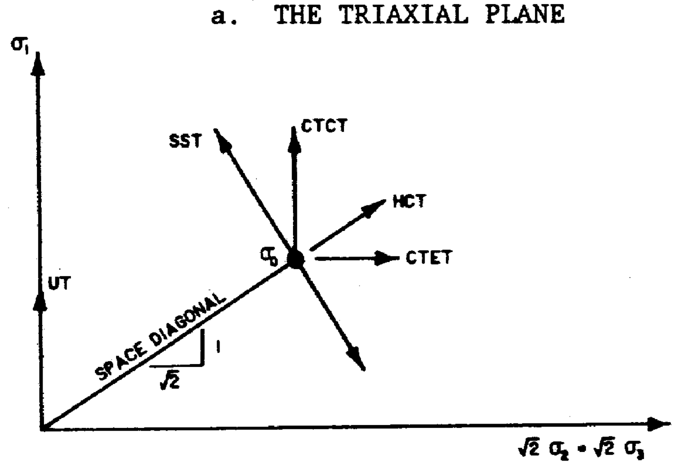
TYPE OF TEST	DESCRIPTION	DIAGRAM
UNIAXIAL STRESS (UT)	LOADING σ_1 ON A SINGLE VERTICAL AXIS. $\sigma_2 = \sigma_3 = 0$. YOUNG'S MODULUS E IS DETERMINED.	
HYDROSTATIC COMPRESSION (HCT)	LOADING OCCURS ALONG THE SPACE DIAGONAL IN EQUAL INCREMENTS $\sigma_0 = \sigma_1 = \sigma_2 = \sigma_3$ AND $\epsilon_{vol} = \epsilon_1 + \epsilon_2 + \epsilon_3$ BULK MODULUS K IS DETERMINED.	
SIMPLE SHEAR (SST)	AFTER HYDROSTATIC LOADING TO $\sigma_0 = \sigma_{OCT}$ $(\sigma_1 + \sigma_2 + \sigma_3)/3$ KEPT CONSTANT, BUT TWO OF THREE STRESS AXES VARIED; i.e., $\Delta\sigma_1 = -\Delta\sigma_3$, $\Delta\sigma_2 = 0$. SHEAR MODULUS G IS DETERMINED.	
CONFINED COMPRESSION (CCT)	LOADING σ_1 WHEN $E_2 = E_3 = 0$ (CONSOLIDATION TEST) CONSTRAINED MODULUS E_d IS DETERMINED.	
CONVENTIONAL TRIAXIAL COMPRESSION (CTCT)	AFTER HYDROSTATIC LOADING TO σ_0, σ_1 INCREASED WHILE $\sigma_2 = \sigma_3$ KEPT CONSTANT AT σ_0. TANGENT MODULUS E_t IN COMPRESSION DETERMINED.	
CONVENTIONAL TRIAXIAL EXTENSION (CTET)	AFTER HYDROSTATIC LOADING TO σ_0, $\sigma_2 = \sigma_3$ INCREASED WHILE σ_1 KEPT CONSTANT AT σ_0. TANGENT MODULUS E_t IN EXTENSION DETERMINED.	

Figure 3-2 Examples of stress paths for different tests (Refer to Table 8-1 for descriptions of tests)



a. THE TRIAXIAL PLANE



b, STRESS PATHS

3.3.2.1.1. Application to soil

Hooke's law, which is applicable to homogeneous and isotropic materials, was originally developed from the observed elastic behavior of metal bars in tension. Soil is sometimes assumed to behave linearly elastic under relatively small loads. A partially elastic material obeys Hooke's law during loading, but this material will not gain its initial dimensions following removal of the applied stress. These materials are nonlinear and include most soils, especially foundation soil supporting heavy structures that apply their weight only once.

3.3.2.1.2. Assumption of Young's elastic modulus

Soils tested in a conventional triaxial compression (CTCT) device under constant lateral stress will yield a tangent elastic modulus E_t equivalent with Young's modulus. The soil modulus E_s is assumed approximately equal to Young's modulus in practical applications of the theory of elasticity for computation of settlement.

3.3.2.1.3. Relationship with other elastic parameters

Table 3-7 relates the elastic modulus E with the shear modulus G, bulk modulus K and constrained modulus E_d. These parameters are defined in Table 3-6.

Table 3-7 Relationships between Elastic Parameters

Parameter	*Relationship*
Shear modulus G, tsf	$\frac{E}{2(1+\nu)}$, ν = Poisson's Ratio
Bulk Modulus K, tsf	$\lambda_L + \frac{2G}{e}$, λ_L = Lame's Constant
Constrained modulus E_d, tsf	$\frac{E(1-\nu)}{(1+\nu)(1-2\nu)}$

3.3.2.2. Empirical Correlations

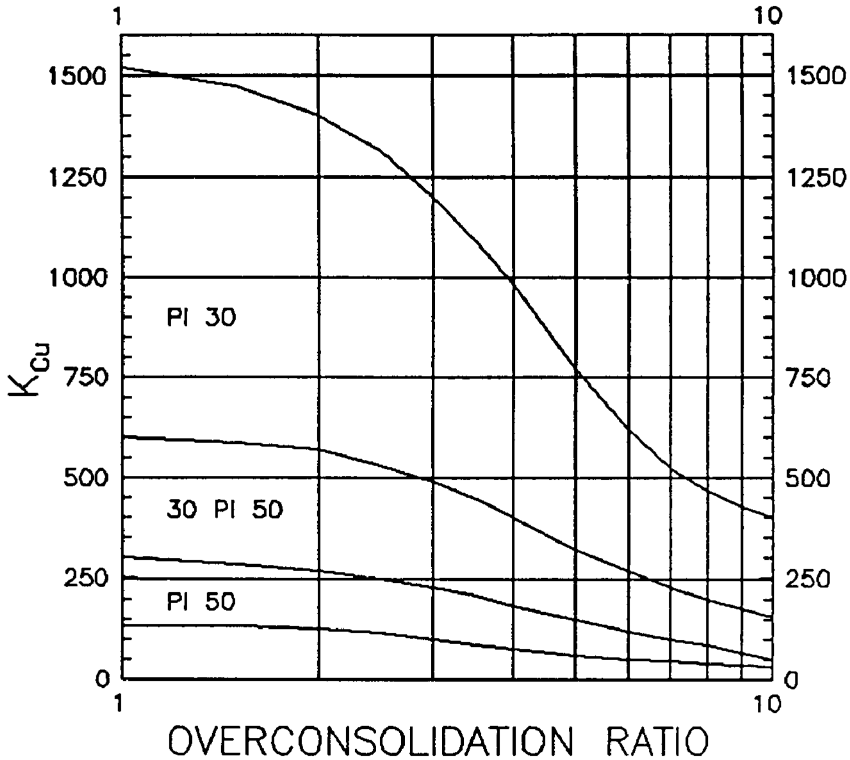
The elastic undrained modulus E_s for clay may be estimated from the undrained shear strength C_u by

$$\textbf{Equation 3-2: } E_s = K_c C_u$$

Where

- E_s = Young's soil modulus, tsf
- K_c = correlation factor, Figure 3-3
- C_u = undrained shear strength, tsf

Figure 3-3 Chart for estimating constant K_{cu} to determine the elastic modulus $E_s = K_{cu}C_u$ from the undrained shear strength



E_s = $K_{Cu}C_u$

E_s = UNDRAINED ELASTIC MODULUS

K_{Cu} = FACTOR RELATING E_s WITH C_u

C_u = UNDRAINED SHEAR STRENGTH

The values of K_{cu} as a function of the overconsolidation ratio and plasticity index PI have been determined from field measurements and are therefore not affected by soil disturbance compared with measurements on undisturbed soil samples. Table 3-8 illustrates some typical values for the elastic modulus.

Table 3-8 Typical Elastic Moduli

Soil	***E_s, tsf***
Clay	
Very soft clay	5 - 50
Soft clay	50 - 200
Medium clay	200 - 500

Stiff clay, silty clay	500 - 1000
Sandy clay	250 - 2000
Clay shale	1000 - 2000
Sand	
Loose sand	100 - 250
Dense sand	250 - 1000
Dense sand and gravel	1000 - 2000
Silty sand	250 - 2000

3.3.2.3. Laboratory Tests on Cohesive Soil

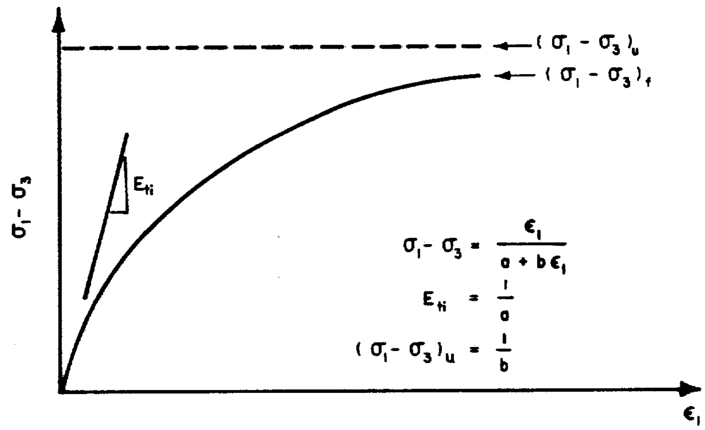
The elastic modulus is sensitive to soil disturbance which may increase pore water pressure and, therefore, decrease the effective stress in the specimen and reduce the stiffness and strength. Fissures, which may have little influence on field settlement, may reduce the measured modulus compared with the *in situ* modulus if confining pressures are not applied to the soil specimen.

3.3.2.3.1. Initial hyperbolic tangent modulus

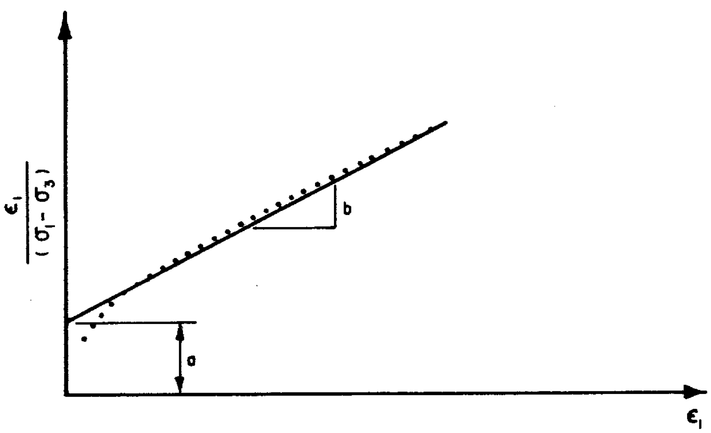
Triaxial unconsolidated undrained (Q or UU) compression tests may be performed on the best available undisturbed specimens at confining pressures equal to the total vertical overburden pressure σ_o for that specimen when in the field using the Q test procedure described in 7.7. An appropriate measure of E_s is the initial tangent modulus $E_{ti} = 1/a$ where a is the intercept of a plot of strain/deviator stress versus strain, Figure 3-4[46].

[46] Duncan, J. M. and Chang, C. Y. 1970. "Nonlinear Analysis of Stress and Strain in Soils," Journal of the Soil Mechanics and Foundations Division, Vol 96, pp 1629-1653. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Figure 3-4 Hyperbolic simulation of stress-strain relationships



a. HYPERBOLIC RELATIONSHIP

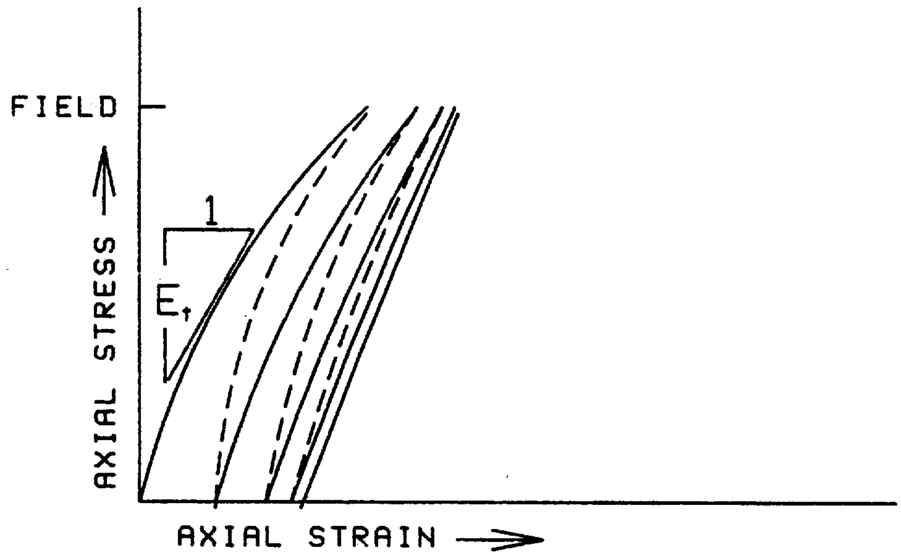


b. EVALUATION OF HYPERBOLIC PARAMETERS a, b

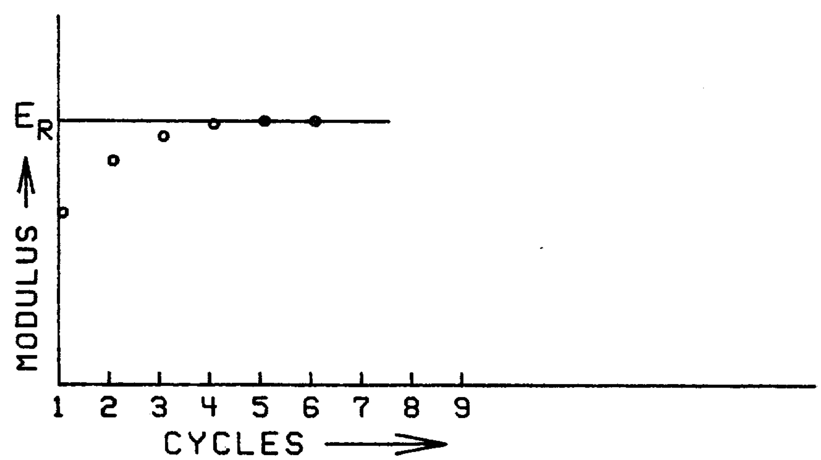
3.3.2.3.2. Reload modulus

A triaxial consolidated undrained (R or CU) compression test may be performed on the best available undisturbed specimens. The specimen is initially fully consolidated to an isotropic confining pressure equal to the vertical overburden pressure σ_o for that specimen in the field. The R test procedure described in 7.7 may be used except as follows: stress is increased to the magnitude estimated for the field loading condition. The axial stress may then be reduced to zero and the cycle repeated until the reload curve shows no further increase in slope. The tangent modulus at 1/2 of the maximum applied stress is determined for each loading cycle and plotted versus the number of cycles, Figure 3-5. An appropriate measure of E_s is the reload tangent modulus that approaches the asymptotic value at large cycles.

Figure 3-5 Elastic modulus from cyclic load tests



a. TANGENT MODULUS AT 1/2 MAXIMUM APPLIED STRESS



b. TANGENT RELOAD MODULUS VERSUS CYCLES

3.3.2.4. Field Tests

The elastic modulus may be estimated from empirical and semiempirical relationships based on results of field soil tests.

3.3.2.4.1. Plate load test

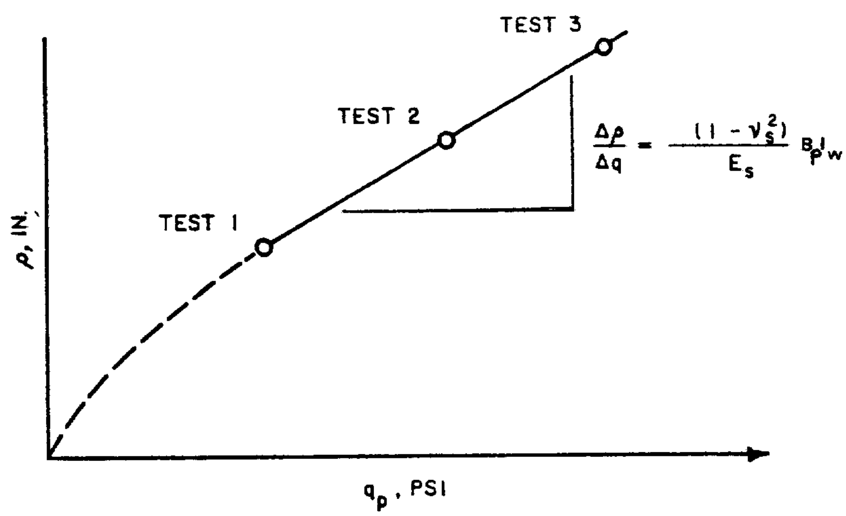
The plate load test performed in accordance with ASTM Standard Test Method D 1194, "Bearing Capacity of Soil for Static Loads on Spread Footings" is used to determine the relationship between settlement and plate pressure q_p, Figure 3-6. The elastic modulus E_s is found from the slope of the curve $\Delta\rho/\Delta q_p$

Equation 3-3: $$E_s = \frac{1-\nu_s^2}{\frac{\Delta p}{\Delta q_p}} B_p I_W$$

Where

- E_s = Young's soil modulus, psi
- ν_s = Poisson's ratio, 0.4
- $\frac{\Delta p}{\Delta q_p}$ = slope of settlement versus plate pressure, inches/psi
- B_p = diameter of plate, inches
- I_w = influence factor, $\pi/4$ for circular plates

Figure 3-6 Graphical solution of soil elastic modulus E_s from the plate load test



- ν_s = Poisson's ratio

This elastic modulus is representative of soil within a depth of $2B_p$ beneath the plate.

3.3.2.4.2. Cone penetration test (CPT)

The constrained modulus E_d has been empirically related with the cone tip bearing resistance by

Equation 3-4: $$E_d = \alpha_c q_c$$

Where

- E_d = Constrained modulus, tsf
- α_c = correlation factor depending on soil type and the cone bearing resistance, Table 3-9.
- q_c = cone tip bearing resistance, tsf

A typical value for sands is $\alpha_c = 3$, but can increase substantially for overconsolidated sand. A typical value for clays is $\alpha_c = 10$ when used with the net cone resistance $q_c - \sigma_o$ where σ_o is the total overburden pressure. The undrained shear strength C_u is related to q_c by

Equation 3-5: $$C_u = \frac{q_c - \sigma_o}{N_k}$$

Where

- C_u = undrained shear strength, tsf
- q_c = cone tip resistance, tsf
- σ_o = total overburden pressure, tsf
- N_k = cone factor

The cone factor often varies from 10 to 20 and can be greater.

Table 3-9 Correlation Factor α_c[47][48]

Soil	*Resistance q_c, ksf*	*Water Content, percent*	α_c
Lean clay (CL)	<14		3 to 8
	14 to 40		2 to 5
	>40		1 to 2.5
Silt (ML)	<40		3 to 6
	>40		1 to 3
Plastic silt clay (CH,MH)	<40		2 to 6
Organic silt	<24		2 to 8
Organic clay peat	<14	50 to 100	1.5 to 4
		100 to 200	1 to 1.5
		>200	0.4 to 1
Sand	<100		2 to 4
	>200		1.5
			$1 + D_r^2$
Clayey sand	6 to 12		
Silty sand	2 to 4		
Chalk	40 to 60	2 to 4	

[47] Mitchell, J. K. and Gardner, W. S. 1975. "*In situ* Measurement of Volume Change Characteristics," SOA Report, Proceedings American Society of Civil Engineers Special Conference on the *In situ* Measurement of Soil Properties, Raleigh, NC. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

[48] Note: D_r = relative density, fraction

3.3.2.4.3. Standard penetration test (SPT)

The elastic modulus in sand may be estimated directly from the blow count by

Equation 3-6: $$E_s = 9.4N^{0.87}\sqrt{B}\left(1 + 0.4\frac{D}{B}\right)$$

Where

- E_s = Young's soil modulus, tsf
- N = average blow count per foot in the stratum, number of blows of a 140 pound hammer falling 30 inches to drive a standard sampler (1.42" ID, 2.00" OD) one foot. Sampler is driven 18 inches and blows counted the last 12 inches.
- B = width of footing, ft
- D = depth of embedment of footing, ft

Equation 3-6 was developed from information in the literature and original settlement observations without consideration of the energy of the hammer. An alternative method of estimating the elastic modulus for footing foundations on clean sand or sand and gravel is[49]

Equation 3-7: $$E_m = 420 + 10N_{ave}$$ **(Preloaded sand)**

Equation 3-8: $$E_m = 194 + 8N_{ave}$$ **(Normally loaded sand or sand and gravel)**

Where

- E_m = deformation modulus
- N_{ave} = average measured blow count in depth H = B below footing, blows/ft

3.3.2.4.4. Pressuremeter test (PMT)

The preboring pressuremeter consists of a cylindrical probe of radius R_o containing an inflatable balloon lowered into a borehole to a given depth. The pressure required to inflate both the balloon and the probe against the side of the borehole and the volume change of the probe are recorded. The self-boring pressuremeter includes cutting blades at the head of the device with provision to permit drilling fluids to circulate and carry cuttings up to the surface. The self-boring pressuremeter should in theory lead to a less disturbed hole than the preboring pressuremeter. The pressure and volume change measurements are corrected for membrane resistance and volume losses leading to the corrected pressuremeter curve, Figure 3-7. The preboring pressuremeter curve indicates a pressuremeter modulus E_i that initially increases with increasing radial dimensional change, $\Delta R/R_o$, as shown in Figure 3-7. The self-boring pressuremeter curve is characteristic of an initially high pressuremeter modulus E_i that decreases with increasing volume change without the initial increasing modulus shown in the figure. The pressuremeter modulus is a measure of twice the shear modulus. If the soil is perfectly elastic in unloading, characteristic of a sufficiently small unload/reload cycle, the gradient will be $2G_{UR}$[50]. The unload-reload modulus should be determined on the plastic portion of the pressuremeter curve.

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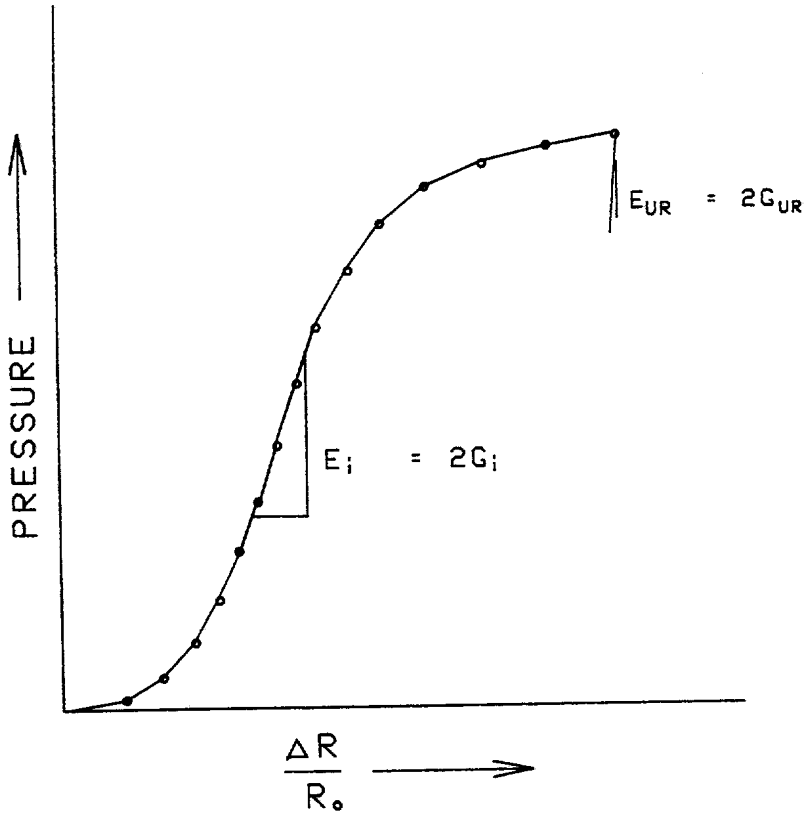
[49] D'Appolonia, D. J., D'Appolonia, E. D., and Brissette, R. F. 1970. "Closure: Settlement of Spread Footings on Sand," Journal of the Soil Mechanics and Foundations Division, Vol 96, pp 754-762. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY

[50] Hughes, J. M. O. 1982. "Interpretation of Pressuremeter Tests for the Determination of Elastic Shear Modulus," Proceedings of Engineering Foundation Conference on Updating Subsurface Sampling of Soils and Rocks and Their

Figure 3-7 Example corrected preboring pressuremeter curve



The pressuremeter modulus may be evaluated from the gradient of the unload/reload cycle by (ASTM 4719)

$$\textbf{Equation 3-9: } E_p = \frac{(1+\nu_s)\Delta P(E_{pc} + \Delta R_{pm})}{\Delta R_p}$$

- Where
- ν_s = soil Poisson's ratio, 0.33
- ΔP = change in pressure measured by the pressuremeter, tsf
- R_{po} = radius of probe, inches
- ΔR_{pm} = change in radius from R_{po} at midpoint of straight portion of the pressuremeter curve, inches
- ΔR_p = change in radius between selected straight portions of the pressuremeter curve, inches

3.3.2.5. Equivalent Elastic Modulus

The following two methods are recommended for calculating an equivalent elastic modulus of cohesive soil for estimating settlement of mats and footings.

In-Situ Testing, pp 279-289, Santa Barbara, CA. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY

3.3.2.5.1. Kay and Cavagnaro approximation

The equivalent elastic modulus E_s may be calculated by

Equation 3-10: $$E_s^* = \frac{2qR\left(1-\nu_s^2\right)}{\rho_c}$$

Where

- E^*_s = equivalent elastic modulus, tsf
- q = bearing pressure, tsf
- R = equivalent mat radius = $\sqrt{LB/\pi}$, L<2B, ft.
- L = length of mat, ft
- B = width of mat, ft
- ρ_c = centre settlement from the Kay and Cavagnaro method, Figure 7-12, ft

3.3.2.5.2. Semiempirical method

The equivalent elastic modulus of a soil with elastic modulus increasing linearly with depth may be estimated by

Equation 3-11: $$E_s^* = \frac{2kR\left(1-\nu_s^2\right)}{0.7+\left(2.3-4\nu_s\right)\log n}$$

Where

- k = constant relating soil elastic modulus E_s with depth z $E_s = E_o + kz$, tons/ft^3
- D = depth of foundation below ground surface, ft n = $kR/(E_o + kD_b)$
- E_o = elastic soil modulus at the ground surface, tsf

Equation 3-11 was developed from results of a parametric study using Equation 3-10.

3.3.2.5.3. Gibson model

The equivalent modulus of a soil with elastic modulus increasing linearly with depth and E_o = 0 is[51]

Equation 3-12: $$E_s^* = \frac{Bk}{2}$$

where B is the minimum width of the foundation, ft.

3.3.3. Shear Modulus

The shear modulus G may be used for analysis of settlement from dynamic loads.

[51] Gibson, R. E. 1967. "Some Results Concerning Displacements and Stresses in a Nonhomogeneous Elastic Half-Space," Geotechnique, Vol 17, pp 58-67. Available from Thomas Telford Ltd., 1-7 Great George Street, Westminster, London SW1P 3AA, England.

3.3.3.1. Definition

Shear stresses applied to an elastic soil will cause a shear distortion illustrated by the simple shear test (SST), Table 3-6.

3.3.3.2. Relationships with Other Parameters

Table 3-7 illustrates the relationship of the shear modulus with Young's elastic E and bulk modulus K.

3.3.4. Poisson's Ratio

A standard procedure for evaluation of Poisson's ratio for soil does not exist. Poisson's ratio ν_s for soil usually varies from 0.25 to 0.49 with saturated soils approaching 0.49. Poisson's ratio for unsaturated soils usually varies from 0.25 to 0.40. A reasonable overall value for ν_s is 0.40. Normal variations in elastic modulus of foundation soils at a site are more significant in settlement calculations than errors in Poisson's ratio.

It can be shown[52] that the theoretical relationship between Poisson's Ratio and the lateral earth pressure coefficient is given by the equation

Equation 3-13: $$K_o = \frac{\nu}{1-\nu}$$

3.3.5. Lateral modulus of subgrade reaction

The modulus of horizontal subgrade reaction E_{ls} is required for evaluation of lateral displacements of both driven and drilled piles and is given by

Equation 3-14: $$E_{ls} = \frac{p}{y}$$

where

- p = lateral soil reaction at a point on the pile per unit length, kips/ft
- y = lateral displacement, ft

The E_{ls} is approximately $67C_u$ for cohesive soil, and for granular or normally consolidated clays is

Equation 3-15: $$E_{ls} = K_s z$$

where

- K_s = constant relating E with depth z, ksf/ft
- z = depth, ft

The value of K_s is recommended to be about 40, 150, and 390 ksf/ft for loose, medium, and dense dry or moist sands, respectively, and 35, 100, and 210 ksf/ft for submerged sands[53] The value of K_s is also recommended to be about 500, 1,700, and 5,000 ksf/ft for stiff clays with average undrained shear strength of 1 to 2, 2 to 4, and 4 to 8 ksf, respectively.

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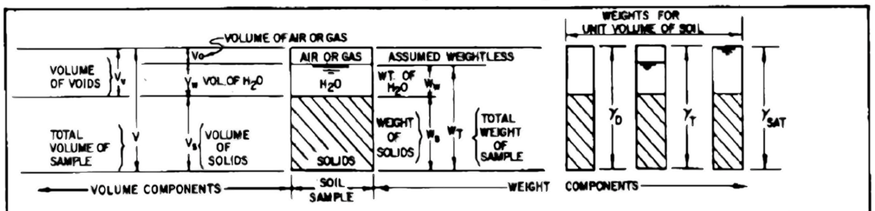
[52] Tschebotarioff, G.A. *Soil Mechanics, Foundations and Earth Structures*. New York: McGraw-Hill Book Company, 1951, pp. 248-9.

[53] After FHWA-RD-85-106, "Behavior of Piles and Pile Groups Under Lateral Load."

3.4. Moisture Content, Unit Weight, and Specific Gravity

A soil mass is considered to consist of solid particles enclosing voids of varying sizes. The voids may be filled with air, water, or both. The fundamental relations of the weights and volumes of the various components of a soil mass can be derived using the simplified sketches shown in Table 3-10. Some of the more important relations used in soils engineering calculations are water content, unit weights, void ratio, porosity, and degree of saturation. The quantities that must be known to compute these relations are the weight and volume of the wet specimen, the weight of the same specimen after oven drying, and the specific gravity of the solids. The weights of the specimens usually can be obtained without difficulty.

Table 3-10 Volume and Weight Relationships



		PROPERTY	SATURATED SAMPLE (W_s, W_w, G, ARE KNOWN)	UNSATURATED SAMPLE (W_s, W_w, G, V, ARE KNOWN)	SUPPLEMENTARY FORMULAS RELATING MEASURED AND COMPUTED FACTORS			
VOLUME COMPONENTS	V_s	VOLUME OF SOLIDS	$\frac{W_s}{G\gamma_w}$		$V-(V_a+V_w)$	$V(1-n)$	$\frac{V}{(1+e)}$	$\frac{V_v}{e}$
	V_w	VOLUME OF WATER	$\frac{W_w}{\gamma_w}$		V_v-V_a	SV_v	$\frac{SV_e}{(1+e)}$	$SV_s e$
	V_a	VOLUME OF AIR OR GAS	ZERO	$V-(V_s+V_w)$	V_v-V_w	$(1-S)V_v$	$\frac{(1-S)V_e}{(1+e)}$	$(1-S)V_s e$
	V_v	VOLUME OF VOIDS	$\frac{W_w}{\gamma_w}$	$V-\frac{W_s}{G\gamma_w}$	$V-V_s$	$\frac{V_s n}{1-n}$	$\frac{V_e}{(1+e)}$	$V_s e$
	V	TOTAL VOLUME OF SAMPLE	V_s+V_w	MEASURED	$V_s+V_a+V_w$	$\frac{V_s}{1-n}$	$V_s(1+e)$	$\frac{V_v(1+e)}{e}$
	n	POROSITY	$\frac{V_v}{V}$		$1-\frac{V_s}{V}$	$1-\frac{W_s}{GV\gamma_w}$	$\frac{e}{1+e}$	
	e	VOID RATIO	$\frac{V_v}{V_s}$		$\frac{V}{V_s}-1$	$\frac{GV\gamma_w}{W_s}-1$	$\frac{W_w G}{W_s S}$	$\frac{n}{1-n}$; $\frac{wG}{S}$

		PROPERTY	SATURATED SAMPLE (W_s, W_w, G, ARE KNOWN)	UNSATURATED SAMPLE (W_s, W_w, G, V, ARE KNOWN)	SUPPLEMENTARY FORMULAS RELATING MEASURED AND COMPUTED FACTORS			
WEIGHTS FOR SPECIFIC SAMPLE	W_s	WEIGHT OF SOLIDS	MEASURED		$\frac{W_T}{(1+W)}$	$GV\gamma_w(1-n)$	$\frac{W_w G}{eS}$	
	W_w	WEIGHT OF WATER	MEASURED		wW_s	$S\gamma_w V_v$	$\frac{eW_s S}{G}$	
	W_t	TOTAL WEIGHT OF SAMPLE	$W_s + W_w$		$W_s(1+w)$			
WEIGHTS FOR SAMPLE OF UNIT VOLUME	γ_D	DRY UNIT WEIGHT	$\frac{W_s}{V_s + V_w}$	$\frac{W_s}{V}$	$\frac{W_t}{V(1+w)}$	$\frac{G\gamma_w}{(1+e)}$	$\frac{G\gamma_w}{1+wG/S}$	
	γ_T	WET UNIT WEIGHT	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+W_w}{V}$	$\frac{W_T}{V}$	$\frac{(G+Se)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w/S+1/G}$	
	γ_{SAT}	SATURATED UNIT WEIGHT	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+V_v\gamma_w}{V}$	$\frac{W_s}{V}+\left(\frac{e}{1+e}\right)\gamma_w$	$\frac{(G+e)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w+1/G}$	
	γ_{SUB}	SUBMERGED (BUOYANT) UNIT WEIGHT	$\gamma_{SAT} - \gamma_w$		$\frac{W_s}{V}-\left(\frac{1}{1+e}\right)\gamma_w$	$\left(\frac{G+e}{1+e}-1\right)\gamma_w$	$\left(\frac{1-1/G}{w+1/G}\right)\gamma_w$	
COMBINED RELATIONS	w	MOISTURE CONTENT	$\frac{W_w}{W_s}$		$\frac{W_t}{W_s}-1$	$\frac{Se}{G}$	$S\left[\frac{\gamma_w}{\gamma_D}-\frac{1}{G}\right]$	
	S	DEGREE OF SATURATION	1.00	$\frac{V_w}{V_v}$	$\frac{W_w}{V_v\gamma_w}$	$\frac{wG}{e}$	$\frac{w}{\left[\frac{\gamma_w}{\gamma_D}-\frac{1}{G}\right]}$	
	G	SPECIFIC GRAVITY	$\frac{W_s}{V_s\gamma_w}$		$\frac{Se}{w}$			

Where:

- w = Water or moisture content = the ratio, expressed as a percentage, of the weight of water in a given soil mass to the weight of solid particles.
- γ_d = Dry unit weigh = the weight of oven dried soil solids per unit of total volume of soil mass, and is usually expressed in pounds per cubic foot.
- γ_m = Wet unit weight = the weight (solids plus water) per unit of total volume of soil mass, irrespective of the degree of saturation. The wet unit weight is usually expressed in pounds per cubic foot.
- e = Void ratio = the ratio of the volume of voids to the volume of solid particles in a given soil mass.
- n = Porosity = , the ratio (usually expressed as a percentage) of the volume of voids of a given soil mass to the total volume of the soil mass.
- S = Degree of saturation = the ratio (expressed as a percentage) of the volume of water in a given soil mass to the total volume of voids.
- Specific Gravity. The specific gravity of a soil mass for use in soils engineering calculations is usually expressed in three different forms:
    a. The specific gravity of solids, G_s, applied to soils finer than the No. 4 sieve. This is the ratio of the weight in air of a given volume of soil particles at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.
    b. The apparent specific gravity, G_a, which is the ratio of the weight in air of a given volume of the impermeable portion of a permeable material (that is, the solid matter including its impermeable pores or voids) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

c. The bulk specific gravity, G_m, which is the ratio of the weight in air of a given volume of a permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature. Both (b) and (c) are applied to soils coarser than the No. 4 sieve.

The specific gravity of solids is not applied to coarse particles because they normally contain voids from which air cannot be displaced unless the particles are ground into finer particles to eliminate the voids. Thus, when dealing with coarser particles it is more convenient to work with the apparent specific gravity of the particle mass. The value G_s or G_a is used in all calculations involving fundamental properties of a soil mass. The bulk specific gravity is used in special calculations, such as corrections of density and water content for soils containing gravel sizes.

3.4.1. Water Content Test - General

3.4.1.1. Apparatus

The apparatus should consist of the following:

a. Oven, preferably of the forced-draft type, automatically controlled to maintain a uniform temperature of 110 ± 5 ºC throughout the oven. Figure 3-8 shows such an oven.

Figure 3-8 Drying Oven for Water Content Test

b. Balances, sensitive to 0.01 g for samples weighing less than 50 g; 0.1 g for samples weighing 50 g to 500 g; 1.0 g for samples weighing over 500 g. Such a balance is shown in Figure 3-9.

Figure 3-9 Balance

c. Specimen Container. Seamless metal containers with lids are recommended. The containers should be of a metal resistant to corrosion (aluminum is satisfactory). They should be as small and light in weight as practicable in relation to the amount of material to be used in the determination. For routine water content determinations in which specimens weighing between 100 and 200 g are used, a 2" high by 3 1/2" diameter container is adequate.

3.4.1.2. Specimen

The amount of material used in the water content determination will generally depend on the maximum size of particles, the amount of material available, and the requirement that the specimen be representative of the material for which the determination is made.

When the water is not uniformly distributed throughout the sample, larger specimens will be needed than would otherwise be required. For routine water content determinations on material passing a No. 4 sieve, specimens weighing between 100 and 200 g are adequate. A minimum specimen weight of 500 g is recommended for material having a maximum particle size in the range of the No. 4 to 3/4" sieves, and a minimum specimen weight of 1 kg is recommended for material having a maximum particle size in the range of the 3/4" to 1 1/2" sieves. Specific amounts of material are required for water content determinations for other laboratory tests; the test procedures should be consulted to determine the proper amounts.

3.4.1.3. Procedure

The procedure shall consist of the following steps:

a. Record all identifying information for the specimen, such as project, boring number, sample number, or other pertinent data, on a data sheet (Figure 11-1 is a suggested form).

b. Record the number and tare weight of the specimen container.

c. Place the specimen in the container, set the lid securely in position and immediately determine the weight of the container and wet soil by weighing on an appropriate balance.

d. Before the specimen is placed in the oven, remove the lid; the lid is usually placed under the container in the oven. Then place the specimen and container in the oven heated to 110 ± 5 ºC.[54] Leave the specimen in the oven until it has dried to a constant weight. The time required for drying will vary depending on the type of soil, size of specimen, oven type and capacity, and other factors. Both good judgment and experience with the soils being tested and the equipment available in the laboratory can generally establish the influence of these factors. When in doubt, reweigh the oven-dried specimens at periodic intervals to establish the minimum drying time required to attain a constant weight.

e. For routine water content determinations, specimens consisting of clean sands and gravels should be oven dried for a minimum of 4 hours. For most other soils, a minimum drying time of 16 hours is adequate. Dry soil may absorb moisture from wet specimens; therefore, any dried specimens must be removed before wet specimens are placed in the oven.

f. After the specimen has dried to constant weight, remove the container from the oven and replace the lid. Allow the specimen to cool until the container can be handled comfortably with bare hands. If the specimen cannot be weighed immediately after cooling it should be placed in a desiccator; if a sample is left in the open air for a considerable length of time it will absorb moisture.

g. After the specimen has cooled, determine its dry weight and record it on the data sheet.

3.4.1.4. Computations

The following quantities are obtained by direct weighing:

a. Weight of tare plus wet soil ($W_{ws} + W_t$), g.

b. Weight of tare plus dry soil ($W_t + W_s$), g. The water content in percent of oven-dry weight of the soil is equal to:

Equation 3-16: $$w = 100\frac{(W_{ws} + W_t) - (W_t + W_s)}{(W_t + W_s) - W_t} = 100\frac{W_w}{W_s}$$

Where

- w = water content, percent
- W_{ws} = weight of wet soil
- W_w = weight of water
- W_s = weight of dry soil
- W_t = weight of tare

[54] Laboratory oven drying at 110º C does not result in reliable water content values for soils containing gypsum or significant amounts of organic material. Reliable water content values for these soils can be obtained by drying in oven at 60º C, or by vacuum desiccation. See: U. S. Army Engineer Waterways Experiment Station, CE, *A Study of Moisture - Content Determinations on Selected Soils*, Miscellaneous Paper No. 4-73 (Vicksburg, Miss., September 1954).

3.4.1.5. Possible Errors

Following are possible errors that would cause inaccurate determinations of water content:

a. Specimen not representative. The specimen must be representative of the sample as required for the purpose of the determination. For example, a stratified soil may have a great variation in, water content between adjacent strata; were it intended to evaluate the strength of the soil based on water content, a large specimen that included material from several strata would not be representative of the weakest stratum. As another example, to determine the average water content of gravelly clay, the specimen must be large enough to contain representative amounts of both coarse and fine fractions.

b. Specimen too small. As a rule, the larger the specimen, the more accurate the determination because of the larger weights involved.

c. Loss of moisture before weighing wet specimen. Even in a covered container, a specimen can lose a significant amount of water unless weighed within a short period.

d. Incorrect temperature of oven. The oven-dry weight of many soils is dependent on the temperature of the oven, so variations in temperature throughout the interior of an oven can cause large variations in the computed water content.[55]

e. Specimen removed from oven before obtaining a constant oven dry weight.

f. Gain of moisture before weighing oven-dry specimen.

g. Weighing oven-dry specimen while still hot. A hot specimen container may affect the accuracy of a sensitive balance.

h. Incorrect tare weight. The weights of specimen containers should be checked periodically and should be scratched on the containers to avoid possible errors in reading such weights from lists.

3.4.2. Unit Weights, Void Ratio, Porosity, and Degree of Saturation

3.4.2.1. Relations

The volume of the wet specimen is determined by linear measurement (volumetric method), or by measurements of the volume or weight of water displaced by the specimen (displacement method.) Definitions of the relations to be determined can be found at 3.3; detailed procedures for determining these values, using the volumetric and displacement methods, are given in the following paragraphs.

3.4.2.2. Unit Weight Test: Volumetric Method

3.4.2.2.1. Description

The volumetric method consists of computing the total volume of soil from linear measurements of a regularly shaped mass. In general, the method is applied to soils that can be cut or formed into a cylinder or parallelepiped. Specimens of this type are used for other laboratory tests, and methods for preparing them are described under the individual test procedures. Progressive trimming in front of a calibrated ring-shaped specimen cutter forms the base of the procedure presented below for obtaining a cylindrical specimen. However, other methods for obtaining a regularly shaped mass, such as cutting and trimming or punching, can often be used successfully. The volumetric method should not be used for soils containing gravel, shells, or foreign materials that would interfere with advance trimming. The calibrated

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[55] T. W, Lambe, Soil Testing for Engineers, John Wiley & Sons, Inc.

specimen cutter method is particularly suitable for obtaining volumes of silty and sandy soils having little cohesion.

3.4.2.2.2. Apparatus

The apparatus should consist of the following:

(1) Calibrated ring-shaped specimen cutter, hereinafter referred to as a volumetric cylinder. Types and sizes of volumetric cylinders may vary widely; two types are shown in Figure 3-10. General requirements are that a volumetric cylinder be made of materials not susceptible of rapid corrosion and that it be as large as possible in relation to the samples being tested. The inside of the cylinder should be polished to a smooth finish, and sharp cutting edges should be provided on the base. It is very important that no voids form between the sample and cylinder; to facilitate detection of such voids, a volumetric cylinder of transparent Lucite with detachable steel cutting edges may be used.

(2) Guide cylinder for guiding cutter into soil (not absolutely necessary).

(3) Trimming tools, such as wire saw, straightedge, or knife.

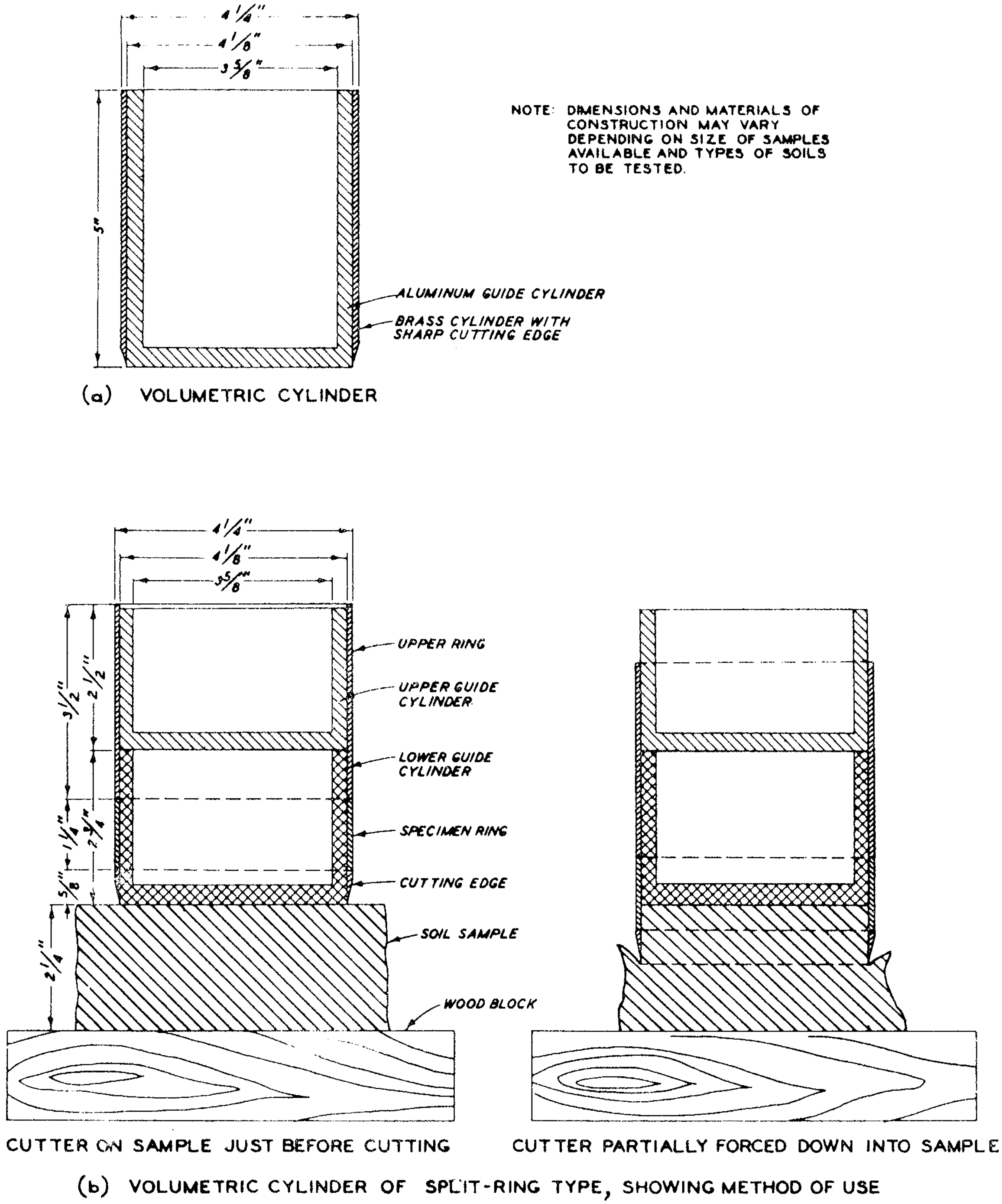
(4) Oven (see 3.3).

(5) Specimen container. The container should be of metal that is resistant to corrosion. Seamless aluminum pans with lids are satisfactory.

(6) Balance, sensitive to 0.1 g.

(7) Glass plate, large enough to cover top of specimen.

Figure 3-10 Examples of Volumetric Cylinders



3.4.2.2.3. Procedure

The procedure shall consist of the following steps:

(1) Record on a data sheet (Figure 11-2 is a suggested form) all identifying information for the sample, such as project, boring number and other pertinent data.

(2) Measure and record the height, H, and inside diameter, D, of the volumetric cylinder. In general, linear measurements shall be made with an accuracy that will result in a volumetric error, dV/V, of less than 1%. The volumetric error is represented by the expression:

Equation 3-17: $$\frac{dV}{V}, percent = 200\frac{dD}{D} + 100\frac{dH}{H}$$

Where

- ∞ dH = accuracy of height measurement
- ∞ dD = accuracy of diameter measurement.

(3) Centre the volumetric cylinder on top of the sample. The sample may be roughly trimmed to a size somewhat larger than the cylinder (see Figure 3-10b) or the entire available sample may be used (see Figure 3-11). Push the cylinder vertically into the sample not more than 1/4" and carefully trim the soil from the edge of the cylinder (see Figure 3-11). Repeat the operation until the specimen protrudes above the top of the calibrated cylinder. Care should be taken that no voids are formed between the cylinder and specimen. Using a wire saw for soft specimens and a knife, straightedge, or other convenient tool for harder specimens, trim the top of the specimen flush with the top of the cylinder. Invert the specimen, place it on a glass plate, and trim the bottom of the specimen.

(4) Remove the specimen from the volumetric cylinder using a guide cylinder, if available, and place it in a container. Weigh the specimen and record this weight on the data sheet as the weight of tare plus wet soil. Alternatively, weighing the volumetric cylinder with the specimen therein and then removing the material and placing it in a container for a water content determination may determine the wet weight of the specimen.

(5) Place the soil and container in an oven and oven-dry the at 110º C ± 5º C, allow it to cool, and then weigh. Record this weight of tare plus dry soil.

Figure 3-11 Determining the Unit Weight of a Soil Specimen with the Volumetric Cylinder (split-ring type)

3.4.2.2.4. Computation

(1) Quantities obtained in test. The following quantities are obtained in the test:

(a) Weight of tare (specimen container or cylinder) plus wet soil. The tare weight is subtracted from this value to obtain the weight of wet soil, W.

(b) Weight of tare (specimen container) plus dry soil. The tare weight is subtracted from this value to obtain the weight of dry soil, W_s, or if the alternate procedure is used, dry weight of specimen is computed by the following formula:

$$\text{Dry Weight Of Specimen} = \frac{\text{Wet Weight Of Specimen}}{1 + \frac{\text{Water Content Of Specimen}}{100}}$$

Equation 3-18:

$$W_s = \frac{W}{1 + \frac{w}{100}}$$

(c) The inside volume of the volumetric cylinder. Volume, V, of the wet soil specimen is equal to this volume. The volume, V, may also be computed from linear measurements of a specimen in the form of a cylinder or parallelepiped.

(2) Unit weights. The wet unit weight, γ_m, and the dry unit weight, γ_d, expressed in terms of pounds per cubic foot.

(3) Void ratio.

(4) Porosity.

(5) Degree of saturation.

See Table 3-10 for equations to compute these values.

3.4.2.3. Unit Weight Test: Displacement Method

3.4.2.3.1. Description

The displacement method consists of determining the total volume of a soil by measuring the volume or weight of water displaced by the soil mass. The method particularly adaptable to irregularly shaped specimens and soils containing gravel, shells, etc.

3.4.2.3.2. Apparatus

The apparatus should consist of the following:

(1) Balance, sensitive to 0.1 g.

(2) Wire basket of sufficient size to contain the soil specimen.

(3) Can, or container, of sufficient size to submerge the wire basket and specimen.

(4) Oven (see 3.3).

(5) Specimen container. The container should be of metal that is resistant to corrosion. Seamless aluminium pans with lids are satisfactory.

(6) Paintbrush.

(7) Microcrystalline wax or paraffin.[56]

(8) Container for melting wax, preferably with a self-contained thermostat.

(9) Thermometer, range 0º to 50º C, graduated in 0.1º C.

3.4.2.3.3. Procedure

The procedure shall consist of the following steps:

(1) Record all identifying information for the sample, such as project, boring number, or other pertinent data, on the data sheet (Figure 11-3).

(2) Determine, if not previously established, the specific gravity of the wax to be used. (About 0.9 g/cm^3, but should be determined for each batch of wax.)

(3) Cut a specimen from the sample to be tested. (The size of the specimen is not very important provided the capacity of the balance is not exceeded. In general, results that are more accurate will be obtained with larger specimens.) Trim the specimen to a regular shape. Re-entrant angles should be avoided, and any cavities formed by large particles being pulled out should be patched carefully with material from the trimmings.

(4) Determine and record the wet weight of the soil specimen.

(5) Cover the specimen with a thin coat of melted wax, either with a paintbrush or by dipping the specimen in a container of melted wax. Apply a second coat of wax after the first coat has hardened. The wax should be sufficiently warm to flow when brushed on the soil specimen, yet it should not be so hot that it penetrates the pores of the soil. If hot wax encounters the soil specimen, it may cause the moisture to vaporise and form air bubbles under the wax.

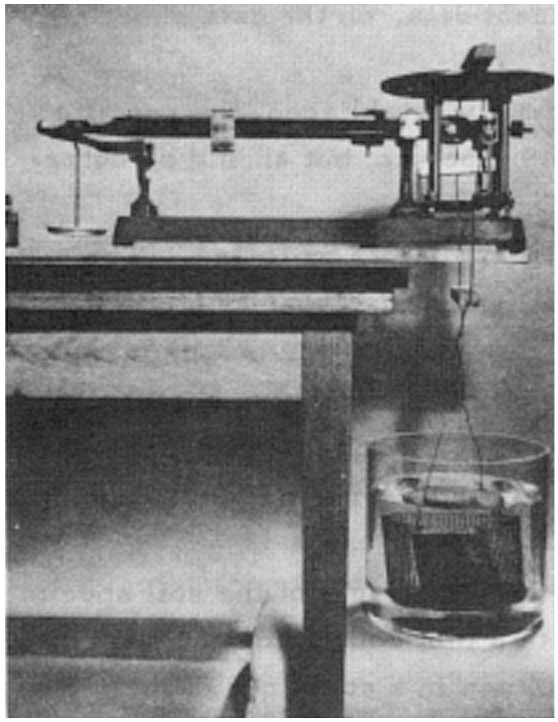
(6) Determine and record the weight of the wax-coated specimen in air.

(7) Determine and record the submerged weight of the wax-coated specimen. This is done by placing the specimen in a wire basket hooked onto a balance and immersing the basket and specimen in a can of water as shown in Figure 3-12. In order to directly measure the submerged weight of the wet soil and wax, the balance must have been previously balanced with the wire basket completely submerged in the can of water. Ensure that the specimen is fully submerged, and that the basket is not touching the sides or bottom of the container. Measure the temperature of the water.

(8) Remove the wax from the specimen. It can be peeled off after a break is made in the wax surface. Use the entire sample, or as much as is free of wax inclusions, for a water content determination (see 3.3).

[56] Among the many microcrystalline waxes found satisfactory are Product 2300 of the Mobil Oil Company, Microwax 75 of the Gulf Oil Corporation, and Wax 1290 of the Sun Oil Company. Paraffin alone is not as suitable for sealing soil specimens because its brittleness and shrinkage upon cooling will cause cracking, especially in thin sections and at corners; a mixture of 50% paraffin and 50% petrolatum has properties that approach those of a microcrystalline wax.

Figure 3-12 Determining the Weight of a Wax-Coated Specimen Submerged in Water



3.4.2.3.4. Computation

The following quantities are obtained directly in the test:

(1) Weight of uncoated specimen, W.

(2) Weight of soil plus wax. The weight of uncoated specimen, W, is subtracted from this value to obtain the weight of wax.

(3) Weight of soil plus wax in water

The following computations shall be made:

(1) Divide the weight of the wax by its specific gravity. This gives the volume of the wax.

(2) Subtract the weight of the wax-coated specimen in water from its weight in air. The difference divided by the density of water at the test temperature (see Table 3-11) is numerically equal to the volume of the coated specimen in cubic centimeters.

(3) Subtract the volume of wax from the volume of the coated specimen to obtain the total volume of the soil specimen, V.

(4) Compute the water content of the specimen (see 3.3). If the entire specimen is used for the water content determination, obtain the dry weight of specimen, Ws, directly. If only a portion of the initial specimen is used for the water content determination, compute the dry weight of specimen the following formula:

$$\text{Dry Weight Of Specimen} = \frac{\text{Wet Weight Of Uncoated Soil}}{1 + \frac{\text{Water Content Of Wet Soil}}{100}}$$

Equation 3-19:

$$W_s = \frac{W}{1 + \frac{w}{100}}$$

Based on the above information, compute the unit weights, void ratio, porosity, and degree of saturation as specified previously.

3.4.2.4. Possible Errors

Following are possible errors that would cause inaccurate determinations of the total volume:

3.4.2.4.1. Volumetric Method

(1) Imprecise measurement of volumetric cylinder (or of cylindrical specimen trimmed by other methods). Three height measurements and nine diameter measurements should be made to determine the average height and diameter of the cylinder. Precise calipers should be used for these measurements rather than flat scales.

(2) Voids formed on side of specimen by trimming beyond cutting edge.

(3) Material lost while removing specimen from cylinder.

3.4.2.4.2. Displacement Method

Voids on surface of specimen not filled by wax or air bubbles formed beneath wax.

3.4.3. Specific Gravity Test

Definitions and detailed procedures for determining the values of specific gravity of solids, apparent specific gravity, and bulk specific gravity are given below.

3.4.3.1. Specific Gravity of Solids

3.4.3.1.1. Apparatus

The apparatus should consist of the following:

(1) Volumetric flask, 500 cm^3 capacity

(2) Vacuum pump, with piping and tubing for connections to each flask (as shown in Figure 3-14). The connection to each flask should be provided with a trap to catch any water drawn from the flask.

(3) Oven (see 3.3)

(4) Balance, sensitive to 0.01 g.

(5) Thermometer, range 0 to 50º C, graduated in 0.1º C.

(6) Evaporating dish.

(7) Water bath.

(8) Sieve, U. S. Standard No. 4 conforming to ASTM Designation: E 44, Standard Specifications for Sieves for Testing Purposes

3.4.3.1.2. Calibration of Volumetric Flask

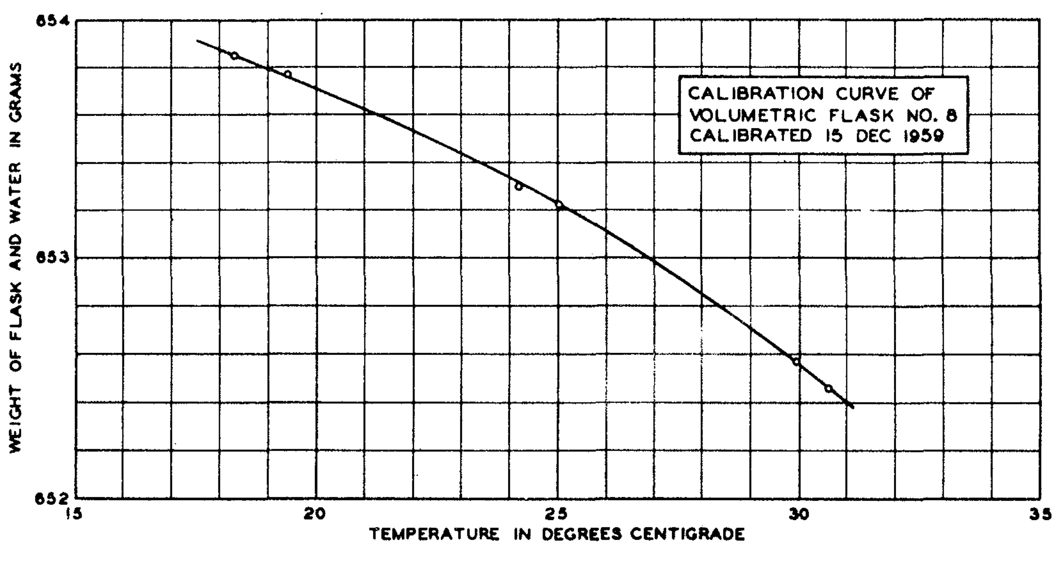
The volumetric flask shall be calibrated for the weight of the flask and water at various temperatures. The flask and water are calibrated by direct weighing at the range of b temperatures likely to be encountered in the laboratory. The calibration procedure is as follows:

(1) Fill the flask with deaired-distilled (or deaired-demineralized) water to slightly below the calibration mark and place in a water bath that is at a temperature between 30º and 35º C. Allow the flask to remain in the bath until the water in the flask reaches the temperature of the water bath. This may take several hours. Remove the flask from the water bath, and adjust the water level in the flask so that the bottom of the meniscus is even with the calibration mark on the neck of the flask. Thoroughly dry the outside of the flask and remove any water adhering to the inside of the neck above the graduation; then weigh the flask and water to the nearest 0.01 g. Immediately after weighing, shake the flask gently and determine the temperature of the water to the nearest 0.1º C by immersing a thermometer to the mid-depth of the flask.

(2) Repeat the procedure outlined in step (1) at approximately the same temperature. Then make two more determinations, one at room temperature and the other at approximately 5º less than room temperature.

(3) Draw a calibration curve showing the relation between temperature and corresponding weights of the flask plus water. Prepare a calibration curve for each flask used for specific gravity determinations and maintain the curves as a permanent record. A typical calibration curve (omitting the fine grid necessary for accurate determinations) is shown in Figure 3-13.

Figure 3-13 Typical Calibration Curve of Volumetric Flask



3.4.3.1.3. Preparation of Sample

Particular care should be taken to obtain representative samples for determination of specific gravity of solids. The sample of soil may be at its natural water content or oven dried; however, some soils, particularly those with a high organic content, are sometimes difficult to rewet after having been oven dried. These soils may be tested without first being oven dried, in which case the oven-dry weight of sample is determined at the end of the test. When the sample contains particles both larger and smaller than the No. 4 sieve, the sample shall be separated on the No. 4 sieve and a determination made of the specific gravity of the fine fraction and the apparent specific gravity of the coarse fraction. The specific gravity value for the sample shall be the composite specific gravity relation based on the solid volume of the components. When the specific gravity value is to be used in calculations in connection with the hydrometer analysis (see 3.6), the specific gravity shall be determined on that portion o f the soil used for the hydrometer analysis (usually that which passes the No. 200 sieve). It may be necessary to use other liquids such as kerosene) in lieu of distilled water for testing soils containing soluble salts.

3.4.3.1.4. Procedure

(1) Soils at natural water content. The procedure for determining the specific gravity of soils at natural water content shall consist of the following steps:

a. Record all identifying information for the sample such as project, boring number, sample number, and other pertinent data, on a data sheet (see Figure 11-4 for suggested form).

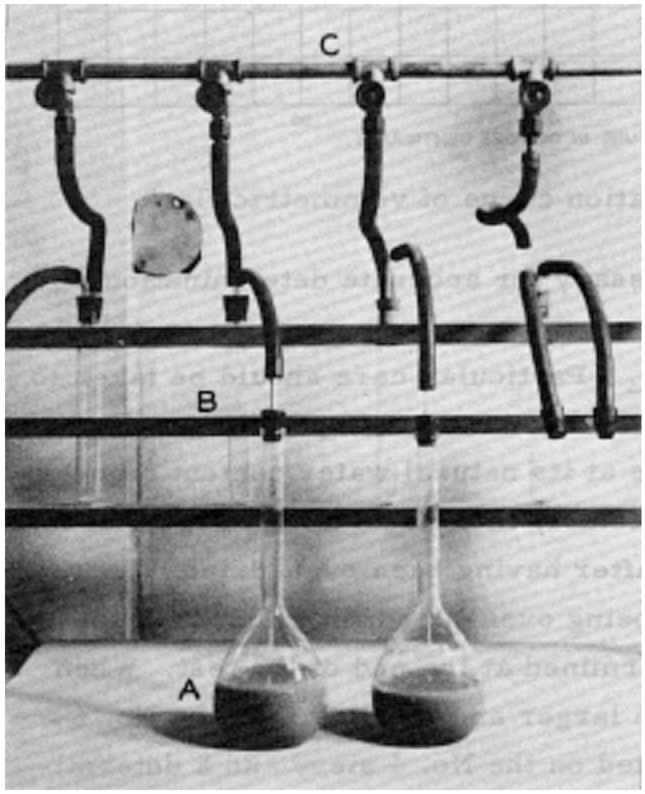
b. Place a representative sample of soil equivalent to approximately 50 to 86 g oven dry weight in a dish and, by means of a spatula, mix with sufficient distilled or demineralized water to form a slurry. Place the slurry in a volumetric flask and fill the flask approximately half full with distilled water.

c. Connect the flask to the vacuum line as shown in Figure 3-14 and apply a vacuum of approximately 29.0" of mercury. Agitate the flask gently at intervals during the evacuation process; commercially available mechanical agitators have been used for this purpose. The length of time that vacuum should be applied will depend on the type of soil being tested. Soils of high plasticity and organic[57] soils usually require 6 to 8 hours; some soils may require less time for removal of air but this should be verified by experimentation. To ensure continuous boiling, the temperature of the flask and contents may be elevated somewhat above room temperature by immersing in a water bath at approximately 35º C. Alternatively, entrapped air may be removed by boiling[58] the suspension gently for at least 10 minutes while occasionally rolling the flask to assist in the removal of air" The boiling process should be observed closely as loss of material may occur. Allow flask and contents to cool, preferably overnight, before filling and checking.

[57] Air removal from organic soils usually cannot be accomplished by the application of vacuum. In this case it will be necessary to boil the suspension contained in the flask for about 30 min, adding distilled or demineralised water carefully from time to time to prevent boiling the sample dry. The flask should at all times be approximately half full.

[58] Use of indirect heat such as a sand bath is recommended.

Figure 3-14 Evacuating air from samples in determination of specific gravity[59]



d. Fill the flask with deaired distilled water to about 3/4” below the 500 cm^3 graduation and apply a vacuum slightly less than that which will cause vigorous boiling (as vigorous boiling may result in a loss of solids). To determine if the suspension is desired, slowly release the vacuum and observe the lowering of the water surface in the neck of the flask. If the water surface is lowered less than 1/8”, the suspension can be considered sufficiently deaired.

e. Fill the flask until the bottom of the meniscus is coincident with the calibration line on the neck of the flask. Thoroughly dry the outside of the flask and remove the moisture *on* the inside of the neck by wiping with a paper towel. Weigh the flask and contents to the nearest 0.01 g. Immediately after weighing, stir the suspension to assure uniform temperature, and determine the temperature of the suspension to the nearest 0.1º C by immersing a thermometer to the mid-depth of the flask.

f. Carefully transfer the contents of the flask to an evaporating dish. Rinse the flask with distilled water to ensure removal of the entire sample from the flask. Oven-dry the sample to a constant weight at a temperature of 110º ± 5º C. Allow the soil to cool to room temperature in a desiccator and determine the weight of the soil to the nearest 0.01 g.

g. Record all weights on the data sheet.

(2) Oven dried soils. The procedure for determining the specific gravity of solids for oven dried soils shall consist of the following steps:

[59] A, flask; B, splash trap; C, vacuum line.

a. Record information identifying the sample on the data sheet (see Figure 11-4).

b. Oven-dry the soil to a constant weight at 110º ± 5º C and cool it to room temperature in a desiccator. Select a representative sample of between 50 g (for cohesive soils) and 150 g (for cohesionless soils) and weigh the sample to the nearest 0.04 g. After weighing, transfer the soil to a volumetric flask, taking care not to lose any material during this operation. To avoid possible loss of preweighed soil, the sample may be weighed after transfer to the flask. Fill the flask approximately half full with deaired distilled water and allow the suspension to stand overnight.

c. Connect the flask to the vacuum line and apply a vacuum of approximately 29.0" of mercury for approximately 2 to 4 hours. Entrapped air may also be removed by boiling as previously discussed; however, the process should be observed closely to avoid loss of material during before boiling. Allow flask and contents to cool, preferably overnight, filling and checking.

d. Perform the remainder of the test as outlined in paragraph (1)d and (1)e.

e. Record all weights on the data sheet.

3.4.3.1.5. Computation

The following quantities are obtained by direct weighing:

(1) Weight of flask plus water plus solids at test temperature = W_{bws} in grams.

(2) Weight of tare plus dry soil in grams. The tare weight is subtracted from this value to obtain the weight of dry soil, W_s. The specific gravity of solids is computed to two decimal places by the formula:

Equation 3-20: $$G_s = \frac{W_s K}{W_s + W_{bw} - W_{bws}}$$

Where

- ∞ K = correction factor based on the density of water at 20 C (see Table 3-11.). Unless otherwise required, specific gravity values reported shall be based on water at 20 C.
- ∞ W_{bw} = weight of flask plus water at test temperature .in grams (obtained from calibration curve as shown in Figure 3-13).

Table 3-11 Relative Density of Water and Correction Factor (K) for Various Temperatures

Temp °C	Relative Density	Correction Factor, K	Temp °C	Relative Density	Correction Factor, K	Temp °C	Relative Density	Correction Factor, K
18.0	0.99862	1.0004	23.0	0.99756	0.9993	28.0	0.99626	0.9980
.1	60	4	.1	54	3	.1	23	0
.2	58	4	.2	51	3	.2	20	0
.3	56	3	.3	49	3	.3	17	0
.4	54	3	.4	46	2	.4	14	79
.5	52	3	.5	44	2	.5	11	9
.6	50	3	.6	42	2	.6	08	9
.7	49	3	.7	39	2	.7	06	8
.8	47	2	.8	37	1	.8	03	8
.9	45	2	.9	34	1	.9	00	8
19.0	0.99843	1.0002	24.0	0.99732	0.9991	29.0	0.99597	0.9977
.1	41	2	.1	29	1	.1	94	7
.2	39	2	.2	27	0	.2	91	7
.3	37	1	.3	24	0	.3	88	6
.4	35	1	.4	22	0	.4	85	6
.5	33	1	.5	20	0	.5	82	6
.6	31	1	.6	17	89	.6	79	6
.7	29	1	.7	14	9	.7	76	5
.8	27	0	.8	12	9	.8	73	5
.9	25	0	.9	09	9	.9	70	5
20.0	0.99823	1.0000	25.0	0.99707	0.9988	30.0	0.99567	0.9974
.1	21	0	.1	04	8	.1	64	4
.2	19	0	.2	02	8	.2	61	4
.3	17	0.9999	.3	699	8	.3	58	3
.4	15	9	.4	97	7	.4	55	3
.5	13	9	.5	94	7	.5	52	3
.6	10	9	.6	91	7	.6	49	3
.7	08	8	.7	89	7	.7	46	2
.8	06	8	.8	87	6	.8	43	2
.9	04	8	.9	84	6	.9	40	2
21.0	0.99802	0.9998	26.0	0.99681	0.9986	31.0	0.99537	0.9971
.1	00	8	.1	78	6	.1	33	1
.2	798	8	.2	76	5	.2	30	1
.3	96	7	.3	73	5	.3	27	0
.4	93	7	.4	70	5	.4	24	0
.5	91	7	.5	68	4	.5	21	0
.6	89	7	.6	65	4	.6	18	69
.7	87	6	.7	63	4	.7	15	9
.8	85	6	.8	60	4	.8	12	9
.9	83	6	.9	57	3	.9	09	9
22.0	0.99780	0.9996	27.0	0.99654	0.9983	32.0	0.99505	0.9968
.1	78	6	.1	51	3	.1	02	8
.2	75	5	.2	48	2	.2	499	8
.3	73	5	.3	46	2	.3	96	7
.4	70	5	.4	43	2	.4	93	7
.5	68	5	.5	40	2	.5	90	7
.6	65	4	.6	37	1	.6	86	6
.7	63	4	.7	34	1	.7	83	6
.8	61	4	.8	32	1	.8	80	6
.9	58	4	.9	29	1	.9	77	5

* Note: Relative density of water based on density of water at 4 C equal to unity. The values given are numerically equal to the absolute density in grams/milliliter (for soil testing purposes, g/ml = g/cc). Data obtained from Smithsonian Tables, compiled by various authors.

Correction factor, K, is found by dividing the relative density of water at the test temperature by the relative density of water at 20 C.

3.4.3.2. Apparent and Bulk Specific Gravity

3.4.3.2.1. Apparatus

The apparatus should consist of the following:

(1) Balance, having capacity of 5 kg or more and sensitive to 1.0 g.

(2) Wire basket of No. 6 mesh, approximately 8" in diameter and 8" high.

(3) Suitable container for immersing the wire basket in water, and suitable apparatus for suspending the wire basket from the centre of the balance scale pan.

(4) Thermometer, range 0 to 50º C, graduated in 0.1º C.

3.4.3.2.2. Sample

The material to be tested shall be separated on the No. 4 sieve and the material retained on the sieve used for the test. A representative sample of approximately 2 kg is required. Samples may be air-dried; however, oven-drying the sample before the test may affect the results and should be avoided when possible.

3.4.3.2.3. Procedure

The procedure for determining the apparent and bulk specific gravity of soils retained on the No. 4 sieve shall consist of the following steps:

(1) Record information identifying the specimen on the data sheet (see Figure 11-4).

(2) Wash the specimen thoroughly to remove dust or other coatings from the surfaces of the particles.

(3) Immerse the specimen in water at 15 to 25º C for a period of 24 hours.

(4) Remove the specimen from the water and roll it in a large absorbent cloth until all visible films of water are removed, although the surfaces of the particles may still appear to be damp. Wipe large particles individually. Take care to avoid excess evaporation during the operation of surface drying.

(5) Obtain the weight in grams of the saturated surface-dry specimen. The specimen in this and subsequent weighings should be weighed to the nearest 1.0 g.

(6) Immediately after weighing, place the specimen in the wire basket and determine the weight of the specimen in water. Determine and record the temperature of the water in which the specimen is immersed.

(7) Oven-dry the specimen to a constant weight at 110º ± 5º C. After cooling to room temperature, weigh the specimen.

(8) Record all weights on the data sheet.

3.4.3.2.4. Computation

The following quantities are obtained by direct weighing:

(1) Weight of tare plus oven dried soil in grams. The tare is subtracted from this value to obtain the weight of dry soil, A.

(2) Weight of tare plus saturated surface-dry soil in grams. The tare weight is subtracted from this value to obtain the weight of saturated surface-dry soil, B.

(3) Weight of wire basket plus saturated soil in water in grams. The weight of wire basket in water is subtracted from this value to obtain the weight of saturated soil in water, C.

The apparent specific gravity is computed to two decimal places by the formula:

Equation 3-21: $$G_a = \frac{AK}{A - C}$$

where K = correction factor based on the density of water at 20º C (see Table 3-11).

The bulk specific gravity is computed to two decimal places by the formula:

Equation 3-22: $$G_m = \frac{AK}{B - C}$$

When a soil is composed of particles both larger and smaller than the No. 4 sieve, the specific gravity of the soil for use in engineering calculations shall be computed as follows:

Equation 3-23: $$G = \frac{100}{\dfrac{\%\text{ passing No 4. Sieve}}{G_s} + \dfrac{\%\text{ retained On No. 4 Sieve}}{G_a}}$$

3.4.3.3. Possible Errors

Following are possible errors that would cause inaccurate determinations of specific gravity:

3.4.3.3.1. Specific Gravity of Solids

(1) Imprecise weighing of flask and contents. Since the computation of the specific gravity of solids is based on a difference in weights that is small in comparison with the weights themselves, the same balance should be used for calibrating the volumetric flask and for determining the specific gravity whenever the calibration curve is used.

(2) Temperature of flask and contents not uniform. Both in calibrating the flask and determining the specific gravity, utmost care should be taken to insure that measured temperatures are representative of the flask and contents during the times when the weighings are made.

(3) Flask not clean. The calibration curve will not remain valid if dirt accumulation changes the weight of the flask. In addition, if the inside of the neck is not clean, an irregular meniscus may form.

(4) Moisture on outside of flask or inside of neck. When calibrating the flask for a temperature lower than room temperature, there is a tendency for condensation to form on the flask despite careful drying and rapid weighing. Whenever possible, weighing should be done at approximately the same temperature as that of the flask.

(5) Meniscus not coincident with mark on neck of flask. One drop of water too much makes an error of approximately 0.05 g. Taking the average of several readings at the same temperature can minimize this error. When the suspension is opaque, a strong light behind the neck is helpful in seeing the bottom of the meniscus.

(6) Use of water containing dissolved solids. It is essential that distilled or demineralized water be used exclusively to insure the continued validity of the flask calibration curve.

(7) Incomplete removal of entrapped air from soil suspension. This is the most serious source of error in the specific gravity determination and will tend to lower the computed specific gravity. The suspension must be thoroughly evacuated or boiled and the absence of verified entrapped air[60].

[60] It should be noted that air dissolved in the water will not affect the results, so it is not necessary to apply vacuum to the flask when calibrating or after filling the flask to the calibration mark.

(8) Gain in moisture of oven-dried specimen before weighing. If the specimen is oven dried before the specific gravity determination, it must be protected against a gain in moisture until it can be weighed and placed in the flask.

(9) Loss of material from oven dried specimen. If the specimen is oven dried and weighed before being placed in the flask, any loss of material will lower the computed specific gravity.

3.4.3.3.2. Apparent and Bulk Specific Gravity

(1) Loss of moisture from saturated surface-dry particles before weighing. Unless the saturated surface-dry material is weighed promptly, evaporation may cause an increase in the computed bulk specific gravity.

(2) Failure to correct for the change in density of water with temperature. This correction is often overlooked when computing either the apparent or the bulk specific gravity.

3.5. Atterberg Limits: Liquid and Plastic Limits

3.5.1. Overview

The Atterberg limits are water contents that define the limits of various stages of consistency for fine-grained soils.

Plasticity of soils is an important concept. During soil identification, a judgment is made that the soil is plastic, or non-plastic but no relative value is assigned. Arbitrary indices have been chosen to define the plasticity of cohesive (clay) soils. These are liquid limit (LL), plastic limit (PL), and plasticity index (PI).

(1) Liquid Limit (LL). The liquid limit is the upper limit of the plastic range of a soil. In the laboratory, the liquid limit of a soil is the water content, expressed as a percentage of the weight of oven dried soil at which two halves of a soil pat separated by a groove of standard dimensions will close at the bottom of the groove along a distance of _" under the impact of 25 blows in a standard liquid limit device.

(2) Plastic Limit (PL). The plastic limit is the lower limit of the plastic range of the soil. In the laboratory, the plastic limit of a soil is the water content, expressed as a percentage of the weight of oven dried soil at which the soil just begins to crumble into short pieces when rolled into a thread 1/8" in diameter.

(3) Plasticity Index (PI). The plasticity index (PI) is the numerical difference between these two limits as shown in Equation 3-24. The plasticity index represents the range of water content in which the soil remains plastic. In general, the plasticity index represents and relative amount of clay particles in the soil. The higher the PI, the greater the amount of clay particles present, and the more plastic the soil. The more plastic a soil, it will:

- Be more compressible.
- Have higher shrink-swell potential.
- Be less permeable.

In addition to their use in soil classification, Atterberg Limits also are indicators of structural properties, as shown in the correlations in this section. The test procedures for Atterberg tests are given below. Atterberg limits are a cheap method of obtaining much useful data. The following are the more important uses of Atterberg limits:

- Help identify and classify the soil.

- PI (plasticity index) is an indicator of soil compressibility and potential for volume change. Estimate compression index (C_c) for normally consolidated and low sensitivity clay using Table 7-12.
- PL (plastic limit) can indicate if clay has been preconsolidated. Most soils are deposited at or near their liquid limit. If the *in situ* natural water content (w) is near the plastic limit (PL), then the soil is probably preconsolidated. Some stress has been applied in the past to squeeze that water out.
- Clay may also be assumed to be preconsolidated if the liquidity index (LI), which is the (moisture content minus plastic limit) divided by plastic index is less than 0.7. Other correlations for this can be found in 5.1.3.5.

Atterberg Limit tests should be performed discriminately, and should be reserved for representative samples selected after evaluating subsoil pattern. Determine Atterberg Limits of each consolidation test sample and each set of samples grouped for triaxial shear tests. For selected borings, determine Atterberg Limits on samples at regular vertical intervals for a profile of limits and corresponding natural water content.

Detailed procedures for determining the liquid and plastic limits for use in classifying soils and developing correlations with engineering properties of soils are given below.

3.5.2. Apparatus

3.5.2.1. Liquid Limit Device

A mechanical device consisting of a brass cup suspended from a carriage designed to control its drop onto a hard rubber base. A drawing showing the essential features of the device and the critical dimensions is given in Figure 3-15. The design of the device may vary if the essential functions are preserved. The device may be operated either by a hand crank or by an electric motor.

Figure 3-15 Hand Operated Liquid Limit Device

(1) Base. The base shall be hard rubber[61] having a D Durometer hardness of 80 to 90, and resilience such that a 8 mm (5/16”) diameter polished steel ball, when dropped from a height of 250 mm (9.84”) will have an average rebound of at least 80% but no more than 90%. The tests shall be conducted on the finished base with feet attached.

(2) Feet. The base shall be supported by rubber feet designed to provide isolation of the base from the work surface and having an A Durometer hardness no greater than 60 as measured on the finished feet attached to the base.

(3) Cup. The cup shall be brass and have a weight, including cup hanger, of 185 to 215 g.

(4) Cam. The cam shall raise the cup smoothly and continuously to its maximum height, over a distance of at least 180º of cam rotation. The preferred cam motion is a uniformly accelerated lift curve.[62] The design of the cam and follower combination shall be such that there is no upward or downward velocity of the cup when the cam follower leaves the cam.

(5) Carriage. The cup carriage shall be constructed in a way that allows convenient but secure adjustment of the height of drop of the cup to 10 mm (0.394”). The cup hanger shall be attached to the carriage by means of a pin that allows removal of the cup and cup hanger for cleaning and inspection.

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[61] Micarta No. 22lA has been used in the past. It is satisfactory as long as it meets the resilience requirement set forth for hard rubber

[62] The cam and follower design in Figure 3-15 is for uniformly accelerated (parabolic) motion after contact and assures that the cup has no velocity at drop off. Other cam designs also provide this feature and may be used; however, if the cam follower lift pattern is not known, zero velocity at drop off can be assured by carefully filing or machining the cam and follower so that the cup height remains constant over the last 20º to 45º of cam rotation.

(6) Optional Motor Drive. As an alternative to the hand crank, the device may be equipped with a motor to turn the cam. Such a motor must turn the cam at 2 ± 0.1 revolutions per second, and must be isolated from the rest of the device by rubber mounts or in some other way that prevents vibration from the motor being transmitted to the rest of the apparatus. It must be equipped with an ON-OFF switch and means of conveniently positioning the cam for height of drop adjustments. The results obtained using a motor-driven device must not differ from those obtained using a manually operated device.

3.5.2.2. Grooving Tool

A grooving tool having dimensions as shown in Figure 3-16. The tool shall be made of plastic or noncorroding metal.[63] The design of the tool may vary as long as the essential dimensions are maintained. The tool may, but need not, incorporate the gage for adjusting the height of drop of the liquid limit device.

[63] Polycarbonate plastic grooving tools meeting the dimensional requirements given above are available to US Government agencies through the US Army Engineer Division Laboratory, Southwestern, 4815 Cass Street, Dallas, TX 75235.

Figure 3-16 Grooving Tool (optional height-of-drop gage attached)

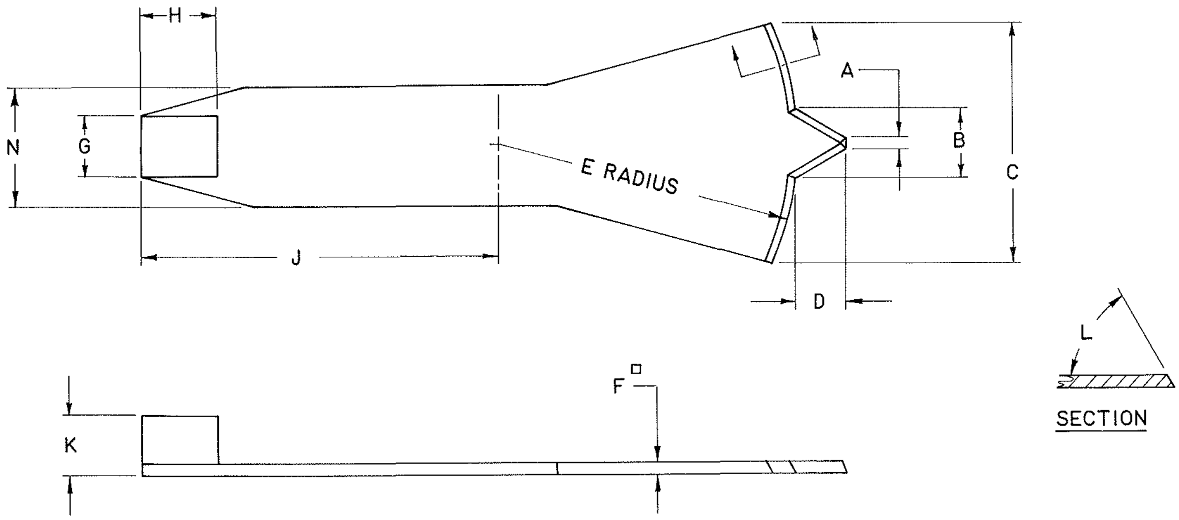
DIMENSIONS

LETTER	A △	B △	C △	D △	E △	F △
MM	2 ± 0.1	11 ± 0.2	40 ± 0.5	8 ± 0.1	50 ± 0.5	2 ± 0.1
LETTER	G	H	J	K △	L △	N
MM	10 MINIMUM	13	60	10 ± 0.05	60 DEG ± 1 DEG	20

△ ESSENTIAL DIMENSIONS

□ BACK AT LEAST 15 MM FROM TIP

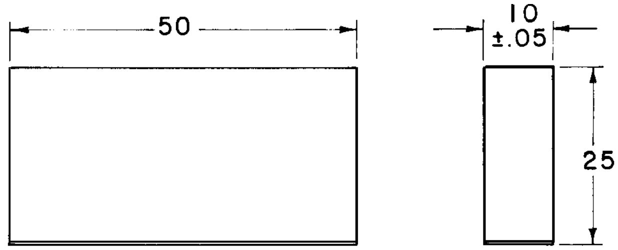
NOTE: DIMENSION A SHOULD BE 1.9-2.0 AND DIMENSION D SHOULD BE 8.0-8.1 WHEN NEW TO ALLOW FOR ADEQUATE SERVICE LIFE



3.5.2.3. Gage

A metal gage block for adjusting the height of drop of the cup should have dimensions as shown in Figure 3-17. The design of the tool may vary provided the gage will rest securely on the base without being susceptible to rocking, and the edge which contacts the cup during adjustment is straight, at least 10 mm (3/8") wide, and without bevel or radius.

Figure 3-17 Height of Drop Gage[64]



3.5.2.4. Container

Small corrosion-resistant containers with snug-fitting lids for water content specimens. Aluminum or stainless steel cans 2.5 cm (1") high by 5 cm (2") in diameter are appropriate.

3.5.2.5. Balance

A balance readable to at least 0.01 g and having an accuracy of 0.03 g within three standard deviations within the range of use. Within any 15 g range, a difference between readings shall be accurate within 0.01 g.

3.5.2.6. Storage Container

The container for storage of the prepared soil specimen should not contaminate the specimen in any way, and also prevent moisture loss. A porcelain, glass, or plastic dish about 114 mm (4 1/2") in diameter and a plastic bag large enough to enclose the dish and be folded over is adequate.

3.5.2.7. Ground Glass Plate

A ground glass plate at least 30 cm (12") square by 1 cm (3/8") thick for mixing soil and rolling plastic limit threads.

3.5.2.8. Spatula

A spatula or pill knife should have a blade about 2 cm (3/4") wide by about 10 cm (4") long. In addition, a spatula having a blade about 2.5 cm (1") wide and 15 cm (6") long has been found useful for initial mixing of samples.

3.5.2.9. Sieve

A 203 mm (8") diameter, 425 μm (No. 40) sieve conforming to the requirements of ASTM Specification E11 and having a rim at least 5 cm (2") above the mesh. A 2 mm (No. 10) sieve meeting the same requirements may also be needed.

3.5.2.10. Wash Bottle

A wash bottle or similar container for adding controlled amounts of water to soil and washing fines from coarse particles.

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[64] Dimensions in millimeters.

3.5.2.11. Drying Oven

A thermostatically controlled oven, preferably of the forced-draft type, should be capable of continuously maintaining a temperature of 110 ± 5º C throughout the drying chamber. The oven shall be equipped with a thermometer of suitable range and accuracy for monitoring oven temperature.

3.5.2.12. Washing Pan

A round, flat-bottomed pan should be at least 76 mm (3") deep, slightly larger at the bottom than a 203 mm (8") diameter sieve.

3.5.2.13. Rod (Optional)

A metal or plastic rod or tube which is 3.2 mm (1/8") diameter and about 100 mm (4") long for judging the size of plastic limit threads.

3.5.2.14. Mixing Water

A supply of distilled or demineralized water.

3.5.2.15. Blender (Optional)

A single speed blender should be equipped with a 1 liter container for preparing clay shale materials.

3.5.3. Calibration of Apparatus

3.5.3.1. Inspection for Wear

(1) Liquid Limit Device. Determine that the liquid limit device is both clean and in good working order. The following specific points should be checked:

   a. Wear of Base. The spot on the base where the cup makes contact should be worn no greater than 10 mm (3/8") in diameter. If the wear spot is greater than this, the base can be machined to remove the wear spot provided the resurfacing does not decrease base thickness to less than that specified in 2(a) and the other dimensional relationships are maintained.

   b. Wear of Cup. The cup must be replaced when the grooving tool has work a depression in the cup 0.1 mm (0.004") deep or when the edge of the cup has been reduced to half its original thickness. Verify that the cup is firmly attached to the cup hanger.

   c. Wear of Cup Hanger. Verify that the cup hanger pivot does not bind and is not worn to an extent that allows more than 3 mm (1/8") side-to-side movement of the lowest point on the rim.

   d. Wear of Cam. The cam shall not be worn to an extent that the cup drops before the cup hanger (cam follower) loses contact with cam.

(2) Grooving Tool. Inspect grooving tools for wear on a frequent and regular basis. The rapidity of wear depends on the material from which the tool is made and the types of soils being tested. Sandy soils cause rapid wear of grooving tools; therefore, when testing these materials, tools should be inspected more frequently than for other soils. Any tool with a tip width greater than 2.1 mm must not be used. The depth of the tip of the grooving tool must be 7.9 to 8.1 mm. The width of the tip of grooving tools is conveniently checked using a pocket sized measuring magnifier equipped with a millimeter scale. Magnifiers of this type are available for most laboratory supply companies. The depth of the tip of grooving tools can be checked using the depth-measuring feature of vernier calipers.

(3) Blender Blades. Blender blades should be replaced when their overall length becomes 3 mm (1/8") less than their original length.

3.5.3.2. Adjustment of Height of Drop

Adjust the height of drop of the cup so that the point of the cup that comes in contact with the base rises to a height of 10 ± 0.2 mm. A convenient procedure for adjusting the height of drop is as follows:

(1) Place a piece of masking tape across the outside bottom of the cup parallel with the axis of the cup hanger pivot. The edge of the tape away from the cup hanger should bisect the spot on the cup that contacts the base. For new cups, placing a piece of carbon paper on the base and allowing the cup to drop several times will mark the contact spot.

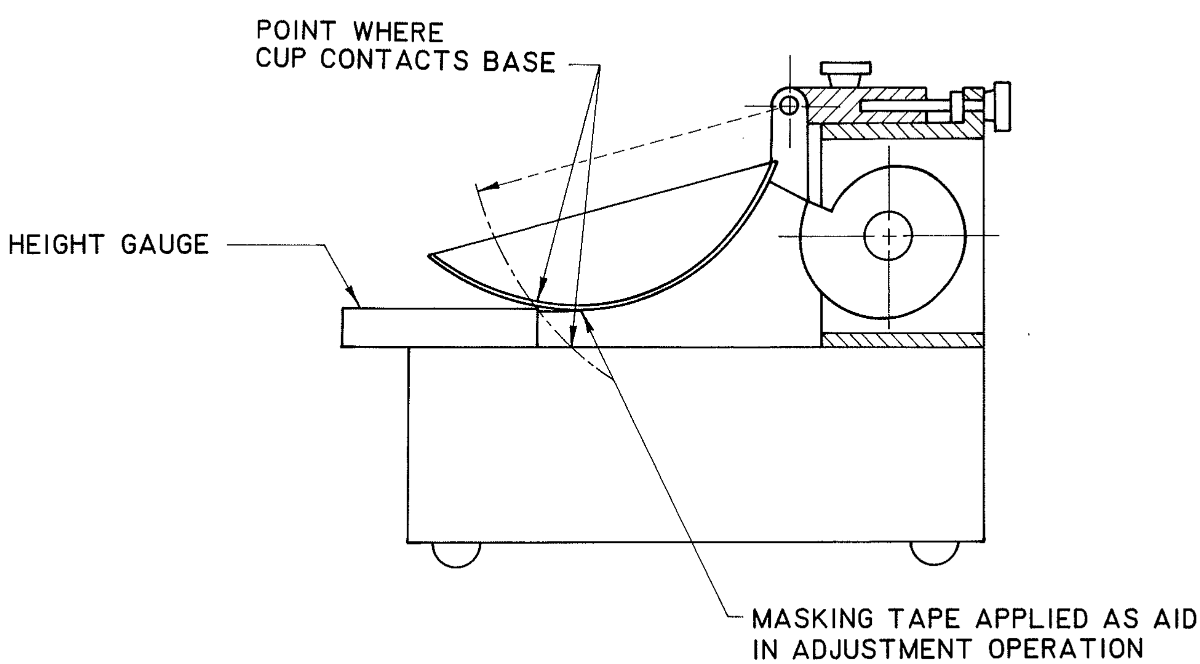
(2) Attach the cup to the device and turn the crank until the cup is raised to its maximum height.

(3) Slide the height gage under the cup from the front, and observe whether the gage contacts the cup or the tape (see Figure 3-18). If the tape and cup are both contacted, the height of drop is approximately correct. If not, adjust the cup until simultaneous contact is made.

(4) Check adjustment by turning the crank at two revolutions per second while holding the gage in position against the tape and cup. If a ringing or clicking sound is heard without the cup rising from the gage, the adjustment is correct. If no ringing is heard or if the cup rises from the gage, readjust the height of drop. If the cup rocks on the gage during this checking operation, the cam follower pivot is excessively worn and the worn parts should be replaced.

(5) Always remove tape after completion of adjustment operation.

Figure 3-18 Calibration for height of drop



3.5.4. Preparation of Material

3.5.4.1. Selection of Material

It is essential that the same carefully prepared soil mixture be used for determining both the liquid and plastic limit. Layers of soil of different plasticity should not be mixed. Furthermore, if the natural water content is to be determined, the specimen must be taken from an identical mixture to permit valid correlations. If other test results are to be correlated with the liquid and plastic limits, the material used for the determinations must be the same as that tested. Clay shale materials require special preparation as discussed in 3.5.9.

3.5.4.2. Effects of Drying

Whenever possible, soils should be at the natural water content when preparation for testing is begun. If drying has occurred before testing, the limit values may change. The plasticity of soils containing organic colloids and certain types of inorganic colloids derived from volcanic rocks is highly sensitive to drying. The effects of drying can be determined by comparing the liquid limit values of specimens in “undried," "air dried," and "oven dried" states.

3.5.4.3. General Preparation of Material

(1) Samples Passing the 425 μm (No. 40) Sieve. When by visual and manual procedures, it is determined that the sample has little or no material retained on the 425 μm (No. 40) sieve, prepare a specimen of 150 to 200 g by mixing thoroughly with distilled or demineralized water on the glass plate using the spatula. If desired, soak soil in a storage dish with small amount of water to soften the soil before the start of mixing. Adjust the water content of the soil to bring it to a consistency that would require 15 to 25 blows of the liquid limit device to close the groove. The time taken to adequately mix a soil will vary greatly depending on the plasticity and initial water content. Initial mixing times of more than 30 minutes may be needed for stiff, fat clays. If, during mixing, a small percentage of material is encountered that would be retained on a 425 μm (No. 40) sieve, remove these particles by hand, if possible. If it is impractical to remove the coarser material by hand, remove small percentages (less than about 15%) of coarser material by working the specimen through a 425 μm (No. 40) sieve using a piece of rubber sheeting, a rubber stopper, or other convenient device provided the operation does not distort the sieve or degrade material that would be retained if the washing method described in the next paragraph were used. If larger percentages of coarse material are encountered during mixing, or it is considered impractical to remove the coarser material by the methods just described, wash the sample as described in the next paragraph. When the coarse particles found during mixing are concretions, shells, or other fragile particles, do not crush these particles to make them pass a 425 μm (No. 40) sieve, but remove by hand or by washing. Place the mixed soil in the storage dish, cover to prevent loss of moisture, and allow to stand for at least 16 hours (overnight). After the standing period and immediately before starting the test, thoroughly remix the soil.

(2) Samples Containing Material Retained on a 425 μm (No. 40) Sieve.

  a) Select a sufficient quantity of soil at natural water content to provide 150 to 200 g of material passing the 4250urn (No. 40) sieve. Place in a pan or dish and add sufficient distilled or demineralized water to cover the soil. Allow soaking until all lumps have softened and the fines no longer adhere to the surfaces of the coarse particles.

  b) When the sample contains a large percentage of material retained on the 425 μm (No. 40) sieve, perform the following washing operations in increments, washing no more than 0.5 kg (1 lb) of material at one time.

i. Place the 425 μm (No. 40) sieve in the bottom of the clean pan.

ii. Pour the soil water mixture onto the sieve. If gravel or coarse sand particles are present, rinse as many of these as possible with small quantities of water from a wash bottle and discard. Alternatively, pour the soil water mixture over a 2-mm (No. 10) sieve nested atop the 425 μm (No. 40) sieve, rinse the fine material through and remove, the 2-mm (No. 10) sieve.

iii. After washing and removing as much of the coarser material as possible, add sufficient water to the pan to bring the level to about 13 mm (1/2”) above the surface of the 425 μm (No. 40) sieve.

iv. Agitate the slurry by stirring with the fingers while raising and lowering the sieve in the pan and swirling the suspension so that fine material is washed from the coarser particles.

v. Disaggregate fine soil lumps that have not slaked by gently rubbing them over the sieve with the fingertips.

vi. Complete the washing operation by raising the sieve above the water surface and rinsing the material retained with a small amount of clean water.

vii. Discard material retained on the 425 μm (No. 40) sieve.

c) Reduce the water content of the material passing the 425 μm (No. 40) sieve until it approaches the liquid limit. Reduction of water content may be accomplished by one or a combination of the following methods:

i. Exposing the air currents at ordinary room temperature,

ii. Exposing to warm air currents from a source such as an electric hair dryer,

iii. Filtering in a Buckner funnel or using filter candles,

iv. Decanting clear water from surface of suspension, or

v. Draining in a colander or plaster of Paris dish lined with high retentivity, high wet-strength filter paper.[65]

If a plaster of Paris dish is used, take care that the dish never becomes sufficiently saturated that it fails to actively absorb water into its surface. Thoroughly dry dishes between uses. During evaporation and cooling, stir the sample often enough to prevent over drying of the fringes and soil pennacles on the surface of the mixture. For soil samples containing soluble salts, use a method of water reduction such as (i) or (ii) that will not eliminate the soluble salts from the test specimen.

d) Thoroughly mix the material passing the 425 μm (No. 40) sieve on the glass plate using the spatula. Adjust the water content of the mixture, if necessary, by adding small increments of distilled or demineralized water or by allowing the mixture to dry at room temperature while mixing on the glass plate. The soil should be at a water content that will result in closure of the groove in 15 to 25 blows. Return the mixed soil to the mixing dish, cover to prevent loss of moisture, and allow standing for at least 16 hours. After the standing period, and immediately before starting the test, remix the soil thoroughly.

[65] S&S 595 filter paper in 32 cm circles has been found satisfactory.

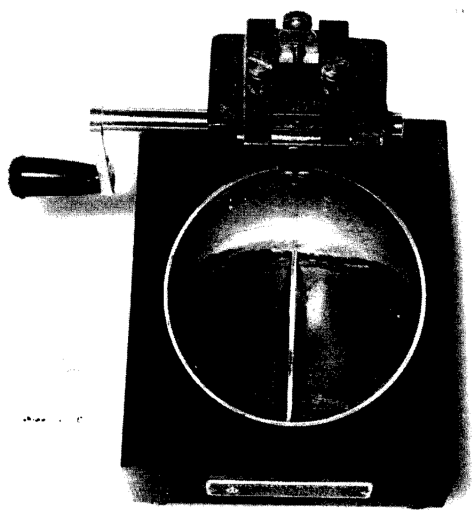
3.5.5. Liquid Limit

3.5.5.1. Procedure

(1) Place a portion of the prepared soil in the cup of the liquid limit device at the point where the cup rests on the base, squeeze it down, and spread it into the cup to a depth of about 10 mm at its deepest point, tapering to form an approximately horizontal surface. Take care to eliminate air bubbles from the soil pat, but form the pat with as few strokes as possible. Heap the unused soil on the glass plate and cover with the inverted storage dish or a wet towel.

(2) Form a groove in the soil pat by drawing the tool, bevelled edge, forward through the soil on a line joining the highest point to the lowest point on the rim of the cup. When cutting the groove, hold the grooving tool against the surface of the cup and draw in an arc maintaining the tool perpendicular to the surface of the cup throughout its movement (see Figure 3-19). In soils where a groove cannot be made in one stroke without tearing the soil cut the groove with several strokes of the grooving tool. Alternately, cut the groove to slightly less than required dimensions with a spatula and use the grooving tool to bring the groove to final dimensions. Exercise extreme care to prevent sliding the soil pat relative to the surface or the cup.

Figure 3-19 Grooved Soil Pat in Liquid Limit Device



(3) Verify that no crumbs of soil are present on the base or the underside of the cup. Lift and drop the cup by turning the crank at a rate of 1.9 to 2.1 drops per second (see Figure 3-20) until the two halves of the soil pat come in contact at the bottom of the groove along a distance of 13 mm (1/2”) (see Figure 3-20). Use the end of the grooving tool (Figure 3-16) or a scale to verify that the groove has closed 13 mm (1/2”.)

Figure 3-20 Cranking Liquid Limit Device

Figure 3-21 Soil pat after groove has closed

(4) Verify that an air bubble has not caused premature closing of the groove by observing that both sides of the groove have flowed together with approximately the same shape. If a bubble has caused premature closing of the groove, reform the soil in the cup by adding a small, amount of soil to make up for that lost in the grooving operation and repeat steps (l) through (3). If the soil slides on the surface of the cup, repeat steps (l) through (3) at higher water content. If, after several trials at successively higher water contents, the soil pat continues to slide in the cup or if the number of blows required to close the groove is always less than 25, record that the liquid limit could not be determined, and report the soil as nonplastic without performing the plastic limit test.

(5) Record the number of drops (N) required to close the groove. Remove a slice of soil approximately the width of the spatula extending from edge to edge of the soil cake at right angles to the groove and including that portion of the groove in which the soil flowed together, place in a weighed container, and cover.

(6) Return the soil remaining in the cup to the glass plate. Wash and dry the cup and grooving tool and reattach the cup to the carriage in preparation for the next trial.

(7) Remix the entire soil specimen on the glass plate to reduce the water content of the soil and increase the number of blows required to close the groove. Repeat (1) through (6) for at least three additional trials producing successively greater numbers of blows to close the groove. Preferably, two trials should produce closure in 25 blows or less, and two trials should produce closure in 25 blows or more.

(8) Determine the water content (W_N) of the soil specimen from each trial. Make all weighings on the same balance. Initial weighings should be performed immediately after completion of the test. If the test is to be interrupted for more than about 15 minutes, the specimens already obtained should be weighed at the time of the interruption.

3.5.5.2. Calculations

(1) Plot the relationship between the water content and the corresponding number of drops, N, of the cup on a semilogarithmic graph with the water content as ordinates on the arithmetical scale, and the number of drops as abscissas on the logarithmic scale. See Figure 11-5 for an example data form. Draw the best straight line through the four or more plotted points.

(2) Take the water content corresponding to the intersection of the line with the 25-drop abscissa as the liquid limit of the soil. Computational methods may be substituted for the graphical method for fitting a straight line to the data and determining the liquid limit.

3.5.6. Plastic Limit

3.5.6.1. Preparation of Test Specimen

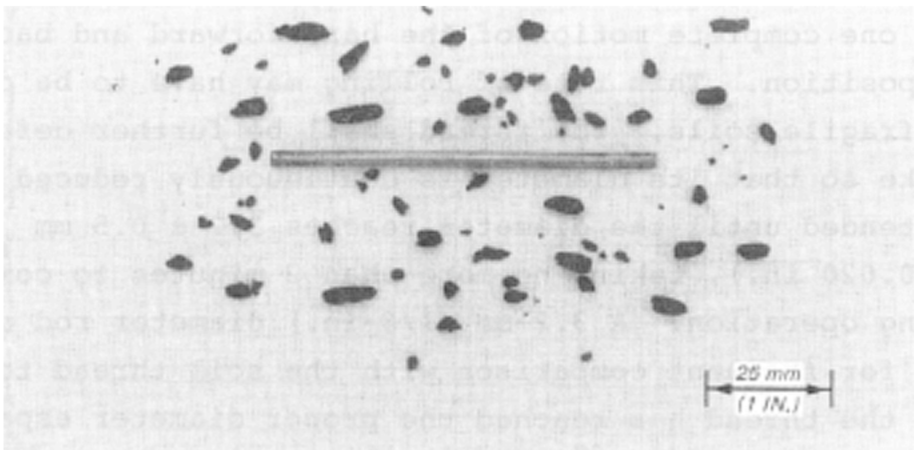
Either after the second mixing before the test or from the soil remaining after completion of the test, select a 20 g portion of soil from the material prepared for the liquid limit test. Reduce the water content of the soil to a consistency at which it can be rolled without sticking to the hands by spreading and mixing continuously on the glass plate. The drying process may be accelerated by exposing the soil to the air current from an electric fan or by blotting with paper that does not add any fiber to the soil such as hard surface paper toweling or high wet strength filter paper.

3.5.6.2. Procedure

(1) From the 20 g mass, select a portion of 1.5 to 2.0 g. Form the test specimen into an ellipsoidal mass. Roll this mass between the palm or fingers and the ground-glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length. A normal rate of rolling for most soils should be 80 to 90 strokes/minute counting a stroke as one complete motion of the hand forward and back to the starting position. This rate of rolling may have to be decreased for very fragile soils. The thread shall be further deformed on each stroke so that its diameter is continuously reduced; and its length extended until the diameter reaches 3.2 ± 0.5 mm (0.125" ± 0.020"), taking no more than 2 minutes to complete the rolling operation. A 3.2 mm (1/8") diameter rod or tube is useful for frequent comparison with the soil thread to ascertain when the thread has reached the proper diameter especially for inexperienced operators. The amount of hand or finger pressure required will vary greatly according to the soil. Fragile soils of low plasticity are best rolled under the outer edge of the palm or at the base of the thumb. When the diameter of the thread becomes 3.2 mm, break the thread into several pieces. Squeeze the pieces together, knead between the thumb and first finger of each hand, reform into an elliposidal mass, and reroll. Continue this alternate rolling to a thread 3.2 mm in diameter, gathering together, kneading and rerolling, until the thread crumbles under the pressure required for rolling, and the soil can no longer be rolled into a 3.2 mm diameter thread (see Figure 3-22.) It has no significance if the thread breaks into threads of shorter length. Roll each of these shorter threads to 3.2 mm in diameter. The only requirement for continuing the test is that they can be reformed into an ellipsoidal mass and rolled out again. The operator shall at no time attempt to produce failure at exactly 3.2 mm diameter by allowing the thread to reach 3.2 mm, then reducing the rate of rolling or the hand pressure or both, while continuing the rolling without further deformation until the thread falls apart. It is permissible, however, to reduce the total amount of deformation for feebly plastic soils by making the initial diameter of the elliposidal mass nearer to the required 3.2 mm final diameter. If crumbling occurs when the thread has a diameter

greater than 3.2 mm, this shall be considered a satisfactory end point provided the soil has been previously rolled into a thread 3.2 mm in diameter. Crumbling of the thread will manifest itself differently with the various types of soil. Some soils fall apart in numerous small aggregations of particles; others may form an outside tubular layer that starts splitting at both ends. The splitting progresses toward the middle, and finally, the thread falls apart in many small platy particles. Fat clay soils require much pressure to deform the thread, particularly as they approach the plastic limit. With these soils, the thread breaks into a series of barrel-shaped segments about 3.2 to 9.5 mm (1/8 to 3/8") in length.[66]

Figure 3-22 Lean clay soil at the plastic limit



(2) Gather the portions of the crumbled thread together and place in a weighed container. Immediately cover the container.

(3) Select another 1.5 to 2.0 g portion of soil from the original 20 g specimen and repeat the operations described in (1) and (2) until the container has at least 9 g of soil.

(4) Repeat (1) through (3) to make another container holding at least 9 g of soil. Determine the water content, in percent, of the soil contained in the containers. Make all weighings on the same balance.

3.5.6.3. Calculation

Compute the average of the two water contents. If the difference between the two water contents is greater than two percentage points, repeat the test. The plastic limit is the average of the two water contents.

3.5.7. Plasticity Index

3.5.7.1. Calculation

Calculate the plasticity index as follows:

Equation 3-24: $PI = LL - PL$

Where

[66] A. Casagrande, R. C. Hirschfield, and S. J. Poulos, *Third Progress Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays*, Soil Mechanics Series No. 70, Harvard University (Cambridge, Mass., November 1963).

- LL = the liquid limit
- PL = the plastic limit

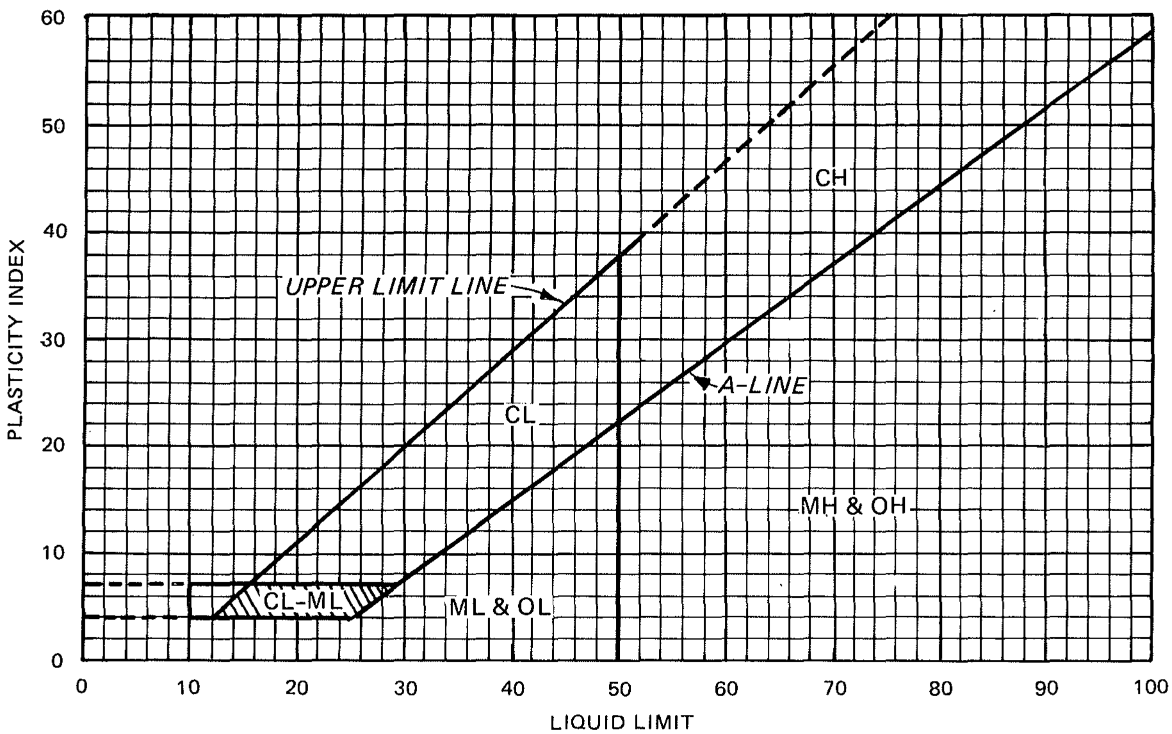
Both LL and PL are whole numbers.

If either the liquid limit or plastic limit could not be determined, or if the plastic limit is equal to or greater than the liquid limit, report the soil as nonplastic.

3.5.7.2. Plasticity chart

Errors in computing the liquid or plastic limits sometimes can be detected by plotting the values of liquid limit versus plasticity index on the plasticity chart[67] as shown in Figure 3-23. The upper limit line starts from a liquid limit of 8 at a plasticity index of 0 and rises toward the right with a slope of 9 vertically on 10 horizontally; the equation of the upper limit line, therefore, is PI = 0.9 (LL - 8). A plot of liquid limit versus plasticity index for natural soils has never been known to fall above the upper limit line. Figure 11-6 is a suggested form for the graphical correlation of the various Atterberg limits data within a project or testing assignment.

Figure 3-23 Plasticity chart showing classification group symbols



3.5.8. Report

Report the following information:

(1) Sample identifying information.

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[67] US Army Engineer Waterways Experiment Station, *The Unified Soil Classification System*, Technical Memorandum No. 3-357, Vol. 1 (Vicksburg, Miss., March 1953, revised April 1960). An abridged version of the material in this report is presented in Military Standard MIL-STD-619A, 20 March 1962.

(2) Any special specimen selection process used such as removal of sand lenses from undisturbed sample.

(3) Liquid limit, plastic limit, and plasticity index to the nearest whole number and omitting the percent designation. If the liquid limit or plastic limit is equal to or greater than the liquid limit, report the soil as nonplastic, NP.

(4) An estimate of the percentage of sample retained on the 425-_m (No. 40) sieve, and

(5) Procedure by which liquid limit was performed, if it differs from the multipoint method.

3.5.9. Preparing Clay Shale Material for Testing

3.5.9.1. General

Investigations have shown that air-drying and slaking, oven drying and slaking, the type and duration of mechanical dispersing, and other variations in procedure affect classification indexes of clay shale materials. While the methods for preparing clay shale material for testing should cover a sufficient range of disaggregation efforts to assess the strength of interparticle bonds, the number of variables allowed to influence the indexes must be minimized by standardized procedures to prevent the classification of each material becoming a minor research project in itself.

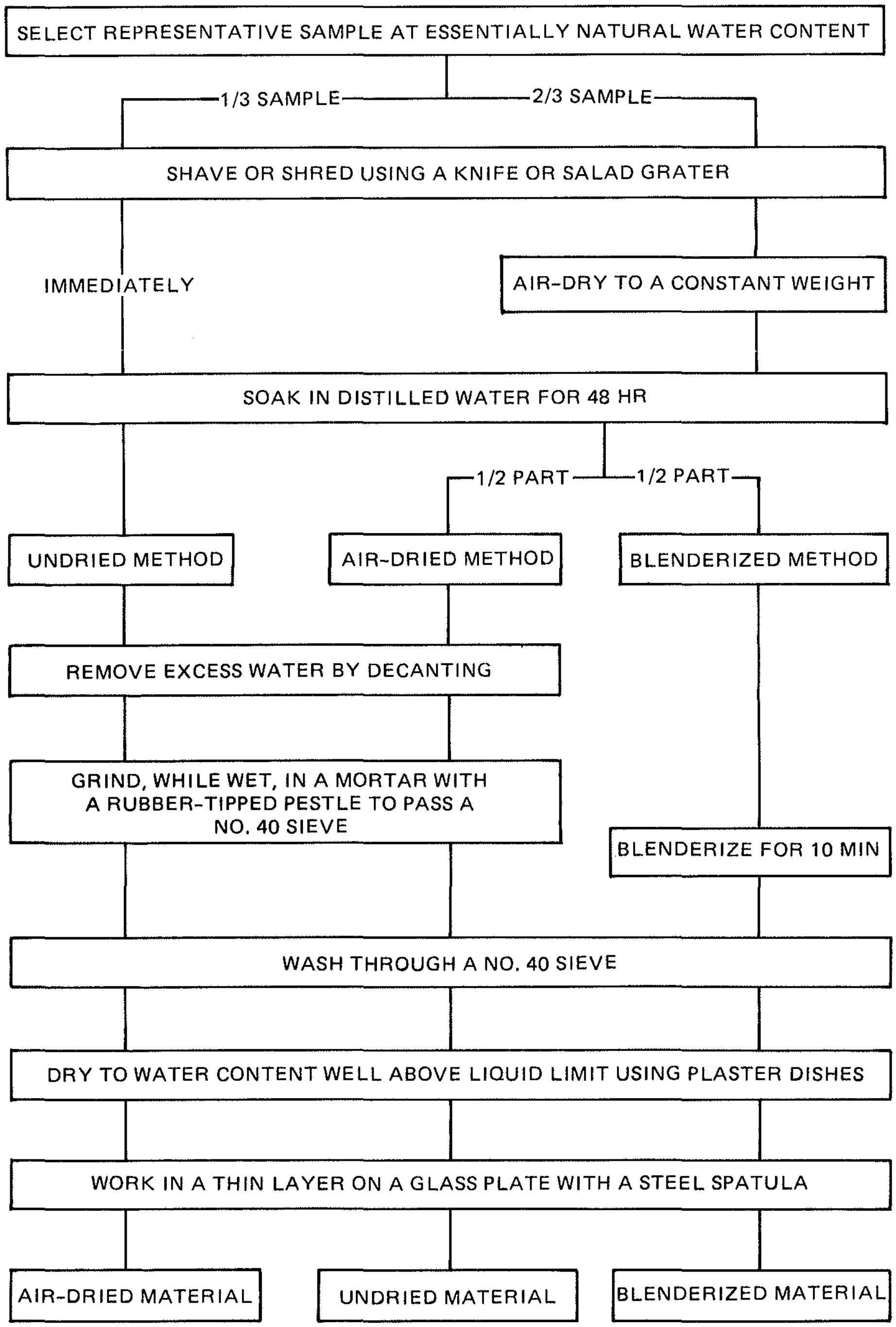
Therefore, three standard methods of processing clay shale material will be used. These will be referred to as the blenderized, undried, and air-dried methods. The primary method is to test material that has been essentially completely disaggregated by high-speed blenderizing; this method will provide a reference value and it should be used for all clay shale samples on which Atterberg limits are to be determined. To provide additional indexes as desired, material that has not been subjected to any drying and material that has been subjected to a single cycle of air-drying and soaking may be tested.

These two methods should be used on sufficient representative samples to cover the range of samples identified by the primary method.

3.5.9.2. Standard Method

When material is to be prepared by all three processing procedures, exercise care that the parent material for the batches is similar. Divide the piece of sample selected by a vertical cut into two parts with one piece about twice as large as the other piece. Shave the smaller piece into distilled water to produce the undried batch, and use the larger piece to produce the other two batches. Figure 3-24 shows a flow diagram of the three preparation methods and indicates when separation of batches is required. Material may be taken from each of the three batches and used for Atterberg limits determinations without further processing. Details of each procedure are as follows.

Figure 3-24 Flow diagram showing the three standard methods of preparing clay shale materials



(1) Blenderized (primary method). Shave or shred material at essentially natural water content and dry to a constant weight in an atmosphere with a temperature less than 50º C and a relative humidity less than 30%. After a constant weight is attained (and after a drying period of at least 48 hours), soak the material in distilled water for at least 48 hours.

a) Place about 500 ml of the slurry in the 1 liter glass container (available from any laboratory supply company) of a single-speed blender. Make the initial water content of the slurry above 300% or more than twice the estimated liquid limit (blenderized) whichever is greater. Typically, the weight of dry soil in the blender at any one time should not exceed 150 g.

b) Blenderize the slurry without interruption for 10 minutes and then wash through a 425-μm (No. 40) sieve. Remove excess water using a plaster of Paris dish lined with filter paper. Work material at a water content above the liquid limit in a thin layer on a glass plate with a steel spatula until no further reduction in the size of lumps can be achieved.

(2) Undried. Shave or shred material at essentially natural water content, immediately place in distilled water, and soak for at least 48 hours After removing excess water by decanting, grind the wet material in a mortar with a rubber tipped pestle and wash through the 425-μm (No. 40) sieve. Remove excess water using a plaster-of-Paris dish lined with filter paper. Work material at a water content above the liquid limit in a thin layer on a glass plate with a steel spatula until no further reduction in the size of lumps can be achieved.

(3) Air-dried. Shave or shred material at essentially natural water content and dry to a constant weight in an atmosphere with a temperature less than 50°C (120°F) and a relative humidity less than 30%. After a constant weight is attained (and after a drying period at least 48 h& soak the material in distilled water for at least 48 hours. After removing excess water by decanting, grind the wet material in a mortar with a rubber-tipped pestle and wash through the 425-μm (No. 40) sieve. Remove excess water using a plaster of Paris dish lined with filter paper. Work material at a water content above the liquid limit in a thin layer on a glass plate with a steel spatula until no further reduction in the size of lumps can be achieved.

3.5.10. Possible Errors

Following are possible errors that would cause inaccurate determinations of liquid and plastic limits:

3.5.10.1. General

(1) Specimen not representative. As described in paragraph 4a.t the liquid and plastic limits must be determined using the same mixture of soil as that used for determinations of natural water content or for other tests. Care should be taken when using the trimmings from preparation of other test specimens that material is as close as possible.

(2) Specimen improperly prepared. The specimens must be thoroughly mixed and be permitted to cure for a sufficient period before testing. Erroneous results may be caused by the loss of colloidal material when removing particles coarser than the No. 40 sieve or by testing air-dried or oven dried soils.

(3) Inaccurate determination of water content. The possible errors described in 3.3 would greatly affect the computed liquid and plastic limits because of the small quantities of material available for the water content determinations.

(4) Computational mistakes.

3.5.10.2. Liquid Limit Test

(1) Improperly constructed or adjusted liquid limit device.

(2) Worn parts of liquid limit device, especially at point of contact between the cup and the base or worn tip of grooving tool.

(3) Soil at point of contact between the cup and the base. Removal of the cup for shaping and grooving the soil pat will also ensure that the bottom of the cup and the top of the base are clean. Any soil that

has dropped onto the base can be removed with one stroke of the back of the hand just before replacing the cup.

(4) Loss of moisture during test. Erratic and erroneous results may be causing by drying of some soil mixtures unless the test is performed in a humid room.

3.5.10.3. Plastic Limit Test

(1) Incorrect final thread diameter. A length of 1/8" diameter metal rod nearby will help in estimating this diameter accurately.

(2) Stopping the rolling process too soon. If there is any doubt as to whether the thread has crumbled sufficiently, it is better to roll the thread once more than to stop the process too soon.

3.6. Grain-Size Analysis

Grain-size analysis is a process in which the proportion of material of each grain size present in a given soil (grain-size distribution) is determined. The grain- size distribution of coarse-grained soils is determined directly by sieve analysis, while that of fine-grained soils is determined indirectly by hydrometer analysis. The grain-size distribution of mixed soils is determined by combined sieve and hydrometer analyses. In addition to their use in classification, grain-size analyses may be applied to seepage and drainage problems, filter and grout design, and evaluation of frost heave.

- Grain-Size Parameters. Coefficient of uniformity, $C_u = D_{60}/D_{10}$, and coefficient of curvature, $C_z = (D_{30})^2/(D_{10}D_{60})$[68], are computed from D_{60}, D_{30}, and D_{10}, which particle size diameter corresponding respectively to 60%, 30%, and 10% passing on the cumulative particle size distribution curves. C_u and C_z indicate the relative broadness or narrowness of gradation. D_{10} is an approximate measure of the size of the void spaces in coarse-grained soils.
- Testing Program. Gradations of a large number of samples usually are not required for identification. Samples should be grouped in principal strata by visual classification before performing grain-size analyses on specimens of major strata.

Detailed procedures for determining the grain-size distribution of soils by sieve, hydrometer, and combined analyses are given below.

3.6.1. Sieve Analysis

3.6.1.1. Description

A sieve analysis consists of passing a sample through a set of sieves and weighing the amount of material retained on each sieve, Sieves are constructed of wire screens with square openings of standard sizes. The sieve analysis is performed on material retained on a U. S. Standard No. 200 sieve. The sieve analysis, in itself, is applicable to soils containing small amounts of material passing the No. 200 sieve provided the grain-size distribution of that portion of the sample passing the No. 200 sieve is not of interest.

3.6.1.2. Apparatus

The apparatus should consist of the following:

[68] Coduto, D.P. *Foundation Design: Principles and Practices*. Upper Saddle River, NJ: Prentice Hall, Inc., 2001, p. 122.

(1) A series of U. S. standard sieves with openings ranging from 3" to 0.074 mm (No. 200), including a cover plate and bottom pan, conforming to ASTM Designation: E11, Standard Specifications for Sieves for Testing Purposes. U. S. Standard Sieve Sizes or numbers and sieve openings in inches and millimetres are shown in Table 3-12. Some sample sieves are shown in Figure 3-25. The number and sizes of sieves used for testing a given soil will depend on the range of soil sizes in the material, and the intended use of the gradation curve. A well-organised sieve rack with sieves is shown in Figure 3-26.

Figure 3-25 Sample Sieves

Figure 3-26 Sieve Rack with Sieves

(2) Sieve shaker, a mechanical unit that can produce on duplicate samples the same consistent results as those obtained by the circular and tapping motion used in hand sieving. Typical commercially available mechanical shakers are the Tyler "Ro-Tap" (see Figure 3-27), the Combs machine and the Syntron machines; there appears to be no significant difference in the results obtained among these machines.[69]

[69] U.S. Army Engineer Waterways Experiment Station, CE, *Sieve Analyses of Granular Soils by Division Laboratories*, Engineering Study 516, (Vicksburg, Miss., October 1963).

Figure 3-27 Tyler Mechanical Shaker

(3) Balances, sensitive to 0.1 g for samples weighing less than 500 g, and to 1.0 g for samples weighing over 500 g.

(4) Paintbrush, 1”, or soft wire brush, for cleaning sieves. Sample splitter or riffle for dividing samples. Mortar and rubber-covered pestle, for breaking up aggregations of soil particles.

(5) Oven, similar to that described in 3.4.1.1.

3.6.1.3. Preparation of Sample[70]

The material to be treated is first air dried, after which the aggregations present in the sample are thoroughly broken up with the fingers or with the mortar and pestle. A representative sample is then obtained by dividing, using the sample splitter or riffle. The size of the sample to be used will depend on the maximum particle size in the sample and the requirement that the sample be representative of the material to be tested. The sample should be limited .in weight so that no sieve in the series will be overloaded. Overloading of a sieve will result in incomplete separation with errors in the test.

Table 3-12 will be used as a guide in obtaining a minimum-weight sample:

[70] Clay shale materials require special preparation. See 3.6.4.

Table 3-12 U.S. Standard Sieve Sizes and Minimum Mass of Samples for Sieve Analysis

U.S. Standard Sieve Size or Number	Sieve Openings, inches	Sieve Openings, mm	Minimum Sample Mass, g
3”	3”	76.2	64,000 g
2”	2”	50.8	19,000 g
1 1/2”	1 1/2”	38.1	8,000 g
1”	1”	25.4	2,400 g
3/4”	3/4”	19.1	1,000 g
1/2”	1/2”	12.7	300 g
3/8”	3/8”	9.52	150 g
No. 3	1/4”	6.35	
No. 4	0.187”	4.76	50 g
No. 6	0.132”	3.36	
No. 8	0.094”	2.38	
No. 10	0.079”	2.00	
No. 16	0.047”	1.19	
No. 20	0.033”	0.84	
No. 30	0.023”	0.59	
No. 40	0.0165”	0.425	
No. 50	0.0117”	0.297	
No. 70	0.0083”	0.210	
No. 100	0.0059”	0.149	
No. 140	0.0041”	0.105	
No. 200	0.0029”	0.074	

If the sample contains more than about 40% of sizes larger than the No. 4 sieve, it is generally advisable to separate the material on the No. 4 sieve, retaining both fractions for independent sieve analysis as subsequently described.

If the sample contains plastic fines which tend to form hard lumps or to coat the coarser particles during air-drying, the entire sample should be placed in a pan filled with water and allowed to soak until all the soil lumps or the coatings have disintegrated, before it is separated on the No. 4 sieve.

The coarser fraction and the fraction passing the No. 4 sieve including the fines and water should be retained for independent sieve analysis as subsequently described.

3.6.1.4. Procedure

(1) Material predominantly finer than the No. 4 sieve. The procedure for samples predominantly finer than the No. 4 sieve consists of the following steps:

   a. Record all identifying information for the sample, such as project, boring number, or other pertinent data, on a data sheet (see Figure 11-7 for suggested form).

b. Oven-dry the sample at 110º ± 5º C, allow to cool, and weigh. If the sample weighs less than 500 g, weigh it to the nearest 0.1 g; over 500 g, weigh to the nearest 1 g. Record the dry on the data sheet.

c. If the sample consists of clean sands or gravels, proceed with step (f).[71] If the sample contains plastic fines which tend to form hard lumps or to coat the coarser particles during oven-drying, place the oven-dry sample in a pan filled with enough water to cover all the material and allow it to soak until all the soil lumps or coatings have disintegrated. The length of time required for soaking will vary from about 2 to 24 hours, depending in general on the amount and plasticity of the fines.

d. Transfer the sample and water from the pan to a No. 200 sieve, or if the sample contains an appreciable amount of coarse particles, to a combined set of No. 4 and No. 200 sieves. Care should be taken not to overload the No. 200 sieve; if necessary, transfer the sample in increments. Wash the sample thoroughly through the sieves, discarding the material passing the No. 200 sieve. Larger particles in the sample may be individually washed and removed from the sieves.

e. Oven-dry the combined material retained on the No. 4 and the No. 200 sieves and, after the sample has cooled, weigh. Record on the data sheet in the "Weight Retained in grams" column the difference between the original oven-dry weight and the oven-dry weight after washing. Use the washed sample for the remainder of the analysis.

f. Select a nest of sieves suitable to the soil being tested. The choice of sieves usually depends on experience and judgment, and the use for which the grain-size curve is intended. Select as the top one with openings slightly larger than the diameter of the largest particle in the sample. Arrange the nest of sieves according to size as shown in Figure 3-28, with decreasing openings from top to bottom. Attach the bottom pan to the bottom of the smallest sieve used. Place the sample on the top sieve of the nest as shown in Figure 3-29 and put the cover plate over the top sieve.

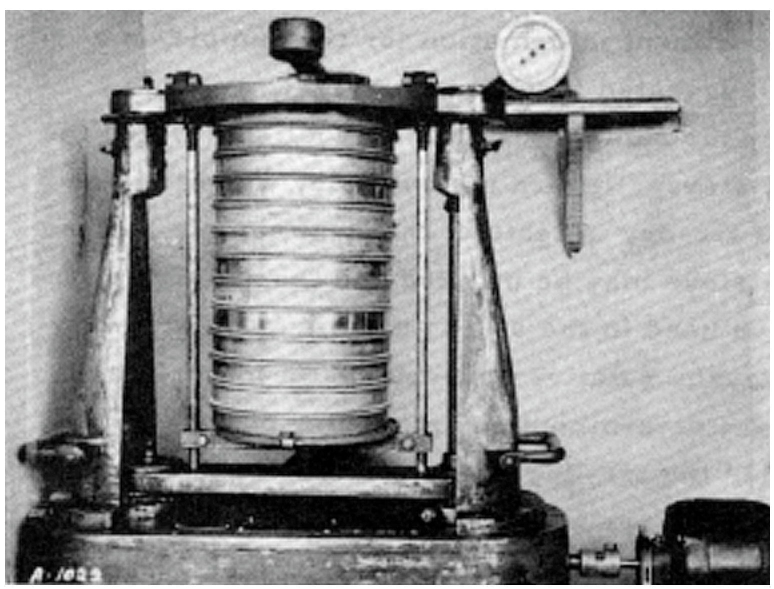
Figure 3-28 Arrangement of sieves for grain-size analysis sieve.

[71] If there is any doubt concerning the cleanness of a sand or gravel, i.e. whether or not the particles may be coated with fines, or if the test is performed to determine whether or not a material complies with specifications, then the sample should be treated as subsequently described in steps (c) through (e).

Figure 3-29 Placing Soil on Sieves

g. Place the nest of sieves in the shaking machine as shown in Figure 3-30 and shake them for 10 minutes, more or less, or until additional shaking does not produce appreciable changes in the amounts of material on each sieve. If a shaking machine is not available, the nest of sieves may be shaken by hand. In the hand operation, shake the nest of sieves with a lateral and vertical motion, accompanied by jarring, to keep the material moving continuously over the surfaces of the sieves. Jarring is accomplished by occasionally dropping the nest lightly on several thicknesses of magazines. The nest should not be broken to rearrange particles or to manipulate them through a sieve by hand. Hand shaking should be continued for at least 15 minutes.

Figure 3-30 Nest of sieves placed in typical machine for shaking



h. Remove the nest of sieves from the mechanical shaker, if used. Beginning with the top sieve, transfer the contents of the sieve to a piece of heavy paper approximately 1 ft square. Carefully invert the sieve on the paper and gently brush the bottom of the sieve, as shown in Figure 3-31, to remove the entire sample. Transfer the sample from the paper to the balance and weigh in accordance with requirements in step (b). Care should be exercised that no loss of material occurs during the transfer. Coarser fractions may be transferred more readily from the sieves directly onto the balance pan. Record the weight of material retained on each sieve on the data sheet.

Figure 3-31 Removing soil from sieves

i. Repeat step (h) for each sieve. The sum of the weights retained on each sieve and pan should equal the initial total weight of the sample within 1%. If the difference is greater than 1%, the sieving should be repeated.

(2) Material split on No. 4 sieve. The procedure for samples which have been split on the No. 4 sieve consists of the following steps:

a. Record pertinent information for the sample on a data sheet (see Figure 11-7 for suggested form).

b. Oven-dry the sample, allow it to cool, and weigh the fraction retained on the No. 4 sieve. Record the oven-dry weight on the data sheet. Alternatively, the air-dry weights of the total sample and the fraction retained on the No. 4 sieve may be utilized and the air-dry material retained on the No. 4 sieve used in the sieve analysis as in step (c) below. In the latter procedure, the relative percentages of materials greater than the No. 4 sieve are determined on an air-dry basis. This method is satisfactory provided the air-dry water contents of the plus and minus No. 4 portions of the sample are approximately equal.

c. Proceed as in paragraphs (1)f through (1)i. In general, it is advisable to use large sieves and a Ty-Lab or Gilson shaker for the coarse fraction.

d. If the sample has not been washed during the preliminary treatment, process the material passing the No. 4 sieve according to paragraphs (1)b through (1)i. If the material has been washed as part of the preliminary treatment, proceed with paragraphs (1)d through (1)i, except that the material passing the No. 200 sieve in paragraph (1)d should be oven dried and weighed. This weight is added to the oven-dry weight of the plus No. 200 material to obtain the total weight of sample.

3.6.1.5. Computation

The percentage of material by weight retained on the various sieves is computed as follows:

Equation 3-25: $$\text{Percent Retained} = 100\frac{\text{Weight In Grams Retained On A Sieve}}{\text{Total Weight In Grams Of Oven} - \text{Dry Sample}}$$

If the sample has been split on the No. 4 sieve during preliminary treatment and the air-dried coarser fraction sieved independently, the percent retained for the coarser fraction is computed as follows:

Equation 3-26: $$\text{Percent Retained} = 100\frac{\text{Air Dry Weight In Grams Retained On A Sieve}}{\text{Air Dry Total Weight In Grams Of Sample}}$$

Similarly, for the finer fraction when oven-dry weights are used:

Equation 3-27:

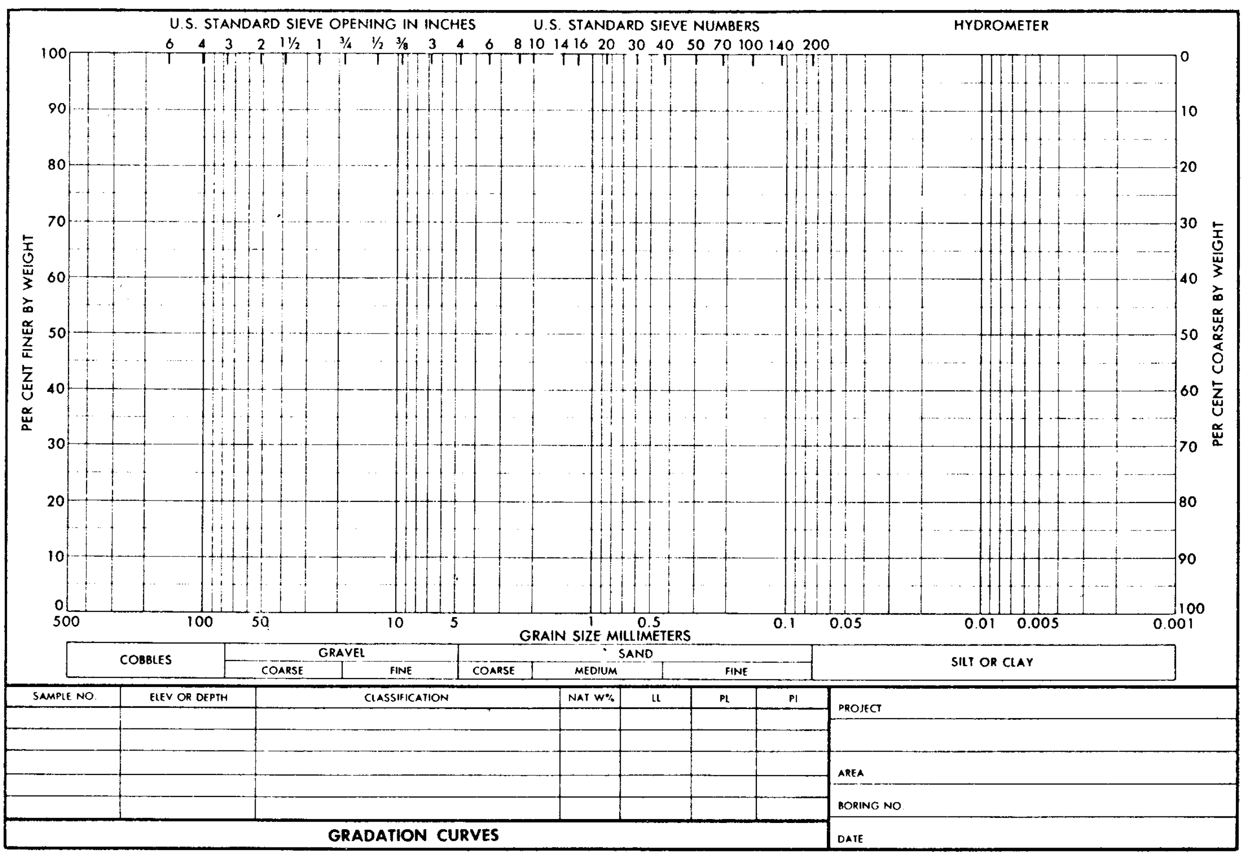
$$\text{Percent Retained} = \text{Percent Passing No. 4}\frac{\text{Weight In Grams Retained On A Sieve}}{\text{Oven Dry Weight In Grams Of Sample Passing No.4 Sieve}}$$

where the percentage passing No. 4 sieve is computed on an air-dry basis. The values of percent retained based on the above formulas refer to the total weight of sample. Computation of a partial percent retained as indicated in Figure 11-7 is necessary only when the sample is initially separated on the No. 200 sieve for purposes of a combined analysis, as subsequently described. The cumulative percent finer by weight than an individual sieve size (percent finer) is calculated by subtracting the percent retained on the individual sieve from the cumulative percent finer than the next larger sieve.

3.6.1.6. Presentation of Results

The results of the sieve analysis are presented in the form of a grain-size distribution curve on a semilogarithmic chart as shown in Figure 3-32, The grain-size distribution curve is obtained by plotting particle diameter (sieve opening) on the abscissa (logarithmic scale) and the percent finer by weight on the ordinate (arithmetic scale).

Figure 3-32 Gradation Curves



3.6.2. Hydrometer Analysis

3.6.2.1. Description

The hydrometer method of analysis is based on Stokes' law, which relates the terminal velocity of a sphere falling freely through a fluid to the diameter. The relation is expressed according to the equation:

Equation 3-28: $$v = \frac{\gamma_s - \gamma_f}{1800\eta} D^2$$

Where

- v = terminal velocity of sphere, cm/sec
- γ_s = density of sphere, g/cm^3
- γ_f = density of fluid, g/cm^3
- ν = viscosity of fluid, g-sec/cm^2
- D = diameter of sphere, mm

It is assumed that Stokes' law can be applied to a mass of dispersed soil particles of various shapes and sizes. The hydrometer is used to determine the percentage of dispersed soil particles remaining in suspension at a given time. The maximum grain size equivalent to a spherical particle is computed for each hydrometer reading using Stokes' law. The hydrometer analysis is applicable to soils passing the No. 10 sieve for routine classification purposes; Even greater accuracy is required (such as in the study of frost-susceptible soils), the hydrometer analysis should be performed on only the fraction passing the No. 200 sieve.

3.6.2.2. Apparatus

The apparatus should consist of the following:

(1) Hydrometer, calibrated at 20º/20º C (68º/68º F), graduated in specific gravity or grams/liter with a range of 0.995 to 1.040 and 0 to 50, respectively. The accuracy of the specific gravity hydrometer shall be ± 0.001 and of the gram-per-liter hydrometer, ± 1.

(2) Dispersion apparatus, either of two types may be used:

   a. A mechanically operated stirring device in which a suitably mounted electric motor turns a vertical shaft at a speed of not less than 10,000 RPM without load. The shaft shall be equipped with a replaceable stirring paddle of metal, plastic, or hard rubber. Details of a typical paddle are shown in Figure 3-33. A special dispersion cup conforming to either of the designs shown in Figure 3-33 shall be provided to hold the sample while it is being dispersed. Such as apparatus assembled is shown in Figure 3-34.

   b. An air dispersion device such as the air-jet dispersion tube device developed at Iowa State Col1ege.[72]

(3) Sedimentation cylinder, of glass, essentially 18" high and 2 1/2" in diameter and marked for a volume of 1000 ml.

(4) Centigrade thermometer, range 0º to 50º C, accurate to 0.5º C.

[72] T. Y. Chu and D. T. Davidson, "Simplified air-jet dispersion apparatus for mechanical analysis of soils," Proceedings, Highway Research Board, vol. 32 (1953), p. 541-547.

(5) Timing device, a watch or clock with a second hand.

(6) Balance, sensitive to 0.1 g.

(7) Oven (see 3.4.1.1)

Figure 3-33 Detail of stirring paddle and dispersion cups

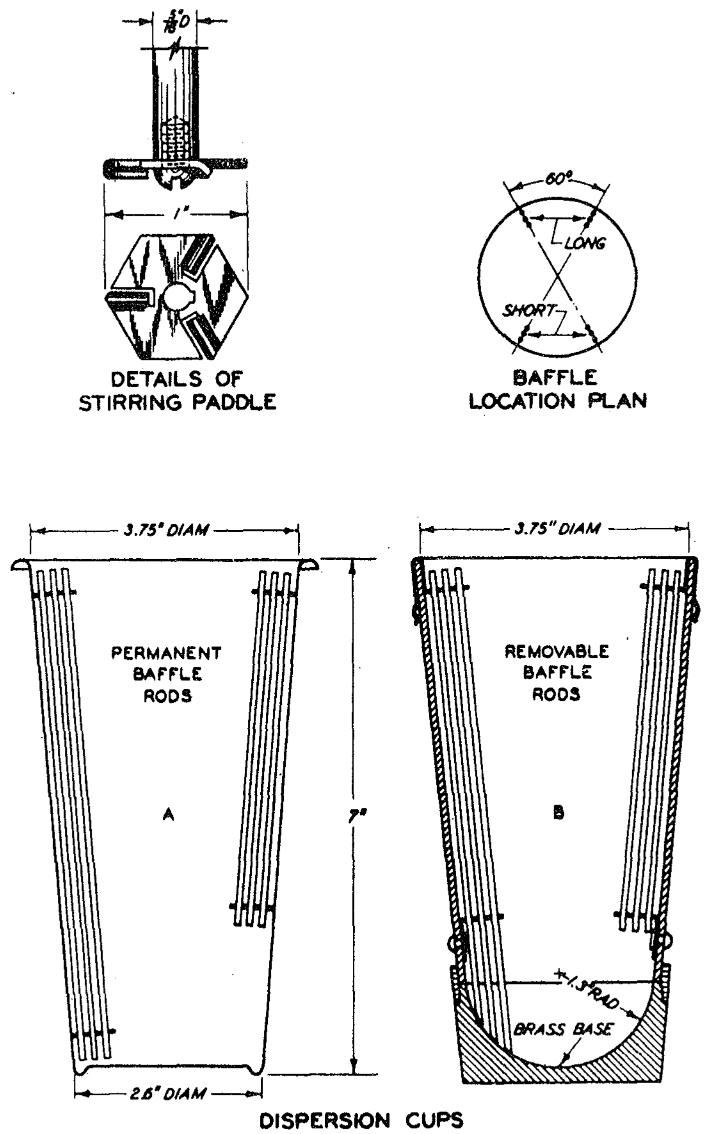


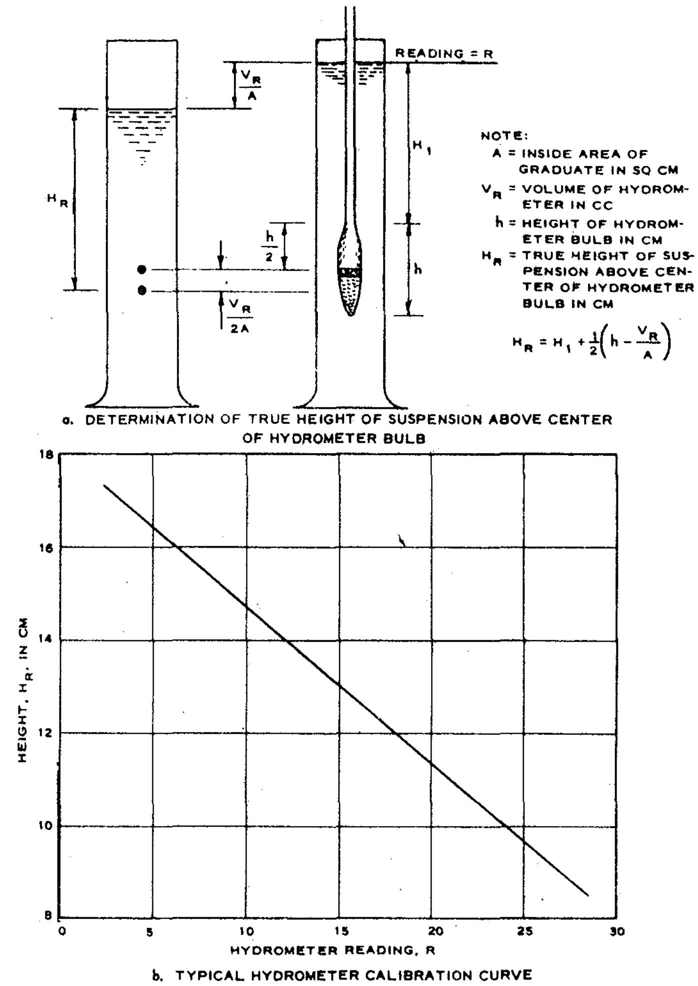
Figure 3-34 Stirring Device with Dispersion Cup

3.6.2.3. Hydrometer Calibration

The hydrometer shall be calibrated[73] to determine its true depth in terms of the hydrometer reading (see Figure 3-35) in the following steps:

[73] ASTM hydrometers 151 H or 152 H (ASTM Designation: E 100) have a uniform size; therefore, only a single calibration is required, which can be applied to all ASTM hydrometers of this type.

Figure 3-35 Hydrometer Calibration



(1) Determine the volume of the hydrometer bulb, VR. This may be determined in either of two ways:

a. By measuring the volume of water displaced. Fill a 1000 cm^3 graduate with water to approximately 700 cm^3. The water should be at about 20º C, Observe and record the reading of the water level. Insert the hydrometer and again observe and record the reading. The difference in these two readings equals the volume of the bulb plus the part of the stem that is submerged. The error due to inclusion of this latter quantity is so small that it may be neglected for practical purposes.

b. By determining the volume from the weight of the hydrometer. Weigh the hydrometer to 0.01 g on the laboratory balance. Since the specific gravity of a hydrometer is about unity, the weight in grams may be recorded as the volume in cubic centimeters. This volume includes the volume of the bulb plus the volume of the stem. The error due to inclusion of the stem volume is negligible.

(2) Determine the area, A, of the graduate in which the hydrometer is to be used by measuring the distance between two graduations. The area, A, is equal to the volume included between the graduations divided by the measured distance.

(3) Measure and record the distances from the lowest calibration mark on the stem of the hydrometer to each of the other major calibration marks, R.

(4) Measure and record the distance from the neck of the bulb to the lowest calibration mark. The distance, H_1, corresponding to a reading, R, equals the sum of the two distances measured in steps (3) and (4).

(5) Measure the distance from the neck to the tip of the bulb. Record this as h, the height of the bulb. The distance, h/2, locates the centre of volume of a symmetrical bulb. If a nonsymmetrical bulb is used, the centre of volume can be determined with sufficient accuracy by projecting the shape of the bulb on a sheet of paper and locating the centre of gravity of this projected area.

(6) Compute the true distances, HR, corresponding to each of the major calibration marks, R, from the formula:

Equation 3-29: $$H_R = H_1 + \frac{1}{2}\left(h - \frac{V_R}{A}\right)$$

(7) Plot the curve expressing the relation between HR and R as shown in Figure

(8) The relation is essentially a straight line for hydrometers having a streamlined shape.

3.6.2.4. Meniscus Correction

Hydrometers are calibrated to read correctly at the surface of the liquid. Soil suspensions are not transparent and a reading at the surface is not possible; therefore, the hydrometer reading must be made at the upper rim of the meniscus. The meniscus correction, C_m, which is a constant for a given hydrometer, is determined by immersing the hydrometer in distilled or demineralized water and observing the height to which the meniscus rises on the stem above the water surface, For most hydrometers it will be found that C_m is equal to approximately 0.5, and this value can be assumed for routine testing.

3.6.2.5. Preparation of Sample

The approximate size of sample to be used for the hydrometer analysis varies according to the type of soil being tested, as shown in Table 3-13:

Table 3-13 Sample Size for Hydrometer Analysis

Soil Type	*Dry Weight, g*
Fat clays	30
Lean clays and silty soils	50
Sandy soils	75[74]

The exact dry weight of the sample in suspension may be determined either before or after the test. However, oven-drying some clays before the test may cause permanent changes in the apparent grain sizes. Samples of such soils should, if possible, be preserved at the natural water content and tested without first being oven dried, the dry weight either being obtained after the hydrometer analysis or computed according to the formula:

[74] Up to 150 g of sandy soil can be used for the hydrometer analysis provided no more than 50 g of the sample is finer than-the No. 200 sieve.

Equation 3-30: $$\text{Dry Weight Of Specimen} = \frac{\text{Weight Of Wet Soil}}{1+\frac{\text{Water Content}}{100}}$$

w having been determined on an untested portion of the sample. Furthermore, if samples are dried and weighed before the test, any loss of material during the test will affect the results.

3.6.2.6. Dispersing Agent

Very fine soil grains in a suspension normally will tend to flocculate, i.e., to adhere with sufficient force that they settle together. Consequently, a dispersing agent to prevent flocculation of the soil grains during the test should be added to all samples. The following dispersing agents, listed in approximate order of effectiveness, have been found to be satisfactory for most types of soi1s.

Table 3-14 Dispensing Agents for Hydrometer Tests[75]

Dispersing Agents	*Stock Solution Concentration*	*Stock Solution, g/litre*	*Manufacturer*
Sodium tripolyphosphate	0.4 N	29	Blockson Chemical Co., Joliet, IL
Sodium polyphosphate	0.4 N	36	Blockson Chemical Co., Joliet, IL
Sodium tetraphosphate (trade name "Quadrafos")	0.4 N	31	Rumford Chemical Works, Rumford, RI
Sodium Hexametaphosphate (sometimes called sodium metaphosphate) adjusted to pH 8-9 with Na_2CO_3	0.4 N	41	Most laboratory chemical supply companies

The chemical product Calgon available in grocery stores shall not be used as a dispersing agent as it no longer contains sodium hexametaphosphate. Sodium silicate shall not be used as a dispersing agent since it gives unsatisfactory dispersion while at the same time permitting flocculation to a point where it is not apparent to visual examination. Phosphate solutions are somewhat unstable and therefore should not be stored for extended periods. In most instances, 15 ml of a dispersing agent solution is adequate. However, should flocculation tend to continue, a second or third addition of 15 ml of solution may be added.

The addition of a dispersing agent to the soil suspension results in an increase in density of the liquid and necessitates a correction to the observed hydrometer reading. The correction factor, C_d, is determined by adding to a 1000-ml graduate partially filled with distilled or demineralized water the amount of dispersing agent to be used for the particular test, adding additional distilled water to the 1000-ml mark, then inserting a hydrometer and observing the reading. The correction factor, C_d is equal to the difference between this reading and the hydrometer reading in pure distilled or demineralized water.

The addition of a dispersing agent also increases the weight o solids in the suspension. If the oven-dry weight of soil used for the hydrometer analysis is obtained at the end of the test, this weight must be corrected by subtracting the dry weight of the dispersing agent used.

[75] A. M. Wintermyer and E. B. Kinter, "A study of dispersing agents for particle-size analysis of soils," *Public Roads*, vol. 28, No. 3 (August 1954), pp 55-62.

3.6.2.7. Procedure

The procedure shall consist of the following steps:

(1) Record all identifying information for the sample, such as project, boring number, or other pertinent data, on a data sheet.

(2) Determine the dispersing agent correction, C_d, and the meniscus correction, C_m, unless they have been previously established. Record this information on the data sheet.

(3) Determine or estimate the specific gravity of solids and record on the data sheet.

(4) If the oven-dry weight is to be obtained at the start of the test, oven-dry the sample, allow cooling, and then weighing to nearest 0.1 g. Record the dry weight on the data sheet. Place the sample in a numbered dish and add distilled or demineralized water until the sample is submerged. Add the dispersing agent at this time. Allow the sample to soak overnight or until all soil lumps have disintegrated. Highly organic soils require special treatment, and it may be necessary to oxidize the organic matter in order to perform a hydrometer analysis on these soils. Oxidation is accomplished by mixing the sample with a solution of 30% hydrogen peroxide; this solution will oxidize all the organic matter. If only small amounts of organic matter are present, treatment with hydrogen peroxide may be omitted.

(5) Transfer the soil-water slurry from the dish to a dispersion cup (Figure 3-33), washing[76] any residue from the dish with distilled or demineralized water. Add distilled water to the dispersion cup, if necessary, until the water surface is 2” or 3” below the top of the cup; if the cup contains too much water, it will splash out while mixing. Place the cup in the dispersing machine and disperse the suspension for 1 to 10 minutes. The lower the plasticity of the soil the shorter the time required to disperse it in the cup.[77]

(6) Transfer the suspension into a 1000-ml sedimentation cylinder and add distilled or demineralized water until the volume of the suspension equals 1000 ml. The suspension should be brought to the temperature expected to prevail during the test.

(7) One minute before starting the test, take the graduate in one hand and, using the palm of the other hand or a suitable rubber cap as a stopper, shake the suspension vigorously for a few seconds in order to transfer the sediment on the bottom of the graduate into a uniform suspension. Continue the agitation for the remainder of the minute by turning the cylinder upside down and back. Sometimes it is necessary to loosen the sediment at the bottom of the cylinder by means of a glass rod before shaking. Alternatively, the suspension may be agitated by means of a hand agitator for one minute prior to testing. A schematic drawing of a hand agitator is shown in Figure 3-36. A uniform distribution of the soil particles in the suspension is accomplished by moving the hand agitator up and down through the suspension for one minute. This process also prevents the accumulation of sediment on the base and sides of the graduate.

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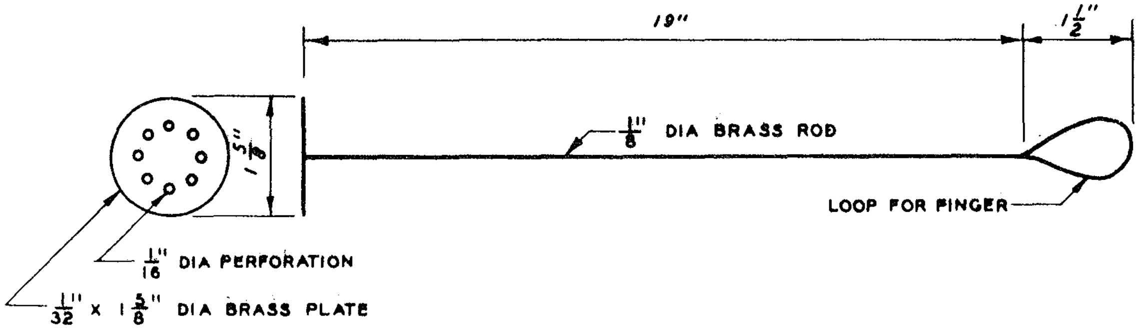
[76] A large syringe or wash-water bottle is a convenient device for handling the water in the washing operation.

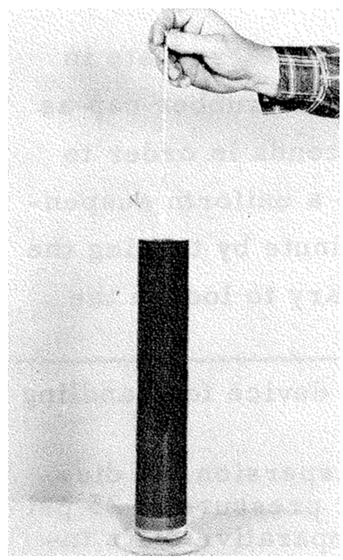
[77] Air dispersion may be used in place of mechanical dispersion. A dispersion time of 10 min is recommended, using an air pressure of 25 psi for clays and silts and 10 psi for sands. Several comparative tests indicate that the air dispersion apparatus gives a higher degree of dispersion of clayey soils while causing less degradation of sands than the mechanical stirring apparatus. See: Chu and Davidson, op. cit., and U. S. Bureau of Reclamation, *Comparison of Dispersion Methods for Soil Gradation Analysis*. Earth Laboratory Report No. EM-618 (Denver, CO, May 1961).

Figure 3-36 Hand agitator for hydrometer cylinder



(8) At the end of 1 minute, set the cylinder on a table. If foam is present, remove it from the top of the suspension by lightly touching it with a piece of soap. Slowly immerse the hydrometer in the liquid 20 to 25 seconds before each reading, as shown in Figure 3-37. Care should be exercised when inserting and removing the hydrometer to prevent disturbance of the suspension.

Figure 3-37 Immersing hydrometer in suspension prior to making observation



(9) Observe and record the hydrometer readings on the data sheet after 1 and 2 minutes have elapsed from the time the cylinder is placed on the table. As soon as the 2-minutes reading has been taken, carefully remove the hydrometer from the suspension and place it in a graduate of clean water. (If a hydrometer is left in a soil suspension for any length of time, material will settle on or adhere to the hydrometer bulb and this will cause a significant error in the reading.) Again, insert the hydrometer in the suspension and record readings after elapsed times of 4, 15, 30, 60, 120,[78] 240, and 1440 minutes, removing the hydrometer from the suspension after each reading and placing it in a graduate of clean water. Make all hydrometer readings at the top of the meniscus. For hydrometers graduated to read in specific gravity of the suspension, read only the last two figures and estimate the third. Record the

[78] A final reading after 120 min is sufficient for most soils when hydrometer analysis is used for classification purposes.

indicated specific gravity, minus 1, multiplied by 1000.[79] For hydrometers graduated to read grams/liter of suspension, record the actual reading.

(10) At the end of 2 minutes and after each subsequent hydrometer reading, place a thermometer in the suspension and record the temperature reading on the data sheet. The temperature shall be recorded to ±0.5º C. Temperature changes of the soil suspension during the test will affect the test results. Keeping the suspension away from heat sources such as radiators, sunlight, or open windows should minimize variations in temperature. A constant-temperature bath provides a convenient means of controlling temperature effects.

(11) If the dry weight of the sample is to be obtained at the end of the test, carefully wash all the suspension into an evaporating dish. Oven-dry the material, allow to cool, and then determine the sample weight. Subtract the dry weight of dispersing agent used from this weight to obtain the oven-dry weight of soil.

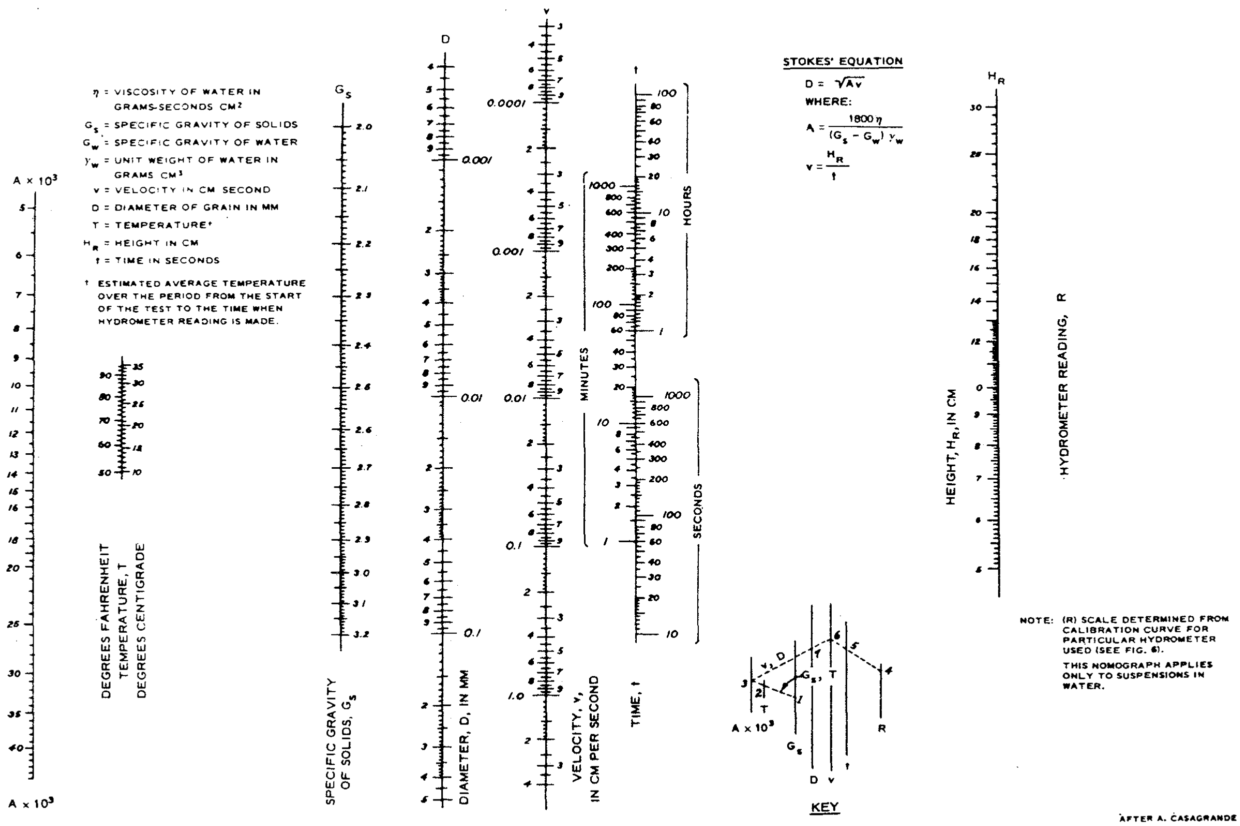
3.6.2.8. Computation

(1) Corrected hydrometer reading. Compute the corrected hydrometer readings, R, for use in computing particle diameter by adding the meniscus correction, C_m, to the actual hydrometer readings, R'. Record the corrected reading, R, on the data sheet.

(2) Computation of particle diameter. Calculate the particle diameter corresponding to a given hydrometer reading based on Stokes' equation, using the nomograph shown in Figure 3-38. The R-scale corresponding to the distances H_R is prepared using the hydrometer calibration curves as shown in Figure 3-38. The R-scale shall be designed for the particular hydrometer used in the test. A key showing the steps to follow in computing D for various values of R is shown on the chart. Record the particle diameters, D, on the data sheet.

[79] Example: the reading 1.0225 should be recorded as 22.5.

Figure 3-38 Nomographic Chart



(3) Percent finer. To compute the percent of particle diameters finer than that corresponding to a given hydrometer reading, subtract the dispersing agent correction, C_d, from the corrected hydrometer reading, R. A temperature correction factor, m, must also be added algebraically to each of the readings. This factor can be either positive or negative depending on the temperature of the suspension at the time of each reading.

(4) Obtain the temperature correction factors from Table 3-15 and record them on the data sheet. Record the values of $R - C_d + m$ on the data sheet. The $R - C_d + m$ values are used to compute percent finer according to the following formulas:

Equation 3-31: $$\text{Percent Finer By Weight} = \frac{G_s}{G_s - 1}\frac{100}{W_s}\left(R - C_d + m\right)$$

(Hydrometer Calibrated in Specific Gravity)

Equation 3-32: $$\text{Percent Finer By Weight} = \frac{100}{W_s}\left(R - C_d + m\right)$$

(Hydrometer Calibrated in Grams/Litre)

Where

- G_s = specific gravity of solids
- $W_s = W_o$ = oven-dry weight in g of soil used for hydrometer analysis
- $R - C_d + m$= corrected hydrometer reading minus dispersing agent correction plus, algebraically, temperature correction

Calculations for routine work can be greatly facilitated by using charts, tables, and other simplifying aids based on a given oven-dry weight of the sample and average specific gravity values for the major soil groups.

3.6.2.9. Presentation of Results

The data obtained from the hydrometer analysis are presented in the form of a grain-size distribution curve on a semilogarithmic chart, as shown in Figure 3-32.

Table 3-15 Temperature Correction Factor, m, for Use in Computing Percent Finer

Degrees C	Degrees F	Correction	Degrees C	Degrees F	Correction
14.0	57.2	-0.9	24.0	75.2	+0.8
14.5	58.1	-0.8	24.5	76.1	+0.9
15.0	59.0	-0.8	25.0	77.0	+1.0
15.5	59.9	-0.7	25.5	77.9	+1.1
16.0	60.8	-0.6	26.0	78.8	+1.3
16.5	61.7	-0.6	26.5	79.7	+1.4
17.0	62.6	-0.5	27.0	80.6	+1.5
17.5	63.5	-0.4	27.5	81.5	+1.6
18.0	64.4	-0.4	28.0	82.4	+1.8
18.5	65.3	-0.3	28.5	83.3	+1.9
19.0	66.2	-0.2	29.0	84.2	+2.1
19.5	67.1	-0.1	29.5	85.1	+2.2
20.0	68.0	0.0	30.0	86.0	+2.3
20.5	68.9	+0.1	30.5	86.9	+2.5
21.0	69.8	+0.2	31.0	87.8	+2.6
21.5	70.7	+0.3	31.5	88.7	+2.8
22.0	71.6	+0.4	32.0	89.6	+2.9
22.5	72.5	+0.5	32.5	90.5	+3.0
23.0	73.4	+0.6	33.0	91.4	+3.2
23.5	74.3	+0.7	33.5	92.3	+3.3
			34.0	93.2	+3.5

3.6.3. Combined Analysis

3.6.3.1. Description

A combined analysis is necessary for soils containing material finer than the U. S. Standard No. 200 sieve when the grain-size distribution of the material passing the No. 200 sieve is of interest. A sieve analysis is performed on the material retained on the No. 200 sieve, and a hydrometer test is performed on the material passing the No. 200 sieve.

3.6.3.2. Apparatus

The apparatus for the combined analysis is the same as that used for both the hydrometer and sieve analyses.

3.6.3.3. Preparation of Sample

A representative sample for the combined analysis is selected and prepared in the manner described in paragraph 2c. The total amount of sample should be sufficient to yield required amounts of material for both the sieve and hydrometer analyses. A visual inspection of the sample will usually suffice to indicate the need for intermediate steps such as large screen processing for the plus No. 4 fraction, washing, etc. Samples of soils having fines with little or no plasticity are oven dried, weighed, and then separated on the No. 200 sieve. The plus and minus No. 200 sieve fractions are preserved for the sieve and hydrometer analyses, respectively.

Soils containing plastic fines may also be oven dried initially. However, if the sample contains plastic fines which tend to form hard lumps or to coat the coarser particles during oven-drying, the sample is placed in a pan filled with enough water to cover all the material and allowed to soak until all the lumps or coatings have been reduced to individual particles. The length of time required for soaking will vary from 2 to 24 hours, depending in general on the amount and plasticity of the fines. The water and soil mixture is then washed over a No. 200 sieve (and No. 4 sieve, if necessary). The coarser fractions are preserved for a sieve analysis, and the soil and water passing the No. 200 sieve are preserved for a hydrometer analysis. Excess water with the lines is removed by evaporation, filtration, or wicking. If oven drying would alter the grain size of the plastic fines, the oven-dry weight of the fines is determined after the hydrometer test.

In routine testing when all soil particles are finer than the No. 10 sieve size, the hydrometer test may be performed on a total sample of known dry weight; the sample is then washed through the No. 200 sieve, and finally the sieve analysis is performed on the oven dried fraction retained on the No. 200 sieve.

3.6.3.4. Procedure

The procedure shall consist of the following steps:

(1) Record identifying information for the sample on both the sieve and hydrometer analysis data sheets (see Figure 11-7).

(2) Perform a sieve analysis on a representative portion of the sample retained on the No. 200 sieve, using the procedures described previously.

(3) Perform a hydrometer analysis on a portion (see paragraph 3 for approximate weight) of the sample passing the No. 200 sieve, using the procedure described previously.

3.6.3.5. Computation

The computations consist of the following steps:

(1) Compute the percentage retained on the No. 200 sieve for the total sample used in the combined analysis as follows:

$$\textbf{Equation 3-33: } \text{Percent Retained On The No. 200 Sieve} = 100\frac{W_1}{W_s}$$

Where

- ∞ W_1 = dry weight of sample retained on No. 200 sieve
- ∞ W_s = total dry weight of sample used for combined analysis

(2) Compute the data from the sieve analysis in the same manner as outlined previously, except that the percent retained for each sieve shall be based only on that portion of the total material used for the sieve analysis. As the amount of material used in the sieve analysis may be less than W_1, it will be necessary to compute a partial percent retained as follows:

Equation 3-34:

$$\text{Partial Percent Retained} = 100\frac{\text{Weight In Grams Retained On A Sieve}}{\text{Total Weight In Grams Of Oven Dry Sample Used For Sieve Analysis}}$$

The total percent retained is computed as follows:

$$\textbf{Equation 3-35: } \text{Total Percent Retained} = \text{Partial Percent Retained}\frac{W_1}{W_s}$$

The total percent finer is computed as follows:

$$\textbf{Equation 3-36: } \text{Total Percent Finer} = 100 - \text{Total Percent Retained}$$

(3) Compute the data from the hydrometer analysis in the same manner as outlined in 3.6.2.7, except that the results shall be shown in terms of a partial percent finer. As in the sieve analysis, the amount of material used for the hydrometer analysis may be less than $W_s - W_1$, therefore, a partial percent finer is computed using Equation 3-31 and Equation 3-32; the partial percent finer is the result rather than the percent finer by weight. The total percent finer is computed by the equation

$$\textbf{Equation 3-37: } \text{Total Percent Finer} = \text{Partial Percent Finer}\frac{W_s - W_1}{W_s}$$

3.6.3.6. Presentation of Results

The results of the combined analysis in terms of particle diameter and total percent finer by weight are presented in the form of grain-size distribution curves on a semilogarithmic chart as shown in Figure 3-32. Constructing a smooth curve between them joins the curves obtained from the sieve and hydrometer analyses.

3.6.4. Procedures for Preparing Clay Shale Material

The procedures for preparing clay shale material shall be the same as those described for liquid and plastic limits testing. Material for a particle-size distribution test should be removed from a processed batch and the test performed in accordance with the procedures described in this section. However, the material should not be oven dried before testing, and the hydrometer analysis should be of duration sufficient to determine the percent finer than 2-μ size.

3.6.5. Possible Errors

Following are possible errors that would cause inaccurate determinations of grain-size distribution:

3.6.5.1. Sieve Analysis

(1) Aggregations of particles not thoroughly broken. If the material contains plastic fines, the sample should be slaked -before sieving.

(2) Overloading sieves. This is the most common and most serious error associated with the sieve analysis and will tend to indicate that a material is coarser than it actually is. Large samples may have to be sieved in several portions, and the portions retained on each sieve recombined afterwards for weighing.

(3) Sieves shaken for too short a period or with inadequate horizontal or jarring motions. The sieves must be shaken so that each particle is exposed to the sieve openings with various orientations and has every opportunity to fall through.

(4) Broken or deformed sieve screens. Sieves must be frequently inspected to ensure they contain no openings larger than the standard.

(5) Loss of material when removing soil from each sieve.

3.6.5.2. Hydrometer Analysis

(1) Soil oven dried before test. Except for inorganic soils of low dry strength, oven drying may cause permanent changes in the particle sizes.

(2) Unsatisfactory type or quantity of dispersing agent. Whenever new or unusual soils are tested, trials may be necessary to determine the type and quantity of chemical that gives the most effective dispersion and deflocculation.

(3) Incomplete dispersion of soil into suspension.

(4) Insufficient shaking or agitating of suspension in cylinder at start of test.

(5) Too much soil in suspension. The results of the hydrometer analysis will be affected if the size of the sample exceeds the recommendations given.

(6) Disturbance of suspension while inserting or removing hydrometer. Such disturbance is most likely to result when the hydrometer is withdrawn too rapidly after a reading.

(7) Stem of hydrometer not clear. Dirt or grease on the stem may prevent full development of the meniscus.

(8) Nonsymmetrical heating of suspension.

(9) Excessive variation in temperature of suspension during test.

(10) Loss of material after test. If the oven-dry weight of the soil is obtained after the test, all of the suspension must be washed carefully from the cylinder.

3.6.5.3. Combined Analysis

(1) Insufficient washing of material over the No. 200 sieve. The dispersing agent should be added to the water in which the sample is soaked and the soil-water mixture should be frequently manipulated to aid the separation of particles; coarser particles may be removed from the mixture and washed free of fines by hand to reduce the quantity of material to be washed on the sieve. While the additional water used for washing should be held to a minimum, enough must be added to insure adequate removal of the fines.

(2) Loss of suspension passing the No. 200 sieve.

3.7. Relative Density

3.7.1. General

Relative density expresses the degree of compactness of a cohesionless soil with respect to the loosest and the densest conditions of the soil that can be attained by specified laboratory procedures; a soil in the loosest state would have a relative density of 0% and in the densest state, of 100%. The dry unit weight of a cohesionless soil does not, by itself, reveal whether the soil is loose or dense, due to the influence of particle shape and gradation on this property. Only when viewed against the possible range of variation, in terms of relative density, can the dry unit weight be related to the compaction effort used to place the soil in an embankment or indicate the volume-change tendency of the soil when subjected to shear stresses. Originally, relative density, D_d was defined by the equation

Equation 3-38: $$D_d = 100\frac{e_{\max} - e}{e_{\max} - e_{\min}}$$

Where

- D_d = Relative Density, percent
- e_{max} = void ratio of the soil in the loosest state which can be attained in the laboratory
- e = void ratio of the soil in place
- e_{min} = void ratio of the soil in the densest state which can be attained in the laboratory

However, for ease of computation, relative density may be expressed in terms of dry unit weight by the equation

Equation 3-39: $$D_d = 100\frac{\frac{1}{\gamma_{\min}} - \frac{1}{\gamma_d}}{\frac{1}{\gamma_{\min}} - \frac{1}{\gamma_{\max}}}$$

Where

- γ_d = dry unit weight of the soil in place, called "in-place density
- γ_{min} = dry unit weight of the soil in the loosest state which can be attained in the laboratory, called "minimum density "
- γ_{max} = dry unit weight of the soil in the densest state which can be attained in the laboratory, called "maximum density."

The in-place density of a soil may be determined by various methods in the field or from undisturbed samples brought into the laboratory; its determination will not be considered in a test procedure that covers only the methods for determining the minimum and maximum densities. There are few difficulties in determining minimum density by a standard method, but restrictions imposed by the availability of special apparatus under certain conditions or the character of the soil may not permit the determination of maximum density by a single, standard method to the exclusion of all others. However, the method described herein, employing a vibratory table, shall be preferred to all other methods;[80] the alternative

[80] Other methods are currently being investigated to overcome the problem of segregation encountered in some soils using the vibratory table.

method for attaining the maximum denseness of a soil by hammering on the mould should be followed only when the use of the vibratory table is not feasible. The method used for determining the maximum density must be reported when presenting relative density data.

3.7.2. Apparatus

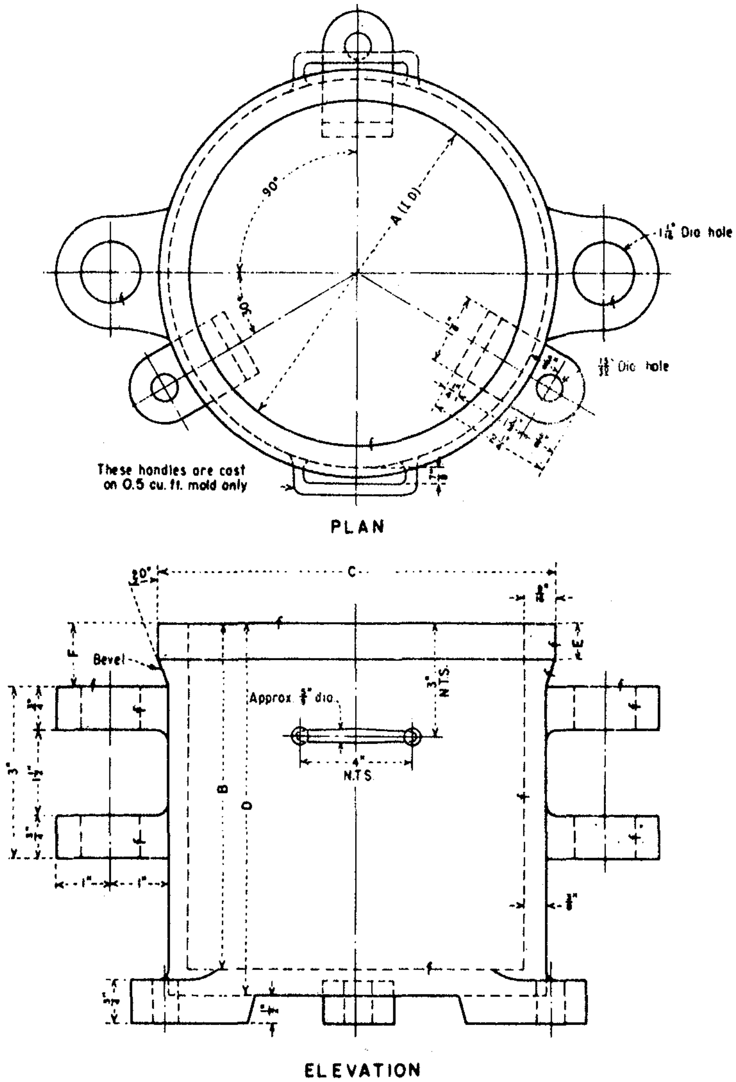
The apparatus for determining the minimum and maximum densities of a cohesionless soil shall consist of the following:

a. Cylindrical moulds or measures of 0.1 ft^3 capacity (6" inside diameter) and 0.5 ft^3 capacity (11" inside diameter), as shown in Figure 3-39.[81] The moulds should be cast of silicon aluminum alloy ASTM – SG70A.

b. Surcharge assemblies, to fit each size mould, as shown in Figure 3-40. Each assembly shall include a surcharge weight (equivalent to 2 psi), a surcharge base plate with handle and a guide sleeve with clamp assemblies.

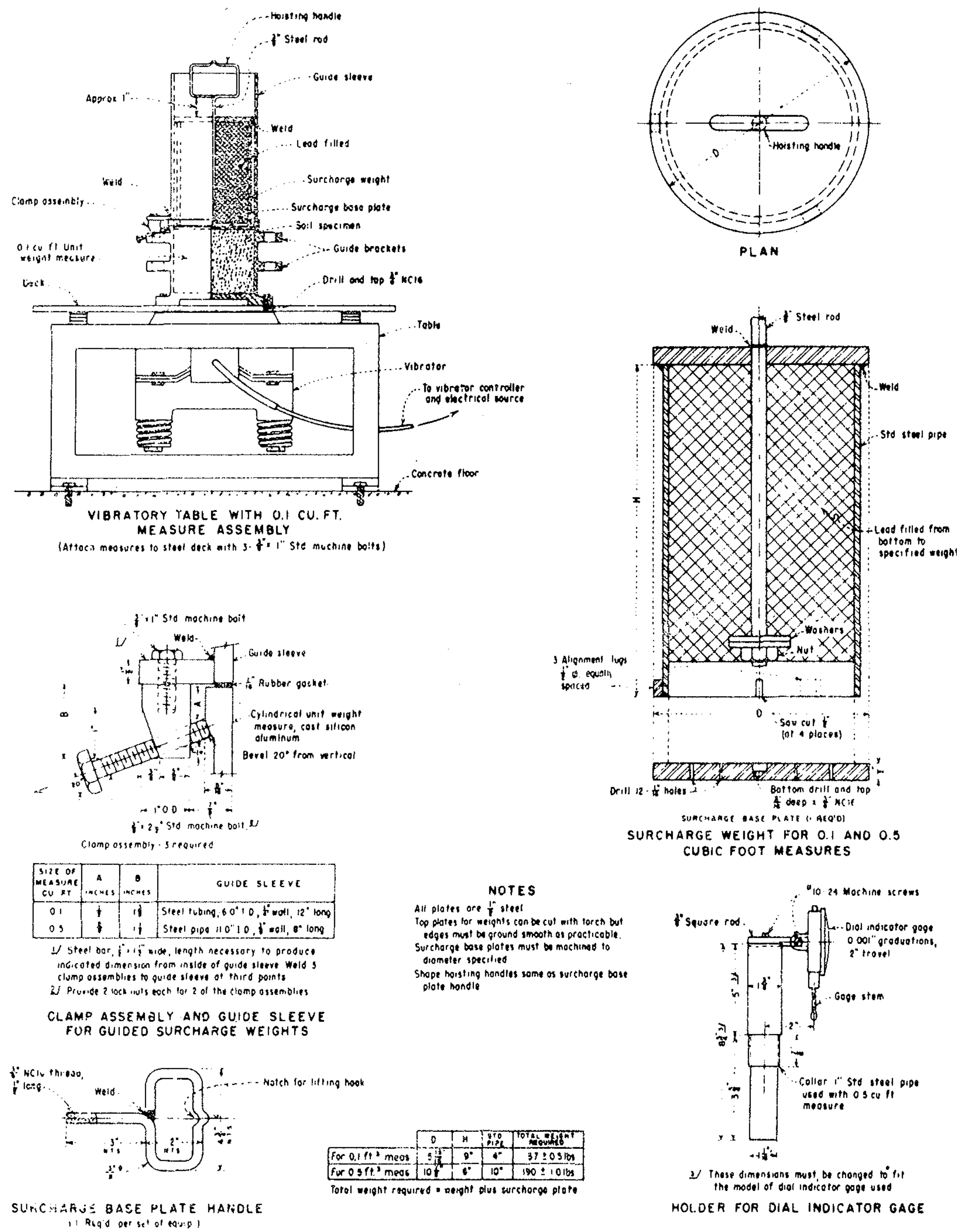
[81] These moulds, as well as the surcharge assemblies and dial indicator holder shown in Figure 3-40, are available commercially from Stebbins Mfg. & Supply Co., 1733 Blake St., Denver, CO.

Figure 3-39 Details of Molds



MOLD SIZE (CU. FT.)	DIMENSIONS					
	A	B	C	D	E	F
0.1	6.000"	6.112"	7[illegible]"	6 1/2"	[illegible]"	1[illegible]"
0.5	11.000"	9.092"	12[illegible]"	9[illegible]"	[illegible]"	2"

Figure 3-40 Equipment for Maximum Density Determination



c. Dial indicator, having 0.001” graduations and a 2” range.

d. Holder, for dial indicator, with collar, as shown in Figure 3-40.

e. Calibration bar, metal, 3” by 1/8” by 12” long.

f. Vibratory table, as shown in Figure 3-40, with a cushioned steel vibrating deck about 30 by 30”, actuated by an electromagnetic vibrator and mounted to a concrete floor or slab of large mass. The vibrator should be a semi-noiseless type with a net weight of over 100 lbs. The vibrator shall have a frequency of 3600 vibrations/minute (60 Hz) and variable vibrator amplitudes to a maximum of at least 0.015" under a 250 pound load, and be suitable for use with 230-volt alternating current.[82]

g. Hoist, having a capacity of at least 300 lb, for handling the heavier surcharge weight.

h. Pouring device, as shown in Figure 3-41, which has metal funnels with 1/2" and 1" diameter cylindrical spouts, each attachable to a metal can 6" in diameter by 12" high.

Figure 3-41 Pouring device, with 1" diameter spout

i. Hand scoop, large, metal.

j. Mixing pans, large, metal.

k. Sample splitter or riffle.

l. Straightedge, 15", steel.

m. Platform scales, having a capacity of at least 100 lb and sensitive to 0.01 lb.

n. Oven.

3.7.3. Calibration of Equipment

Each mould must be calibrated as follows:

a. Determine the weight, W_m, of each mould to the nearest 0.01 lb.

---

[82] The Syntron VP-80 and VP-240 Vibratory Packers, manufactured by Syntron Co., Homer City, PA, have proven satisfactory. (The VP-80 has been replaced by a later model, VP-86, with the same characteristics.)

b. Determine the capacity, or total inside volume, V_m, of the 0.1 ft^3 mold to the nearest 0.0001 ft^3 and of the 0.5 ft^3 mould to the nearest 0.001 ft^3.

c. Determine the inside cross-section area, A_m, at the open end of the 0.1-ft^3 mould to the nearest 0.0001 ft_ and of the 0.5-ft^3 mould to the nearest 0.001 ft^3.

d. Determine the initial dial reading, h_o, for each mould and surcharge base plate combination in the following manner:

   (1) Measure the thickness of the calibration bar, t_c, and of the surcharge base plate, t_s, to the nearest 0.001"

   (2) Lay the calibration bar across the top of the mould along the axis defined by the brackets for the dial indicator holder, as shown in Figure 3-42a.

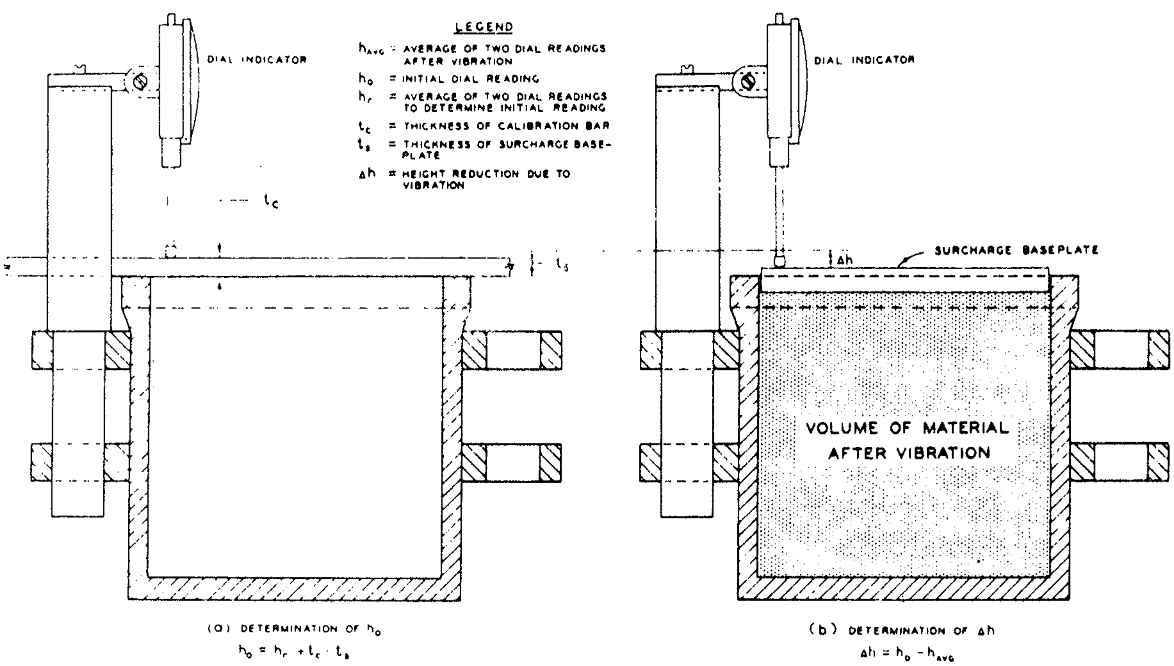
   (3) Insert the dial indicator holder into the brackets on one side of the mould so that the dial indicator stem rests on the calibration bar, and note the dial reading. Then insert the dial indicator holder into the brackets on the opposite side of the mould and note the dial reading. The dial indicator holder should be placed in the same position in the guide brackets for each reading by means of match marks on the holder and on the brackets.

   (4) Compute the average, h_r, of the two dial readings.

   (5) Compute the initial dial reading, h_o, by the equation

Equation 3-40: $h_o = h_r + t_c - t_s$

Figure 3-42 Determination of Reduction in Sample Height due to Vibration



3.7.4. Preparation of Sample

The soil to be tested must be oven dried and then permitted to cool in an airtight container. Aggregations of fine particles shall be thoroughly broken and a representative sample removed from the soil using a sample splitter or riffle. The representative sample should weigh at least 25 lb if the maximum particle size is less than 1 1/2" and at least 100 lb if the maximum particle size is between 1 1/2" and 3". If the sample contains more than 10% by weight particles larger than 3”, the determination the maximum and minimum density becomes a test of a research nature.

3.7.5. Procedure

Every precaution must be observed while handling the sample to prevent segregation and to preserve the oven dried condition. The minimum and maximum densities shall be determined using the same sample, by first placing the soil into a mould in the loosest possible state to attain the minimum density and then vibrating it into the densest possible state to attain the maximum density. The 0.1-ft^3 mould shall be used for the determinations if the maximum particle size is less than 1 1/2” and the 0.5-ft^3 mould shall be used if the maximum particle size is between 1 1/2” and 3”

3.7.5.1. Minimum Density Determination

The procedure for determining the minimum density shall consist of the following steps:

(1) Record all identifying information for the sample such as project, boring number, etc., on a data sheet (Figure 11-8 is a suggested form).

(2) Record the weight, inside volume, and end area of the mould on the data sheet.

(3) Carefully place the oven-dried soil into the mould in the loosest possible condition.[83] Fill the mould in layers, using the pouring device for material having maximum particle sizes less than 3/8” or the hand scoop for material having larger particle sizes. Exercise the greatest care at all times to avoid jarring the mould or otherwise disturbing the previously placed layers. When using the pouring device (with the 1/2” diameter spout if the maximum particle size passes the No. 4 sieve and the 1” diameter spout if the maximum particle size is between the No. 4 sieve and 3/8” sieve), adjust the height of the spout to maintain a free fall of the soil of about 1”. With a steady flow of soil from the spout, move the pouring device in a spiral path from the outside to the centre of the mould to form each layer of uniform thickness without segregation.[84] When the maximum particle size of the sample exceeds 3/8”, place the soil into the mould by means of the scoop held as closely as possible to the previously placed layer so the soil slides but does not fall from the scoop; restrain the larger particles with the hand where necessary to prevent their rolling from the scoop. Continue filling the mould until the soil rises slightly above the top of the mould, with care that no large particles that project above the top of the mould are placed in the final layer. Using the straightedge, carefully trim the soil surface level with the top of the mould.

(4) If the maximum density of the oven dried sample is not to be determined,[85] weigh the mould and soil to the nearest 0.04 lb and record the weight on the data sheet; alternatively, the contents of the mould may be emptied into a mixing pan and weighed.

(5) Steps (3) and (4) should be repeated until consistent results (within 1%) are attained.

3.7.5.2. Maximum Density Determination with Oven dried Sample

The procedure for determining the maximum density shall consist of the following steps:

---

[83] If the maximum density of the oven dried sample is to be determined also, the mould may be attached to the deck of the vibratory table before filling (see paragraph 5b(2)).

[84] Static electricity in dry sand can cause bulking similar to that produced by a trace of moisture on the particles; a static eliminating balance brush can be applied to the equipment in contact with the sand when this effect becomes bothersome.

[85] If the maximum density of the oven dried sample is to be determined, also proceed in accordance with 3.7.5.2.

(1) Proceed in accordance with the first three steps of the "Minimum Density Determination" described above.

(2) Attach the mould to the deck of the vibratory table, if this had not been done prior to filling the mould with soil.

(3) Place the guide sleeve on the top of the mould and clamp it firmly to the mould.[86] Lower the surcharge base plate onto the surface of the soil and remove the handle. Using the hoist if necessary, lower the surcharge weight onto the surcharge base plate.

(4) It has been determined that for a particular vibrating table, mould, and surcharge assembly, the maximum dry density of a specimen may be obtained at a displacement amplitude (rheostat setting) less than the maximum amplitude of which the apparatus is capable; i.e., dry density may increase with increase in rheostat setting to a setting, beyond which the dry density decreases. Therefore, each laboratory should determine for its apparatus the rheostat setting at which maximum density is produced and use this setting for subsequent maximum density testing.[87] A clean, durable, subrounded to rounded material should be used in making these determinations (in both 0.1- and 0.5-ft^3 moulds). The test should be performed as given in this appendix, except that the rheostat setting should be increased from zero to 100 in increments of 10 and measurements taken after each period of vibration. The material should be vibrated for a period of 8 minutes at each rheostat setting and not removed until after the last determination has been made (100 percent rheostat setting). The particle size distribution of the specimen should be determined before and after the test to assess the extent of degradation, if any.

(5) Remove the surcharge weight and guide sleeve from the mould, and obtain dial indicator readings on opposite sides of the surcharge base plate, as shown in Figure 3-42b. Record the dial readings on the data sheet.

(6) Remove the surcharge base plate from the mould and detach the mould from the vibratory table.

(7) Weigh the mould and soil to the nearest 0.01 lb. and record the weight on the data sheet; alternatively, the contents of the mould may be emptied into a mixing pan and weighed.

3.7.6. Computations

The computations shall consist of the following steps:

a. Compute the weight of the dry soil, W_s, by subtracting the weight of the mould, W_m (or mixing pan) from the weight of the mould (or mixing pan) and soil, W.

b. For the minimum density determination, the volume of the soil, V, is equal to the total inside volume of the mould, V_m.

c. For the maximum density determination, the volume of the soil, V, is equal to the total inside volume of the mould, V_m, minus the volume change, ΔV, caused by the vibrating of the soil. The volume change shall be computed in the following manner:

   (1) Compute the average dial reading, h_{avg}, of the two readings taken after vibrating the soil.

   (2) Subtract the initial dial reading, h_o, from the average dial reading, h_{avg}, to obtain the height change, Δ_h.

---

[86] The inside surface of the guide sleeve must align with the inside surface of the mould, so two of the three clamping bolts should be provided with lock nuts, as noted in Figure 3-40. By properly adjusting and locking these two bolts, the guide sleeve will be drawn automatically into correct alignment when the third bolt is tightened.

[87] It may be desirable to redetermine the optimum rheostat setting at the inception of testing for each major project.

(3) Compute the volume change by the equation

$$\textbf{Equation 3-41: } \Delta V = \frac{\Delta h}{12} A_m$$

Where

ΔV = volume change, ft_

Δh = height change, inches

A_m = end area of mould, ft_

d. Compute the minimum and maximum densities to the nearest 0.1 lb/ft^3 by the equation

$$\textbf{Equation 3-42: } \gamma_d = \frac{W_s}{V}$$

Where

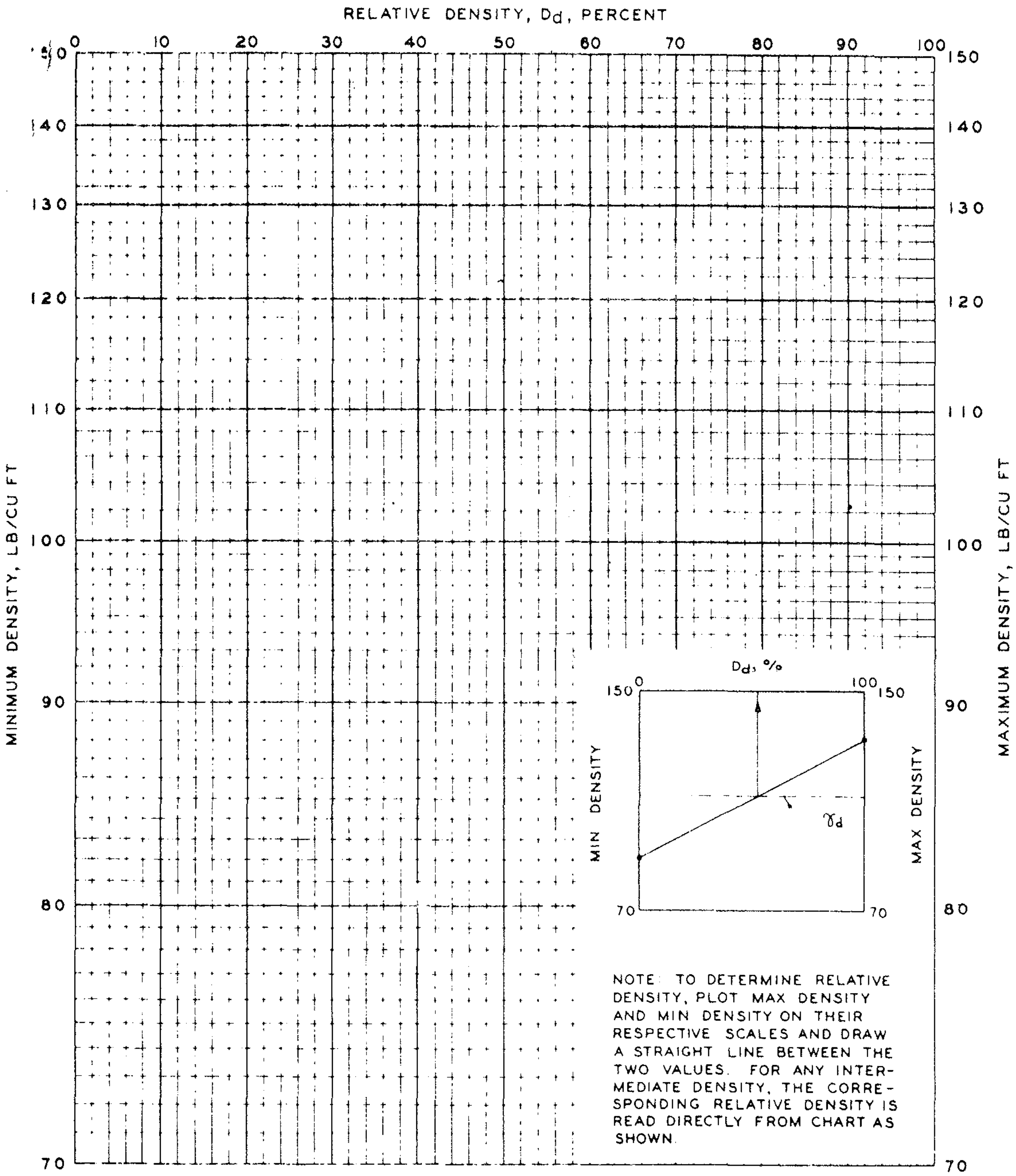
∞ γ_d = dry density, lb/ft_

∞ W_s = weight of dry soil, lbs.

∞ V = volume of soil, ft_

e. If the in-place density of the soil is known, the relative density D_d can be computed by Equation 3-39 or determined graphically by means of Figure 3-43.

Figure 3-43 Graphical Determination of Relative Density



3.7.7. Possible Errors

Following are possible errors that would cause inaccurate determinations of relative density:

3.7.7.1. General

(1) Test not appropriate to type of soil. The relative density is meaningful only for cohesionless materials; if a soil has any appreciable dry strength, the methods for determining the minimum and maximum densities described here are not applicable.

(2) Material segregated while being processed.

(3) Gain in moisture of oven-dried material before or during testing. A small amount of moisture in the soil can cause erroneous measurements of the minimum density and, to a much lesser degree, of the maximum density.

(4) Moulds not accurately calibrated.

3.7.7.2. Minimum density determination

(1) Disturbance of mould during filling. Inadvertent jarring of the mould or impact of the falling particles will increase the measured minimum density.

(2) Segregation of material while filling mould.

(3) Loss of material from mould before weighing. To prevent spilling any material before the mould and contents can be weighed, rap the side of the mould carefully to settle the contents.

3.7.7.3. Maximum Density Determination

(1) Insufficient amplitude of vibratory table under load. Measurements should be made at least once to verify that the requirements given in paragraph 2f are satisfied.

(2) Loss of fine material from mould during vibration. Fine particles may escape from the mould during vibration as dust.

(3) Misalignment of guide sleeve with mould. The adjustment of the clamping bolts must be checked periodically to ascertain that misalignment of the guide sleeve will not cause binding of the surcharge weight.

§ 4. Field Exploration, Testing, and Instrumentation

This section contains information on exploration methods including use of air photos and remote sensing, geophysical methods, test pits, test borings, and penetrometers. Also presented is information on methods of sampling, measuring *in situ* properties of soil and rock, field measurements, and geotechnical monitoring equipment.

4.1. Overview

The initial step in any project must include consideration of the soil or rock on which the embankments and structures are to be supported. The extent of the site investigation will depend on many factors, not the least of which will be the project scheduling, general subsurface conditions, and the nature of the loads to be supported. In any event, certain basic steps should be followed before a drill rig moves onto the project. The first step in the investigation is to collect and analyse all existing data.

The area concept of site investigation allows the foundation engineer to extend the results from a limited number of explorations in a particular landform to the entire deposit. This concept is a powerful tool in reducing subsurface exploration costs and in providing the planning engineer the following useful data in the location phase:

1. *Design*. Knowledge of the landforms and of the engineering properties of the soils enables the designer to determine the most economical location for highway alignment and grade, to evaluate design problems for each type of soil deposit, and to determine sources of granular borrow.
2. *Construction*. The type and extent of problem soils to be encountered during construction may be predetermined, and construction cost more accurately estimated.

4.2. General Requirements of Field Investigations

The initial phase of field investigations should consist of detailed review of geological conditions at the site and in its general environs. This should include a desktop study of available data including historical data, remote sensing imagery, aerial photography, and a field reconnaissance. The information obtained should be used as a guide in planning the exploration. Sources of historical site data are described in Table 4-1.

Table 4-1 Sources of Historical Site Data

Source	Functional Use	Location	Examples
Utility Maps	• Identifies buried utility locations • Identifies access restrictions • Prevents damage to utilities	Local agencies/utility companies	Power line identification prior to an intrusive investigation prevents extensive power outage, expensive repairs, and bodily harm
Aerial Photographs	• Identifies manmade structures • Identifies potential borrow source areas • Provides geologic and hydrological information which can be used as a basis for site reconnaissance • Track site changes over time	Local Soil Conservation Office, United States Geological Survey (USGS), Local Library, Local & National aerial survey companies	Evaluating a series of aerial photographs may show an area on site which was filled during the time period reviewed
Topographic Maps	• Provides good index map of site area • Allows for estimation of site topography • Identifies physical features in the site area • Can be used to assess access restrictions	USGS, State Geological Survey	Engineer identifies access areas/restrictions, identifies areas of potential slope instability; and can estimate cut/fill capacity before visiting the site
Existing Subsurface Investigation Report	• May provide information on nearby soil/rock type; strength parameters; hydrogeological issues; foundation types previously used; environmental concerns	USGS, United States Environmental Protection Agency (USEPA), State DOTs	A five year old report for a nearby roadway widening project provides geologic, hydrogeologic, and geotechnical information for the area, reducing the scope of the investigation
Geologic Reports and Maps	• Provides information on nearby soil/rock type and characteristics; hydrogeological issues, environmental concerns	USGS and State Geological Survey	A twenty year old report on regional geology identifies rock types, fracture and orientation and groundwater flow patterns
Water/Brine Well Logs	• Provide stratigraphy of the site and/or regional area • Varied quality from state to state • Groundwater levels	State Geological Survey/Natural Resources	A boring log of a water supply well two miles from the site area shows site stratigraphy facilitating evaluations of required depth of exploration
Flood Insurance Maps	• Identifies 100 and 500 yr. floodplains near water bodies • May prevent construction in a floodplain • Provide information for evaluation of scour potential	Federal Emergency Management Agency (FEMA), USGS, State/Local Agencies	Prior to investigation, the flood map shows that the site is in a 100 yr floodplain and the proposed structure is moved to a new location
Soil Survey	• Identifies site soil types • Permeability of site soils • Climatic and geologic information	Local Soil Conservation Service	The local soil survey provides information on near-surface soils to facilitate preliminary borrow source evaluation
Sanborn Fire Insurance Maps	• Useful in urban areas • Maps for many cities are continuous for over 100 yrs. • Identifies building locations and type • Identifies business type at a location (e.g., chemical plant) • May highlight potential environmental problems at an urban site	State Library/Sanborn Company (www.sanborncompany.com)	A 1929 Sanborn map of St. Louis shows that a lead smelter was on site for 10 years. This information prevents an investigation in a contaminated area.

To the extent possible, borings should be supplemented by lower cost exploration techniques such as test pits, probes, seismic refraction surveys, and electrical resistivity surveys. This is particularly true in the offshore environment where borings are exceptionally expensive.

The extent of the exploration will vary considerably with the nature of the project. However, the following general standards apply to all investigation programs or as appropriate for the specific project and agreed upon:

1. Preliminary exploration depths should be estimated from data obtained during field reconnaissance, existing data, and local experience. The borings should penetrate unsuitable founding materials (organic soils, soft clays, loose sands, etc.) and terminate in competent material. Competent materials are those suitable for support of the foundations being considered.
2. All borings shall be extended below the estimated scour depths.
3. Each boring, sounding, and test pit should be given a unique identification number for easy reference.
4. The ground surface elevation and actual location should be noted for each boring, sounding, and test pit. Offshore borings should be referenced to mean sea level with the aid of a tide gauge. (Note: There are two vertical data. They are the 1927 datum and the 1988 datum; ensure that the proper one is being referenced.)

5. A sufficient number of samples, suitable for the types of testing intended, should be obtained within each layer of material.
6. Water table observation within each boring or test pit should be recorded when first encountered, at the end of each day and after sufficient time has elapsed for the water table to stabilize. Refer to ASTM D 4750. Other groundwater observations (artesian pressure, etc.) should also be recorded.
7. Unless serving as an observation well, each borehole, sounding, and test pit should be backfilled or grouted according to applicable environmental guidelines.

4.3. Initial Field Investigations

Following review of the existing data, the geotechnical engineer should visit the project site. This will enable the engineer to gain first-hand knowledge of field conditions and correlate this information with previous data. The form included as Figure 4-1 indicates the type of information the engineer should look for. In particular, the following should be noted during the field reconnaissance:

1. Nearby structures should be inspected to ascertain their foundation performance and potential to damage from vibration or settlement from foundation installation. In addition, the structure's usages must be looked at to check the impact the foundation installation may have (i.e. a surgical unit, printing company, etc.).
2. On water crossings, banks should be inspected for scour and the streambed inspected for evidence of soil deposits not previously indicated.
3. Note any feature that may affect the boring program, such as accessibility, structures, overhead utilities, signs of buried utilities, or property restrictions.
4. Note any feature that may assist in the engineering analysis, such as the angle of any existing slopes and the stability of any open excavations or trenches.
5. Any drainage features, including signs of seasonal water tables.
6. Any features that may need additional borings or probing such as muck pockets.

Figure 4-1 Field Reconnaissance Report

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

FIELD RECONNAISSANCE REPORT

FORM 675-020-14
MATERIALS
08/94

PROJECT NO.: ______ COUNTY ______ STA NO. ______

REPORTED BY: ______ DATE ______

1. STAKING OF LINE
- ☐ WELL STAKED
- ☐ POORLY STAKED (WE CAN WORK)
- ☐ POORLY STAKED (WE MUST REPLACE)
- ☐ REQUEST DIVISION TO RESTAKE

2. BENCH MARKS
IN PLACE: ☐ YES ☐ NO
DISTANCE FROM BRIDGE - METERS ______

3. PROPERTY OWNERS
GRANTED PERMISSION: ☐ YES ☐ NO
REMARKS ON BACK

4. UTILITIES
WILL DRILLERS ENCOUNTER UNDERGROUND UTILITIES?
☐ YES ☐ NO
AT WHICH HOLES? ______
WHAT TYPE? ______
WHO TO SEE FOR DEFINITE LOCATION ______

5. GEOLOGIC FORMATION

6. SURFACE SOILS
☐ SAND ☐ CLAY ☐ SANDY CLAY ☐ SILT ☐ MUCK
☐ OTHER

7. GENERAL SITE DESCRIPTION
☐ LEVEL ☐ ROLLING ☐ HILLSIDE ☐ VALLEY
☐ SWAMP ☐ GULLIED
GROUND COVER
☐ CLEARED ☐ FARMED ☐ BUILDINGS
☐ HEAVY WOODS ☐ LIGHT WOODS
☐ OTHER
REMARKS ON BACK

8. BRIDGE SITE
REPLACING ______
WIDENING ______
RELOCATION ______
RIG TYPE ______
- ☐ TRUCK MOUNTED SKID RIG
- ☐ SKID RIG
- ☐ ROCK CORING RIG
- ☐ WASH BORING EQUIPMENT
- ☐ WATER WAGON
- ☐ PUMP
- ☐ HOSE - METERS ______

8. BRIDGE SITE - CONTINUED
CUT SECTION - METERS ______
FILL SECTION - METERS ______
IF STREAM CROSSING:
WILL BARGE BE NECESSARY: ______
EASILY PLACED IN WATER ______
CAN CABLE BE STRETCHED ACROSS STREAM ☐ YES ☐ NO
HOW LONG? ______
CURRENT: ☐ SWIFT ☐ MODERATE ☐ SLOW
IF PRESENT BRIDGE NEARBY:
TYPE OF FOUNDATION ______
ANY PROBLEMS EVIDENT IN OLD BRIDGE (DESCRIBE ON BACK)
IS WATER NEARBY FOR WET DRILLING - METERS ______

9. GROUND WATER TABLE
CLOSE TO SURFACE - METERS ______
NEARBY WELLS - DEPTH - METERS ______
INTERMEDIATE DEPTH - METERS ______

10. ROCK
BOULDERS OVER AREA? ☐ YES ☐ NO
DEFINITE OUTCROP? ☐ YES ☐ NO
(SHOW SKETCH ON BACK) WHAT KIND? ______

11. SPECIAL EQUIPMENT NECESSARY

12. REMARKS ON ACCESS
DESCRIBE ANY PROBLEMS ON ACCESS

13. DEBRIS AND SANITARY DUMPS
STATIONS ______
REMARKS ______

4.4. Guidelines for Detailed Explorations

Following is a description of the recommended minimum explorations for various types of projects. It is stressed that these guidelines represent the minimum extent of exploration and testing anticipated for most projects and must be adapted to the specific requirements of each individual project.

It is noted that the guidelines below consider the use of conventional borings only. While this is the most common type of exploration, the engineer may deem it appropriate on individual projects to include soundings, test pits, geophysical methods, or in-situ testing as supplementary explorations or as substitutes for some, but not all, of the conventional borings noted in the following sections.

4.4.1. General Scope of Program

In regard to the scope of the subsurface program for a structure, one must carefully consider the small cost of a boring in relation to the foundation cost. A 2 1/2" diameter drill hole will cost less than one 12" diameter pile. Yet, the knowledge gained from that boring would permit proper design techniques to be used that may allow elimination of all piles for that structure. Without adequate boring data, the foundation design engineer cannot utilize his technique or experience and must rely on extremely conservative designs with high safety factors.

Planning a soils or foundation exploration program should include determining the depth and location of borings, test pits, or other procedures to be used and establishing the methods of soil sampling and testing to be employed. Usually, the extent of the work is established as it progresses, unless knowledge of foundation conditions is available from geological studies, earlier investigations, or records of existing structures. The number, depth, spacing, and character of tests to be made in any individual exploration program are so dependent upon site conditions, type of structure, and its requirements, that no rigid rules may be established. However, certain general principles for the guidance of those charged with the investigation can be outlined.

Embankments are less sensitive than structures to variations in subsurface conditions. Embankment loads are spread over a wide area while structure loads are concentrated. Designers of highways in cut sections are less concerned with deep exploration of subsurface conditions than defining the properties of the soil or rock on which the subgrade materials will be placed. The subsurface exploration program for embankments or cuts must necessarily be widely spaced, as the major portion of a highway alignment is one or the other. This section of the manual will deal primarily with approach embankments. Highway embankment and cut explorations are done using the same procedures, but the spacing and depth of borings vary, as shown below.

4.4.2. Investigation Steps

The objective of either deep or shallow borings is to obtain information and samples necessary to define soil and rock subsurface conditions. The following program will produce the minimum foundation data for a typical structure site. Soft ground conditions may require undisturbed sample explorations or *in situ* testing as previously mentioned.

- Stratigraphy
    - Physical description and extent of each stratum.
    - Thickness and elevation of various locations of top and bottom of each stratum.
- For cohesive soils (each stratum).
    - Natural moisture contents.
    - Atterberg limits.
    - Presence of organic materials.

    - Evidence of desiccation or previous soil disturbance, shearing, or slickensides.
    - Swelling characteristics.
    - Shear strength
    - Compressibility
- For granular soils (each stratum).
    - In-situ density (average and range) typically determined from Standard Penetration Tests or Cone Tests.
    - Grain-size distributions (gradation).
    - Presence of organic materials.
- Ground water (for each aquifer if more than one is present).
    - Piezometric surface over site area, existing, past, and probably range in future (observe at several times).
    - Perched water table.
- Bedrock
    - Depth over entire site.
    - Type of rock.
    - Extent and character of weathering.
    - Joints, including distribution, spacing, whether open or closed, and joint infilling.
    - Faults.
    - Solution effects in limestone or other soluble rocks.
    - Core recovery and soundness (RQD).

The reasons for obtaining this minimum data are clear; the engineer must have adequate data to determine the soil type and relative compactness, and the position of the static water level. Methods such as driving open-end rod without obtaining soil samples or water level readings taken after the last soil sample was removed must be discouraged. Good communication between the driller and the foundation engineer is essential during all phases of the subsurface investigation program.

When soft ground is encountered, field (*in situ*) testing and/or undisturbed sample explorations should be done.

4.4.3. Field Boring Procedures

The importance of good logging and field notes cannot be overemphasized. It is most necessary for the logger to realize that a good field description must be recorded. The field-boring log is the major portion of the factual data used in the analysis of foundation conditions.

The log is a record that should contain all of the information obtained from a boring whether or not it may seem important at the time of drilling. It is important to record the maximum amount of accurate information. This record is the "field" boring log, as opposed to the "finished" boring log used in the preparation of the final report made to the designer. The finished log is drawn from the data given in the field log supplemented by the results of lab visual identification of samples and lab classification tests.

The person who actually logs the field information will vary from organization to organization. Some will have an engineering geologist, or trained technician accompany the drill crew, while others may train the drill crew supervisor to log the borehole. In order to obtain the maximum amount of accurate data, the logger should work closely with the driller and consult with him as to changes in materials and operations while drilling.

Generally, the logger should be responsible for recording the following information:

1. General description of each rock and soil stratum, and the depth to the top and bottom of each stratum.
2. The depth at which each sample is taken the type of sample taken, its number, and any loss of samples taken during extraction from the hole.
3. The depths at which field tests are made and the results of the test.
4. Information generally required by the log format, such as:
    a. Boring number and location.
    b. Date of start and finish of the hole.
    c. Name of driller (and of logger, if applicable).
    d. Elevation at top of hole.
    e. Depth of hole and reason for termination.
    f. Diameter of any casing used.
    g. Size of hammer and free fall used on casing (if driven).
    h. Blows per foot to advance casing (if driven).
    i. Description and size of sampler.
    j. Size of drive hammer and free fall used on sampler in dynamic field tests.
    k. Blow count for each 6 inches to drive sampler. (Sampler should be driven three 6" increments or to 100 blows).
    l. Type of drilling machine used.
    m. Type and size of core barrel used.
    n. Length of time to drill each core run or foot of core run.
    o. Length of each core run and amount of core per run.
    p. Recovery of sample in inches and RQD of rock core.
    q. Project identification.
5. Notes regarding any other pertinent information and remarks on miscellaneous conditions encountered, such as:
    a. Depth of observed groundwater, elapsed time from completion of drilling, conditions under which observations were made, and comparison with the elevation noted during reconnaissance (if any).
    b. Artesian water pressure.
    c. Obstructions encountered.
    d. Difficulties in drilling (caving, coring boulders, surging or rise of sands in casing, caverns, etc.).

e. Loss of circulating water and addition of extra drilling water.

f. Drilling mud and casing as needed and why.

g. Odor of recovered sample.

6. Any other information the collection of which may be required by policy.

During progression of a boring, the field drilling personnel should only roughly identify and describe the soils encountered. Unfortunately the drillers are usually delegated the task of exactly identifying and describing the soil samples. This is unfair, as drillers must be concerned with many other tasks involving mechanical operation of the rig and preparation of pertinent data for the subsurface log. In addition, the visual identification test should not be done outdoors in an atmosphere subjected to the elements, as this ingle operation will provide the basis for later testing and soil profile development. Instead, the soil samples should be sent to a laboratory and visually identified by a technician experienced in soils work. This is of great importance where no laboratory testing is to be performed and design values are estimated on the visual description and SPT results.

When additional undisturbed sample borings are taken, the undisturbed samples are sent to a soils laboratory for testing. Drilling personnel should exercise great care in extracting, handling, and transporting these samples to avoid disturbing the natural soil structure. Tubes should only be pressed, not driven with a hammer. The length of press should be 4 to 6 inches less than the tube length (DO NOT OVERPRESS). A one-inch thick plug composed of a mixture of bees wax and paraffin should be poured to seal the tube against moisture loss. The void at the upper tube end should be filled with sawdust and then both ends capped and taped before transport. The most common sources of disturbance are rough, careless handling of the tube (such as dropping the tube samples in the back of a truck and driving 50 miles over a bumpy road), or temperature extremes (leaving the tube sample outside in below zero weather or storing in front of a furnace). Proper storage and transport should be done with the tube upright and encased in an insulated box partially filled with sawdust or Styrofoam to act as a cushion. Each tube should be physically separated from adjacent tubes like bottles in a case. An alternate method to ease transportation and storage problem is to extrude tube in the field. These samples should be carefully sectioned in 6 to 8 inch lengths, wrapped in wax paper and sealed in a cardboard container (such as ice cream cartons) using liquid paraffin.

4.4.4. *Roadway Soil Surveys*

Soil survey explorations are made along the proposed roadway alignment for defining subsurface materials. This information is used in the design of the pavement section, as well as in defining the limits of unsuitable materials and any remedial measures to be taken. Soil survey information is also used in predicting the probable stability of cut or fill slopes.

Minimum criteria for soil surveys vary substantially, depending on the location of the proposed roadway, the anticipated subsurface materials, and the type of roadway. The following are basic guidelines covering general conditions. It is important that the engineer visit the site to ensure that all features are covered. In general, if a structure boring is located in close proximity to a planned soil survey boring, the soil survey boring may be omitted.

a. At least one boring shall be placed at each 100-foot (30 m) interval. Generally, borings are to be staggered left and right of the centerline to cover the entire roadway corridor. Borings may be spaced further apart if pre-existing information indicates the presence of uniform subsurface conditions. Additional borings shall be located as necessary to define the limits of any undesirable materials or to better define soils stratification.

b. In areas of highly variable soil conditions, additional borings shall be located at each interval considering the following criteria.

1. For interstate highways, three borings are to be placed at each interval, one within the median and one within each roadway.
2. For four lane roadways, two borings are to be placed at each interval, one within each roadway.

c. For roadway widenings that provide an additional lane, one boring shall be placed within the additional lane at each interval.

d. In areas of cut or fill, where stability analysis is anticipated, a minimum of two additional borings shall be placed at each interval near the outer reaches of the sloped areas.

e. In all cases, at least three samples per mile (two samples per kilometer) or 3 per project whichever is greater shall be obtained for each stratum encountered. Each of the samples representing a particular stratum shall be obtained from a different location, with sampling locations spread out over each mile (kilometre). Samples should be of adequate size to permit classification and moisture content testing.

f. Additional samples shall be obtained to permit LBR and corrosion testing. As a minimum, three LBR samples per mile (two samples per kilometer) or 3 per project whichever is greater per stratum of all materials shall be obtained and tested. LBR samples shall also be obtained of all strata located in excavation areas (i.e., water retention areas, ditches, cuts, etc.). Corrosion series samples shall be obtained (unless no structures are to be installed) on a frequency of at least one sample per stratum per 1,500 feet (450 m) of alignment. When a rigid pavement is being considered for design, obtain sufficient samples to perform laboratory permeability tests.

g. Borings in areas of little or no grade change shall extend a minimum of 5 feet (1.5 m) below grade, drainage pipe or culvert invert level whichever is deeper. Every 500 feet (150 m), one boring shall be extended to a nominal depth of 20 feet (6 m) below grade. The 20 feet (6 m) borings apply to projects with proposed buried storm sewer systems; project specifics may dictate adjustments. Borings may or may not include Standard Penetration Tests (SPT), depending on the specific project and its location.

h. In areas of cut, borings shall extend a minimum of 10 feet (3 m) below the proposed grade. If poor soil conditions are encountered at this depth, borings shall be extended to firm materials or to a depth below grade equal to the depth of cut, whichever occurs first. Bag, SPT, undisturbed and core samples shall be obtained as appropriate for analyses.

i. In areas of fill, borings shall extend to firm material or to a depth of twice the embankment height. Bag, SPT, and undisturbed samples shall be obtained as appropriate.

j. Areas of muck must be probed to delineate both the vertical and the horizontal extents.

4.4.5. *Structures*

4.4.5.1. General Notes

The purpose of structure borings is to provide sufficient information about the subsurface materials to permit design of the structure foundations and related geotechnical construction. The following general criteria should satisfy this purpose on most projects; however, it is the engineer's responsibility to assure that appropriate explorations are carried out for each specific project.

The procedure for this is outlined as follows, with details for different types of structures given in the following sections:

- Progress the drill holes according to the recommendations given below for different types of structures. The drill holes may be advanced with casing, drilling mud, or continuous flight augers.

- Estimate the boring depth from existing data obtained during the terrain reconnaissance phases or, less preferred, from requested boring resistance data.
- Obtain standard split spoon samples at proper intervals or at changes in material.
- Record the standard penetration test data on each drill hole in accordance with ASTM D-1586. This is the most economical method presently available of procuring useful data regardless of the oft-cited frailties of the test.
- Instruct the drilling crew to perform a rough visual analysis of the soil samples and record all pertinent data on a standard drill log form. The disturbed spoon samples must be carefully sealed in plastic bags, placed in jars, and sent to the laboratory for analysis. Undisturbed tube samples must be sealed and stored upright in a shock proof, insulated container normally constructed from plywood and filled with cushioning material.
- Observe the water level in each boring and record the depth below top of hole and the date of the reading on the drill log for:
    - Water seepage or artesian pressure encountered during drilling. Artesian pressure may be measured by extending drill casing above the ground until flow stops. Report the pressure as the number of feet of head above ground.
    - Water level at the end of each day and at completion of boring.
    - Water level 24 hours (minimum) after hole completion. Long-term readings may require installation of a perforated plastic tube before abandoning the hole.

  A false indication of water level may be obtained when water is used in drilling and adequate time is not permitted after hole completion for the water level to stabilize. In low permeability soils, such as clays, more than one week may be required to obtain accurate readings.
- Designate a unique identification number for each drill hole to prevent duplication during later exploration phases. Much confusion has resulted on projects where exploration phases. Much confusion has resulted on projects where only single numbers did exploration numbering.

4.4.5.2. Bridges

Perform at least one 2.5" (63.5 mm) minimum diameter borehole at each pier or abutment location. The hole pattern should be staggered so that borings occur at the opposite ends of adjacent piers. Pier foundations or abutments over 100 feet (30 m) in plan length may require at least two borings, preferably at the extremities of the proposed substructure. For structure widenings, the total number of borings may be reduced depending on the information available for the existing structure.

1) If pier locations are unknown, their probable approximate locations may be deduced based on experience and a preliminary design concept for the structure. If this is not possible, place borings at no more than 100-foot (30 m) intervals along the alignment. Additionally, for projects which include a water crossing that includes a pier in the water, at least one boring should be located in the water when practical depending on the width of the crossing.
2) Borings shall be continued until all unsuitable foundation materials have been penetrated and the predicted stress from the foundation loading is less than 10% of the original overburden pressure (see Figure 4-2), or until a minimum of 10 feet (3 m) of competent rock has been penetrated. If no data is available for predicting the foundation stress, extend the boring until at least 20 feet (6 m) of bedrock or other competent bearing material (N-values of 50 or greater) is encountered. (Scour and lateral requirements must be taken into account.) Other possible criteria could be: "The borings for structure foundations shall be terminated when a minimum resistance criteria of 100 blows per foot on the

casing and 20 blows per foot on the sample spoon has been achieved for 20 feet of drilling," or "the boring shall extend 10 feet into rock having an average recovery of 50% or greater."

3) When using the Standard Penetration Test, split-spoon samples shall be obtained at a maximum interval of 2.5 to 3.0 feet (one meter) and at the top of each stratum. Continuous SPT sampling in accordance with ASTM D 1586 is recommended in the top 15 to 20 feet (5 to 6 m) unless the material is obviously unacceptable as a founding material. These spoon samples are "disturbed" samples generally not suited for laboratory determination of strength or consolidation parameters. Undisturbed Shelby tube samples should be obtained at 5-foot intervals in at least one boring in cohesive soils. For cohesive deposits greater than 30 feet in depth, tube sample interval can be increased to 10 feet. In soft clay deposits, *in situ* vane shear strength tests are recommended at 5-10 foot intervals.
4) When cohesive soils are encountered, undisturbed samples shall be obtained at 5-foot (1.5 m) intervals in at least one boring. Undisturbed samples shall be obtained from more than one boring where possible.
5) When rock is encountered, successive core runs shall be made with the objective of obtaining the best possible core recovery. **SPT's shall be performed between core runs, typically at 5-foot (1.5 m) intervals**.
6) In-situ vane, pressuremeter, or dilatometer tests are recommended where soft clays are encountered.
7) Corrosion tests are required on all new bridge projects. As a minimum one on the soil and one on the water shall be done.
8) In the case of water crossing, samples of streambed materials and each underlying stratum shall be obtained for determination of the median particle diameter, D_{50}, needed for scour analysis.
9) For projects with large ship impacts, the pressuremeter test is recommended to be performed within seven (7) foundation element diameters below the deepest scour elevation at the pier location.

4.4.5.3. Approach Embankments

1) At least one boring shall be taken at the point of highest fill; usually the borings taken for the bridge abutment will satisfy this purpose. If settlement or stability problems are anticipated, as may occur due to the height of the proposed embankment and/or the presence of poor foundation soils, additional borings shall be taken along the alignment. The first of these borings shall be no more than 15 feet (5 m) from the abutment. The remaining borings shall be placed at 100-foot (30 m) intervals until the height of the fill is less than 5 feet (1.5 m). Borings shall be taken at the toes of the proposed embankment slopes as well as the embankment centerline.
2) Borings shall be continued until the superimposed stress is less than 10% of the original overburden pressure (see Figure 4-3) and unsuitable founding materials have been penetrated.
3) Sampling and in-situ testing criteria are the same as for bridges, above.

4.4.5.4. Retaining Walls

1) At retaining wall locations, borings shall be taken at a maximum interval of one per 150 feet (50 m) of the wall, as close to the wall alignment as possible. Borings shall be extended below the bottom of the wall a minimum of twice the wall height or at least 10 feet (3 m) into competent material. This applies to all walls, proprietary systems as well as precast and cast-in-place.
2) Sampling and in-situ testing criteria are the same as for bridges.

4.4.5.5. Buildings

In general, one boring should be taken at each corner and one in the centre. This may be reduced for small buildings. For extremely large buildings or highly variable site conditions, one boring should be taken at each support location. Other criteria are the same as for bridges.

4.4.5.6. Drainage Structures

1) Borings shall be taken at proposed locations of box culverts. Trenches or hand auger borings may suffice for smaller structures.
2) For box culverts, borings shall extend a minimum of 15 feet (5 m) below the bottom of the culvert or until firm material is encountered, whichever is deeper.
3) For smaller structures, borings or trenches shall extend at least 5 feet (1.5 m) below the bottom of the structure or until firm material is encountered, whichever is deeper.
4) Corrosion testing must be performed for each site. Material from each stratum above the invert elevation and any standing water shall be tested. For drainage systems parallel to roadway alignments, tests shall be performed at 1,500-feet (500 m) intervals along the alignment.

4.4.5.7. High Mast Lighting, Strain Poles and Sign Structures

1) One boring shall be taken at each designated location.
2) Borings shall be 50 feet (15 m) into suitable soil or 5 feet (1.5 m) into competent rock. Deeper borings may be required for cases with higher torsional loads.
3) Other criteria are the same as for bridges.

4.4.5.8. Mast Arms Assemblies

1) One boring (Auger, SPT or CPT) shall be taken in the area of each designated location (for uniform sites a boring can cover several foundation locations).
2) For mast arm assemblies an analysis and design must be done.

4.4.5.9. Tunnels

Due to the greatly varying conditions under which tunnels are constructed, investigation criteria for tunnels shall be established for each project on an individual basis.

4.4.6. *Borrow Areas*

Test pits, trenches, and various types of borings can be used for exploration of potential borrow areas. Samples should be obtained to permit classification, moisture, compaction, permeability test, LBR, and/or corrosion testing of each material type, as applicable. The extent of the exploration will depend on the size of the borrow area and the amount and type of borrow needed.

4.4.7. *Retention Ponds*

A minimum of 2 borings shall be taken per 40,000 feet^2 (4,000 m^2) of pond, with a minimum depth of 5 feet (1.5 m) below the deepest elevation of the pond, or until a confining layer is encountered or local water management criteria are satisfied. A minimum of 2 field permeability tests per pond shall be performed, with this number increasing for larger ponds.

Sufficient testing must be accomplished to verify whether the excavated material can be used for embankment fill. In addition, if rock is to be excavated from the pond sufficient borings and soundings must be accomplished to estimate the volume of rock to be removed and the hardness of the rock.

Figure 4-2 Depth below which the Foundation-Induced Vertical Normal Stress Increase is likely less than 10% of the Effective Overburden Pressure

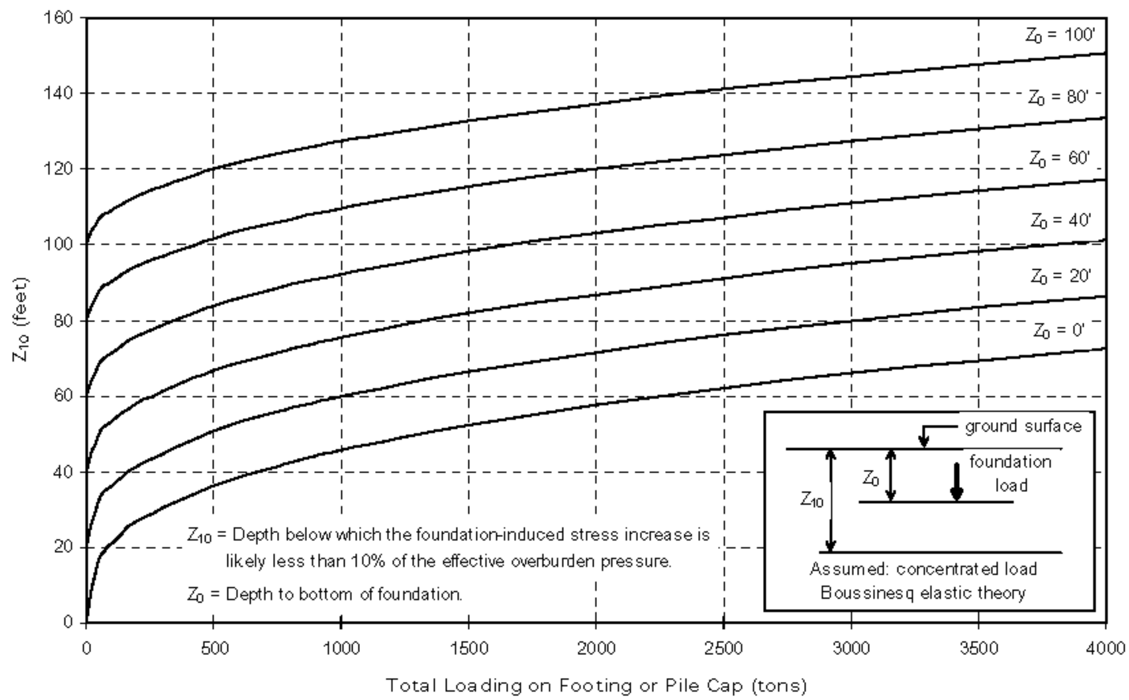
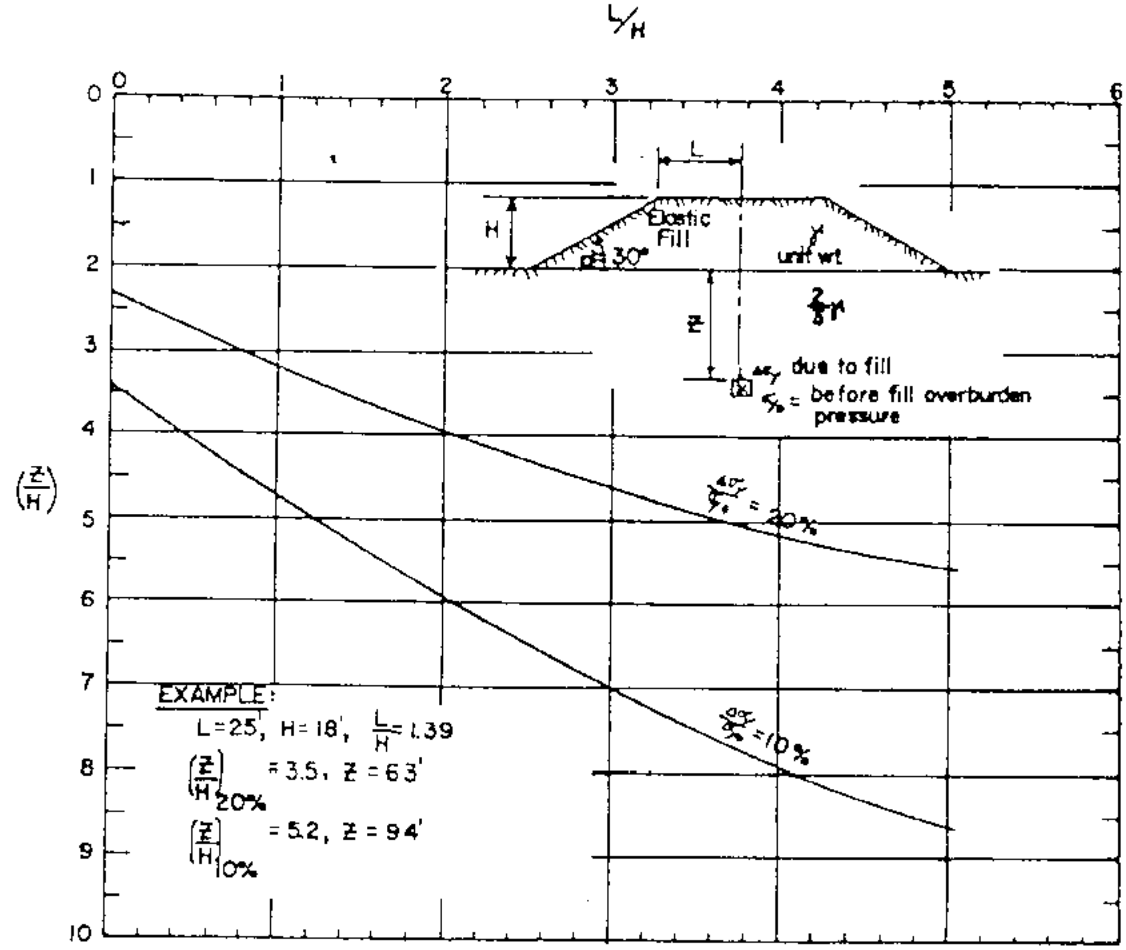


Figure 4-3 Chart for Determining the Maximum Depth of Significant Increase in Vertical Stress in the Foundation Soils Resulting from an Infinitely Long Trapezoidal Fill (both fill and foundation assumed homogeneous, isotropic and elastic)



4.5. Presentation of Geotechnical Information

Upon completion of the subsurface investigation and analysis, the information, which has been obtained, must be compiled in a format, which will present to others the results of the work, which has been performed. This compilation will serve as the permanent record of all geotechnical data known to be pertinent to the project and will be referred to throughout the design, construction, and service life of the project. It is perhaps the most critical function of the geotechnical process. The data is typically compiled in a geotechnical report. The purpose of the geotechnical report is to present the data collected in a clear manner, to draw conclusions from the data and to make recommendations for the geotechnical related portions of the project. The format and contents of the geotechnical report are somewhat dependent on the type of project. Most projects will generally require either a roadway soil survey or a structure related foundation investigation, or both. For reports prepared by consultants the consultant's recommendations shall be documented and retained. The department's final decision shall be documented separately (i.e. in letter form to the structures engineer in charge of the project).

This section describes the format for presentation of geotechnical data for each type of project. General outlines of the topics to be discussed in the geotechnical report are presented. Not every project will follow these formats exactly, however; for any given project, certain items may be unnecessary while other items will need to be added. Also included in this section are discussions on the finalization and distribution of the geotechnical report and on the incorporation of its recommendations into the design.

4.5.1. Roadway Soil Survey

The geotechnical report for a roadway soil survey present conclusions and recommendations concerning the suitability of in-situ materials for use as embankment materials. Special problems affecting roadway design, such as slope stability or excessive settlement may also be discussed if applicable. The following is a general outline of the topics, which should be included.

4.5.1.1. General Information

a. List of information provided to the geotechnical consultant (alignment, foundation layout, 30% plans, scour estimate, etc.).

b. Description of the project, including location, type, and any design assumptions.

c. Description of significant geologic and topographic features of the site.

d. Description of width, composition, and condition of existing roadway.

e. Description of methods used during the subsurface explorations, in-situ testing, and laboratory testing.

f. Soil conservation (SCS/USDA) and USGS maps.

4.5.1.2. Conclusion and Recommendations

a. Explanation of stratification of in-situ materials including ground water table.

b. Evaluation of strength and extent of unsuitable soils within the proposed alignment including their probable effect on roadway performance. The extent of removal of the unsuitable material should be stated. Recommendations for special construction considerations, which minimize anticipated problems should be included.

c. Recommended design LBR based on the most conservative value from either the 90% Method or the + 2% of Optimum LBR Method.

d. Estimated soil drainage characteristics and permeability or infiltration rates. In the case of rigid pavement design, include average laboratory permeability values for each stratum.

e. Recommendations for cut or fill sections when seepage, stability or settlements are significant.

f. Recommendations for any cast-in-place or MSE walls.

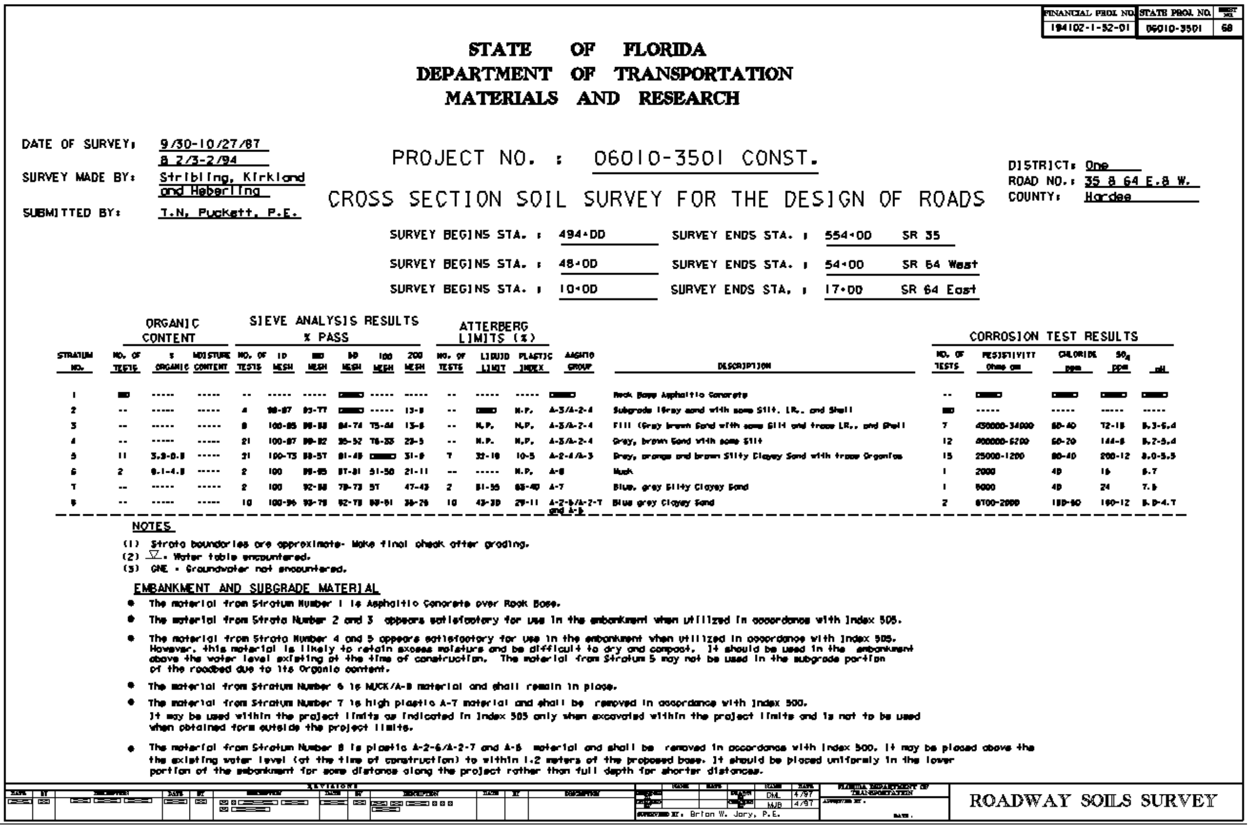
g. Any storm water retention pond considerations.

h. Effect of roadway construction (vibratory rollers, utility excavations, etc.) on surrounding structures and effect on the usage of the structures during roadway construction.

4.5.1.3. Roadway Soils Survey Sheet

This sheet presents a material description and results of classification and corrosivity tests for each stratum. Recommendations for material utilization are provided. Visual classification of muck is not sufficient; organic content test results should be included in the material description. The number of lab tests runs for each stratum shall be included for corrosion tests results as well as classification tests. Include the range of values of all tests performed for each stratum. The report of test results sheet is included in the construction plans. Figure 4-4 is an example of a typical test results sheet.

Figure 4-4 Typical Report of Test Results Sheet



FINANCIAL PROJ. NO.	STATE PROJ. NO.	SHEET NO.
194102-1-52-01	06010-3501	68

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

DATE OF SURVEY: 9/30-10/27/87 & 2/3-2/94
SURVEY MADE BY: Stribling, Kirkland and Heberling
SUBMITTED BY: T.N. Puckett, P.E.

PROJECT NO. : 06010-3501 CONST.

CROSS SECTION SOIL SURVEY FOR THE DESIGN OF ROADS

DISTRICT: One
ROAD NO.: 35 & 64 E.& W.
COUNTY: Hardee

SURVEY BEGINS STA. : 494+00 SURVEY ENDS STA. : 554+00 SR 35
SURVEY BEGINS STA. : 48+00 SURVEY ENDS STA. : 54+00 SR 64 West
SURVEY BEGINS STA. : 10+00 SURVEY ENDS STA. : 17+00 SR 64 East

	ORGANIC CONTENT			SIEVE ANALYSIS RESULTS % PASS						ATTERBERG LIMITS (%)					CORROSION TEST RESULTS				
STRATUM NO.	NO. OF TESTS	% ORGANIC	MOISTURE CONTENT	NO. OF TESTS	10 MESH	40 MESH	60 MESH	100 MESH	200 MESH	NO. OF TESTS	LIQUID LIMIT	PLASTIC INDEX	AASHTO GROUP	DESCRIPTION	NO. OF TESTS	RESISTIVITY Ohms cm	CHLORIDE ppm	SO_4 ppm	pH
1	[illegible]	-----	-----	--	-----	-----	[illegible]	-----	-----	--	-----	-----	[illegible]	Rock Base Asphaltic Concrete	--	[illegible]	[illegible]	[illegible]	[illegible]
2	--	-----	-----	4	98-87	93-77	[illegible]	-----	13-8	--	[illegible]	N.P.	A-3/A-2-4	Subgrade (Gray sand with some Silt, LR., and Shell	[illegible]	-----	-----	-----	-----
3	--	-----	-----	8	100-85	98-88	84-74	75-44	13-6	--	N.P.	N.P.	A-3/A-2-4	Fill (Gray brown Sand with some Silt and trace LR., and Shell	7	450000-34000	80-40	72-18	8.3-5.4
4	--	-----	-----	21	100-87	99-82	95-52	76-33	23-5	--	N.P.	N.P.	A-3/A-2-4	Grey, brown Sand with some Silt	12	400000-6200	60-20	144-8	8.2-5.4
5	11	3.9-0.8	-----	21	100-75	98-57	81-49	[illegible]	31-9	7	32-18	10-5	A-2-4/A-3	Grey, orange and brown Silty Clayey Sand with trace Organics	15	25000-1200	80-40	200-12	8.0-5.5
6	2	8.1-4.8	-----	2	100	99-95	87-81	51-50	21-11	--	-----	N.P.	A-8	Muck	1	2000	40	16	8.7
7	--	-----	-----	2	100	92-88	79-73	57	47-43	2	81-55	65-40	A-7	Blue, grey Silty Clayey Sand	1	8000	40	24	7.6
8	--	-----	-----	10	100-96	93-79	92-78	88-61	38-26	10	43-30	29-11	A-2-6/A-2-7 and A-6	Blue grey Clayey Sand	2	8700-2000	180-80	190-12	8.0-4.7

NOTES
(1) Strata boundaries are approximate- Make final check after grading.
(2) ▽ - Water table encountered.
(3) GNE - Groundwater not encountered.

EMBANKMENT AND SUBGRADE MATERIAL

- The material from Stratum Number 1 is Asphaltic Concrete over Rock Base.
- The material from Strata Number 2 and 3 appears satisfactory for use in the embankment when utilized in accordance with Index 505.
- The material from Strata Number 4 and 5 appears satisfactory for use in the embankment when utilized in accordance with Index 505. However, this material is likely to retain excess moisture and be difficult to dry and compact. It should be used in the embankment above the water level existing at the time of construction. The material from Stratum 5 may not be used in the subgrade portion of the roadbed due to its Organic content.
- The material from Stratum Number 6 is MUCK/A-8 material and shall remain in place.
- The material from Stratum Number 7 is high plastic A-7 material and shall be removed in accordance with Index 500. It may be used within the project limits as indicated in Index 505 only when excavated within the project limits and is not to be used when obtained form outside the project limits.
- The material from Stratum Number 8 is plastic A-2-6/A-2-7 and A-6 material and shall be removed in accordance with Index 500. It may be placed above the the existing water level (at the time of construction) to within 1.2 meters of the proposed base. It should be placed uniformly in the lower portion of the embankment for some distance along the project rather than full depth for shorter distances.

REVISIONS

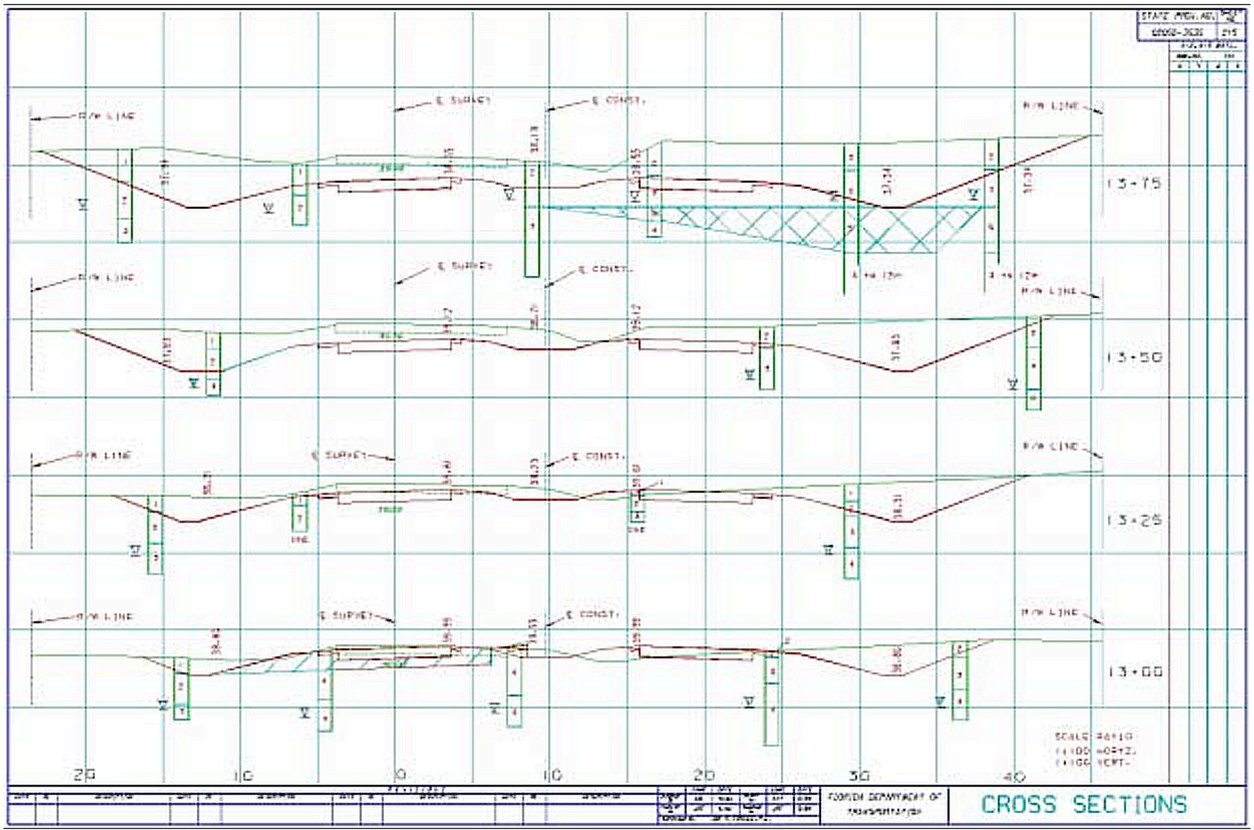
DML 4/97
MJB 4/97
SUPERVISED BY: Brian W. Jory, P.E.
FLORIDA DEPARTMENT OF TRANSPORTATION

ROADWAY SOILS SURVEY

4.5.1.4. Roadway Cross Sections

Simplified boring logs are plotted on the cross section sheets included in the construction plans. Each material stratum is numbered corresponding to the strata on the test results sheet. Figure 4-5 is an example of a typical cross sections sheet. If cross sections sheets are to be prepared by others, the appropriate subsurface information should be provided. The geotechnical engineer should then verify that the data has been correctly incorporated. Removal of unsuitable materials should be indicated on the cross sections.

Figure 4-5 Typical Roadway Cross-Section Sheet



4.5.2. Structures Investigation

4.5.2.1. Introduction

The geotechnical report for a structure presents the conclusions and recommendations for the most suitable foundation types and information required for incorporating such foundations into the design of the structure. Recommendations for related work, such as approach embankments and retaining walls, are also included. Special construction considerations are noted. Only the site-specific items should be recommended for the special provisions. The following is a general guide to the contents of a typical structure foundation report.

4.5.2.2. Scope of Investigation

a. Description of type of project, location of project, and any assumptions related to the project.

b. Vicinity map, including potentiometric map, USGS and soil survey maps (SCS/USDA), depicting project location.

c. Summary of general content of report.

4.5.2.3. Interpretation of Subsurface Conditions

a. Description of the methods used in the field investigation, including the types and frequencies of all in-situ tests.

b. Description of the laboratory-testing phase, including any special test methods employed.

c. Boring location plan and plots of boring logs and cone soundings. Note the size of rock core sampled, and the minimum acceptable rock core diameter to be used shall be 2.4 inch (61 mm) (although 4 inch {101.6 mm} diameter rock cores are preferable). See Figure 4-6 and Figure 4-7 for examples of report of core borings and report of cone soundings sheets. These sheets are included in the final plans. Standard soil type symbols used in plotting the borings are shown in Figure 4-8.

d. Estimated depths of scour used, if applicable.

e. Environmental class for both substructure and superstructure, based on results of corrosivity tests. This information is also reported on the report of core borings sheet. For extremely aggressive classification note what parameter placed it in that category.

f. Summary table of soil parameters determined from field and laboratory testing.

g. Table of soil parameters to use with computer modeling. These parameters can be broken up into zones across the bridge length.

h. MSE or cast-in-place wall recommendations.

Figure 4-6 Typical Report of Core Borings Sheet

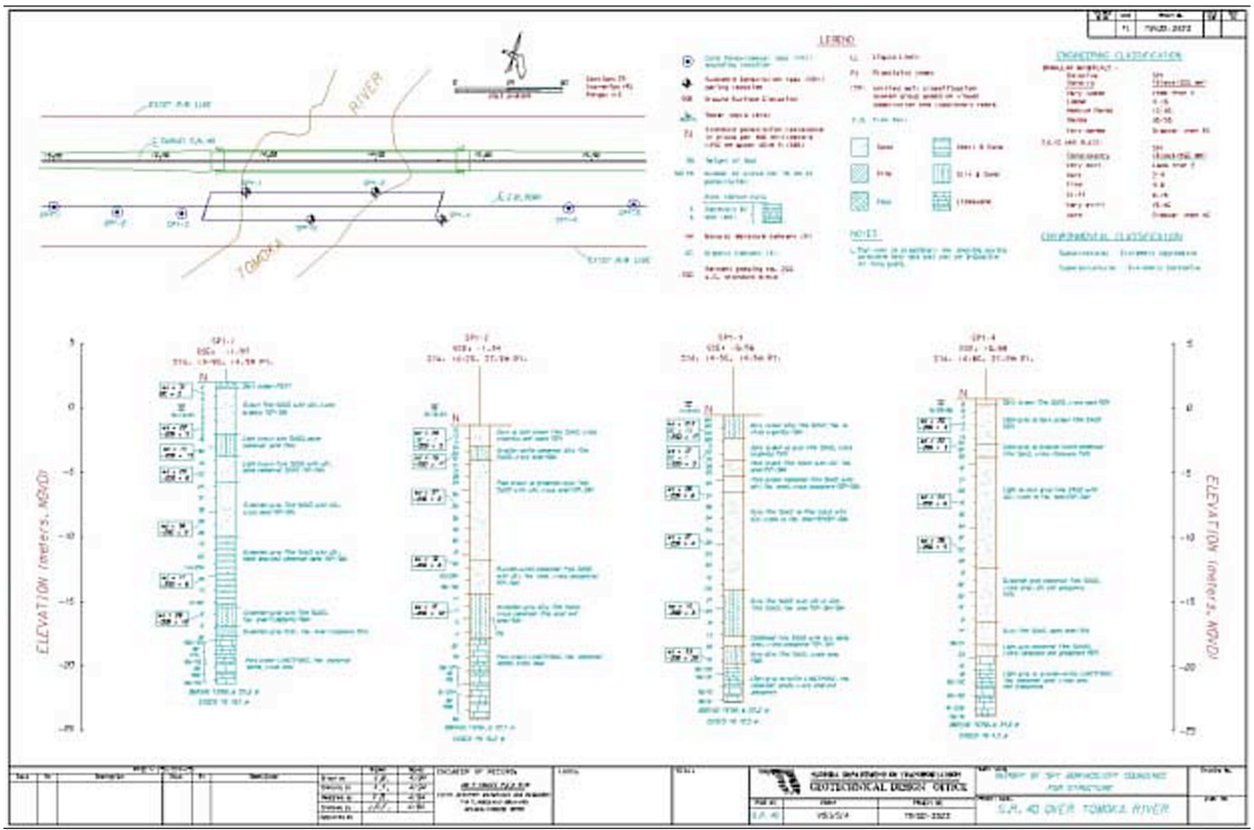


Figure 4-7 Typical Report of Cone Soundings Sheet

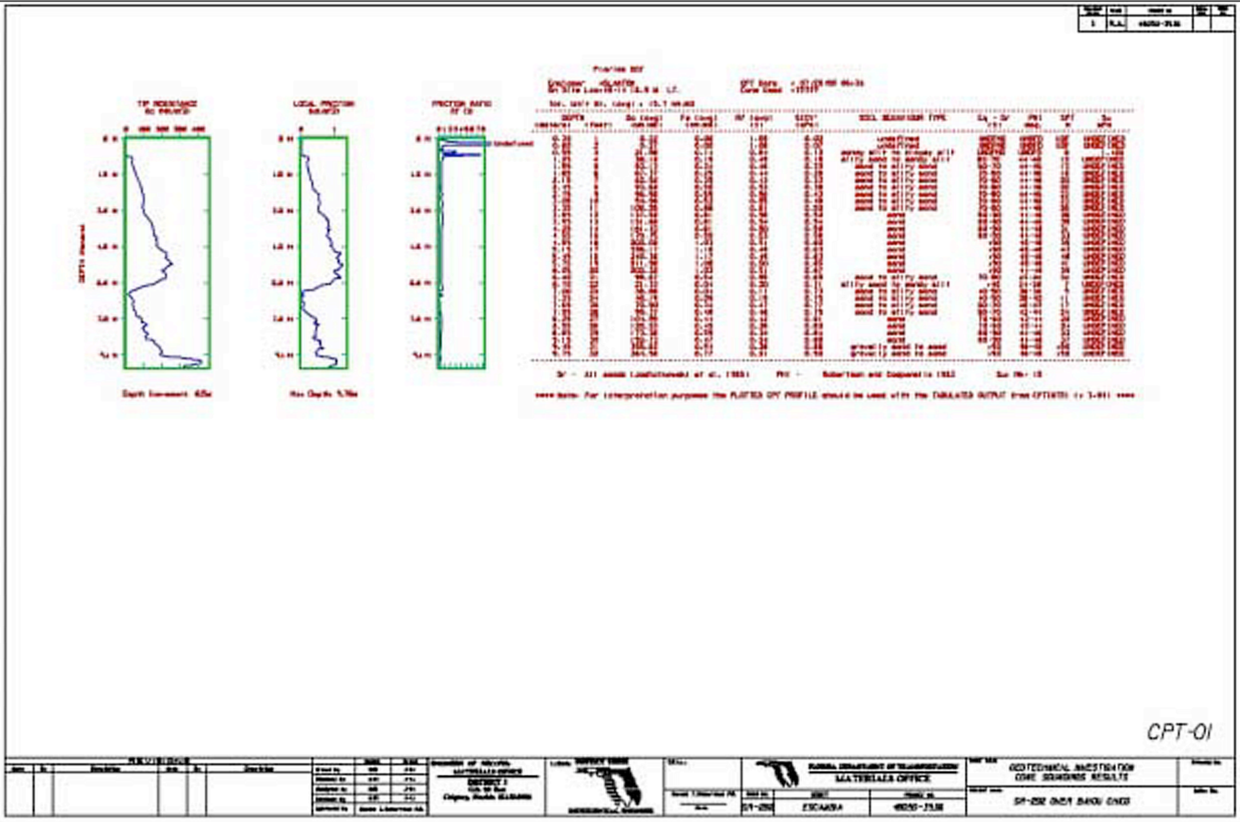
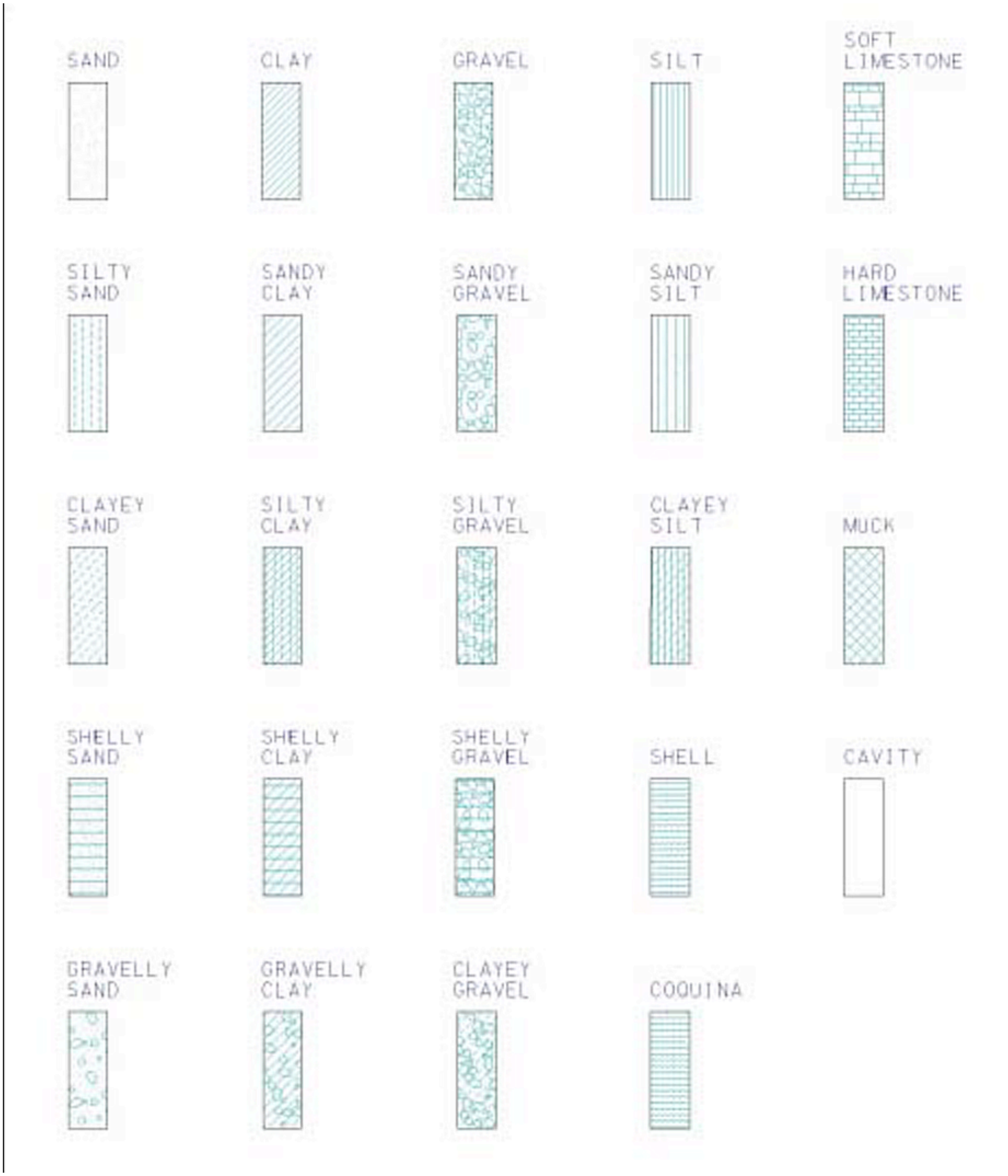


Figure 4-8 Standard Soil Type Symbols



4.5.2.4. Existing Structures Survey and Evaluation

Structures in close proximity to construction activities should be evaluated for potential damages caused by these activities. The usage of the structures should also be included in this evaluation. This needs to happen early in the design process. Vibration, settlement, noise and any other damaging results of these construction activities should be considered in the evaluation. When warranted, the recommendations should include possible means of reducing the damaging effects of the construction activity, such as time restraints on certain operations, underpinning, monitoring, or even purchasing of the property. Table 4-2 shows what is needed in a report. Table 4-3 and the notes that follow are examples of what may be shown on the plan sheets.

Where there is a potential impact on existing structures in the surrounding area, the report should include the structures address, type of construction, the estimated vibration level that may cause damage, the usage (storage building, hospital, etc.), what the potential problem may be and what actions should be taken to minimize the impact.

Table 4-2 Existing Structures Evaluation Table for Geotechnical Report

Address	Structure Type	Potential Vibration Damage Level	Structure Usage	Potential Problem	Recommendation
230 Walnut Street	Concrete	4 in/sec	Storage Units	Damage from vibration	Vibration monitoring during installation of piers 3 – 7.
235 Walnut Street	Brick	1.5 in/sec	House	Damage from vibration	Vibration monitoring during installation of piers 13 – 14.
238 Spruce Ave.	Concrete	2 in/sec	Hotel	Noise	Limit pile drive from 9 am to 7 pm
245 Spruce Ave.	Stucco	0.75 in/sec	House	Vibration causing cracking of stucco	Pre & Post survey, repair any new cracks.

Table 4-3 Plan Note and Table for Existing Structures

Address	Structure Type	Structure Usage	
230 Walnut Street	Concrete	Storage Units	Perform vibration and settlement monitoring during the installation of piers 3-7
235 Walnut Street	Brick	House	Perform vibration and settlement monitoring during the installation of piers 13-14

Typical Notes:

- Noise Restrictions: The contractor shall strictly adhere to all local noise ordinances. All pile driving operations shall be limited to the hours of 7:00 am to 6:00 pm. Methods of maintaining construction noise levels may include but not be limited to temporary noise barriers, enclosures for equipment, mufflers, etc. There will be no separate payment for any of these measures.
- Vibration: The contractor shall provide surveys and settlement/vibration monitoring of the existing structures listed.

4.5.2.5. Structure Foundation Analysis and Recommendations

Alternate foundation recommendations should be provided for all structures including recommendations for spread footings, driven piles, and drilled shafts. An explanation should be included for any of these alternates judged not to be feasible. The types of analyses performed should be summarized.

4.5.2.5.1. Spread Footings

1. Reason(s) for selections and exclusions.
2. Elevation of bottom of footing or depth to competent bearing material.
3. Allowable soil pressure based on settlement and bearing capacity.
4. Settlement potential.
5. Recommendation for special provisions for footing construction, including compaction requirements and the need for particular construction methods such as dewatering or proof rolling.
6. Sinkhole potential.
7. Soil improvement method(s).

4.5.2.5.2. Driven Piles

1. Suitable pile types and reasons for design selections and exclusions.
2. Plots of soil resistance for selected pile size alternates.
3. Recommendations for minimum pile length or bearing elevation (non-lateral).
4. Minimum pile spacing shall be at least three times the diameter/width of the pile size used.
5. Estimated pile settlement or pile group settlement, if significant.
6. Effects of scour, downdrag, and lateral squeeze, if applicable.
7. Maximum driving resistance to be encountered in reaching the estimated bearing elevation. Capacity analyses may have included the estimated scour of substantial amounts of materials. The presence of this material during driving will contribute added resistance to driving.
8. Recommended locations of test piles and pile installation criteria for dynamic monitoring.
9. Selection of load test types, locations and depths where applicable. For static, Statnamic or Osterberg load testing the ultimate load the test should be taken to must be shown in the plans.
10. Recommendations for special provisions for pile installation (special needs or restrictions). Special construction techniques may be needed to minimize the effects of foundation installation.
11. Present recommendations for information to be placed in the pile installation table when applicable.
12. Present soil parameters to be used for lateral analysis accounting for installation techniques and scour. The geotechnical engineer shall check the final lateral load analysis for correct soil property application.

4.5.2.5.3. Drilled Shafts

1. Include plots of soil resistance versus elevation for selected alternate shaft sizes.
2. Recommendations for minimum shaft length or bearing elevation, for shaft diameter, and design soil resistance. The minimum socket length should be indicated, if applicable (non-lateral).
3. Minimum shaft spacing or influence of group effects on capacity.
4. Effects of scour, downdrag, and lateral squeeze, if any.
5. Estimated drilled shaft settlement or shaft group settlement.

6. Recommended locations of test. Selection of load test types, locations and depths. For static, Statnamic or Osterberg load testing the ultimate load the test should be taken to must be shown in the plans.
7. Recommendations for special provisions for shaft installation (special needs or restrictions). Special construction techniques may be needed to minimize the effects of foundation installation.
8. Present recommendations for information to be placed in the drilled shaft installation table.
9. Include the potentiometric surface map information.
10. Present soil parameters to be used for lateral analysis accounting for installation techniques and scour. The geotechnical engineer shall check the final lateral load analysis for correct soil property application.

4.5.2.6. Approach Embankments Considerations

4.5.2.6.1. Settlement

1. Estimated magnitude and rate of settlement.
2. Evaluation of possible alternatives if magnitude or time required for settlement is excessive and recommended treatment based on economic analysis, time and environmental constraints.

4.5.2.6.2. Stability

1. Estimated factor of safety.
2. Evaluation of possible treatment alternatives if factor of safety is too low. Recommended treatment based on economic analysis, time and environmental constraints.

4.5.2.6.3. Construction Considerations

1. Special fill requirements and drainage at abutment walls.
2. Construction monitoring program.
3. Recommendations for special provisions for embankment construction.

4.5.2.7. Retaining Walls and Seawalls

a. Recommended wall type.
b. Recommended lateral earth pressure parameters.
c. Factored soil resistance or alternate foundation recommendations.
d. Settlement potential.
e. Factored soil resistance and loads with respect to sliding and overturning (including standard index wall designs).
f. Overall stability of walls.
g. Recommendations for special provisions for fill material (except MSE walls), drainage.
h. Special considerations for tiebacks, geotextiles, reinforcing materials, etc., if applicable.
i. MSE reinforcement lengths required for external stability, if applicable.

4.5.2.8. Steepened Slopes

a. Estimated factor of safety for internal and external stability.

b. Spacings and lengths of reinforcement to provide a stable slope.

c. Design parameters for reinforcement (allowable strength, durability criteria, and soil-reinforcement interaction).

d. Fill material properties.

e. Special drainage considerations (subsurface and surface water runoff control).

4.5.2.9. Appendix

All structure investigation reports should include an appendix, which contains the following information.

a. Report of Core Boring Sheets. (See Figure 4-6)

b. Report of Cone Sounding Sheet. (See Figure 4-7)

c. Data logs or reports from specialized field tests.

d. Laboratory test data sheets. The following are examples of what should be provided.

   1. Rock Cores: Location, elevation, Maximum Load, Core Length, Core Diameter, Moist Density, Dry Density, Split Tensile Strength, Unconfined Compressive Strength and Strain at Failure.

   2. Gradations: Location, elevation, test results.

   3. Corrosion Tests: Location, elevation, test results.

e. Engineering analyses and notes.

f. FHWA checklist (if applicable).

g. Any other pertinent information.

4.5.3. Final or Supplementary Report

To obtain the optimum benefit from the geotechnical investigation, it is imperative that the geotechnical engineer and the project design and construction engineers interact throughout the duration of the project. The input from the geotechnical engineer should be incorporated into the project as it develops. Often, the geotechnical report, which is initially prepared, is considered preliminary. As the design of the project progresses, the geotechnical recommendations may have to be modified. When the project approaches the final design stage, the geotechnical engineer should prepare a final or supplementary report to revise his assumptions and recommendations if necessary in accordance with the final design plans. The following topics should be included in this report.

1. Final recommended foundation type and alternates.
2. Size and bearing elevation of footing or size, length, and number of piles or drilled shafts at each structural foundation unit.
3. Final factored design loads.
4. Requirements for construction control for foundation installation.
5. Possible construction problems, such as adjacent structures, and recommended solutions.

If revisions to the preliminary report are not necessary, a letter should be submitted stating that the initial report is final.

4.5.4. Signing and Sealing

Geotechnical documents shall be signed and sealed by a professional engineer in responsible charge in accordance with local professional engineering laws and regulations. The following documents are included:

Table 4-4 Signing and Sealing Placement

Placement of Signature/Seal Geotechnical Report	First page of official copy
Supplemental Specifications and Special Provisions	First page of official copy
Roadway Soils Survey Sheet	Title Block
Report of Core Borings Sheet	Title Block
Report of Cone Soundings Sheet	Title Block

For supplemental specifications and special provisions, which cover other topics in addition to geotechnical engineering, the engineer in responsible charge of the geotechnical portions should indicate the applicable pages.

Originals of the sheets for plans shall be signed and dated by the responsible engineer within the space designated "Approved By". One record set of full size prints shall be signed, sealed, and dated.

4.5.5. Plan and Specification Review

In addition to writing a report, the geotechnical engineer should review all phases of the plans and specifications to ensure that the geotechnical recommendations have been correctly incorporated.

4.5.6. Unwanted

Some of the things undesirable in a report are:

1. Do not include specifications in the report. The specification can be referenced by number or section number but do not put the specification in the report.
2. Changes to the specification without valid justification.
3. Long verbal descriptions when a simple table will do it better.

4.6. Geophysical Methods

Geophysical testing is often used as part of the initial site exploration phase of a project and/or to provide supplementary information collected by widely-spaced observations (i.e., borings, test pits, outcrops etc.). Geophysical testing can be used for establishing stratification of subsurface materials, the profile of the top of bedrock, depth to groundwater, limits of types of soil deposits, rippability of hard soil and rock, and the presence of voids, buried pipes, and depths of existing foundations. Data from geophysical testing should always be correlated with information from direct methods of exploration.

Geophysical testing offers some notable advantages and some disadvantages that should be considered before the technique is recommended for a specific application. The advantages are summarized as follows:

- Many geophysical tests are non-invasive and thus offer significant benefits in cases where conventional drilling, testing, and sampling are difficult (e.g., deposits of gravel, talus deposits) or where potentially contaminated soils may occur in the subsurface.
- In general, geophysical testing covers a relatively large area, thus providing the opportunity to characterize large areas with few tests. It is particularly well-suited to projects that have large longitudinal extent compared to lateral extent (such as for new highway construction).
- Geophysical measurement assesses the characteristics of soil and rock at very small strains, typically on the order of 0.001 percent thus providing information on truly elastic properties.
- For the purpose of obtaining information on the subsurface, geophysical methods are relatively inexpensive when considering cost relative to the relatively large areas over which information can be obtained.

Some of the general disadvantages of geophysical methods include:

- Most methods work best for situations in which there is a large difference in stiffness between adjacent subsurface units.
- It is difficult to develop good stratigraphic profiling if the general stratigraphy consists of hard material over soft material.
- Results are generally interpreted qualitatively and therefore only an experienced engineer or geologist familiar with the particular testing method can obtain useful results.
- Specialized equipment is required (compared to more conventional subsurface exploration tools).

There are a number of different geophysical in-situ tests that can be used for stratigraphic information and in the determination of engineering properties. Table 4-5, Table 4-6 and Table 4-7 provide a summary of the various geophysical methods that are currently available in U.S. practice.

Table 4-5 Seismic and Electrical Methods of Geophysical Testing

Method	Basic Field Procedures	Applications	Limitations
SEISMIC METHODS			
Seismic Refraction	Impact load is applied to the ground surface. Seismic energy refracts off soil/rock layer interfaces and is recorded on the ground surface using several dozen geophones positioned along a line or performing repeated events using a single geophone.	• depth to bedrock • depth to water table • thickness and relative stiffness soil/rock layers	• does not work if stiffness decreases with depth or if soft layer underlies stiff layer • works best when sharp stiffness discontinuity is present
Spectral-Analysis-of-Surface-Waves (SASW)	Impact load is applied to the ground surface. Surface waves propagate along ground surface and are recorded on the ground surface with two geophones positioned along a line.	• depth to bedrock • measurement of shear wave velocity • thickness and stiffness of surface pavement layer • qualitative indicator of cracking in pavement	• resolution decreases significantly with increasing depth • accurate interpretation may require a significant amount of expertise • interpretation is difficult if a stiff layer overlies a soft layer and soft layer properties are desired
ELECTRICAL METHODS			
DC Resistivity	DC current is applied to the ground using electrodes. Voltages are measured at different points on the ground surface with other electrodes positioned along a line.	• depth to water table • inorganic groundwater contamination • groundwater salinity • soil layer thickness • delineation of certain vertical features (e.g., sinkholes, contamination plumes, waste trenches)	• slow; must install electrodes directly in the ground • resolution decreases significantly with increasing depth • resolution is difficult in highly heterogeneous deposits
Electromagnetics	Electrical coils are held over the ground. Current passing through the coils induces a magnetic field in the ground, which is measured with receiver coils.	• groundwater salinity • inorganic groundwater contamination • detection of buried metal objects • delineation of certain vertical features (e.g., sinkholes, contamination plumes, waste trenches)	• extra effort is required to characterize depth of target • resolution decreases significantly with increasing depth
Ground Penetrating Radar (GPR)	Electromagnetic energy is pulsed into the ground. This energy reflects off boundaries between different soil layers and is measured at the ground surface.	• depth to water table • identification of buried objects • thickness of pavement layers • void detection	• not effective below the water table or in clay • depth of penetration is limited to about 10 meters

Table 4-6 Gravity, Magnetic and Near-Surface Nuclear Methods of Geophysical Testing

Method	Basic Field Procedures	Applications	Limitations
GRAVITY AND MAGNETIC METHODS			
Gravity	The Earth's gravitational field is measured at the ground surface.	• identification of subsurface voids • identification of large objects possessing unusually high or low densities	• results are non-unique (i.e. more than one subsurface condition can give the same results) • primarily, large-scale reconnaissance tool; applications in engineering are limited
Magnetics	The Earth's magnetic field is measured at the ground surface.	• identification of ferrous materials • identification of soil/rock containing large amounts of magnetic minerals	• results are non-unique (i.e. more than one subsurface condition can give the same results) • primarily a large-scale reconnaissance tool; applications in engineering are limited
NEAR-SURFACE NUCLEAR METHODS			
Neutron Moisture Content	Instrument is placed on the ground surface and neutrons are emitted into the ground. Energy of returning neutrons is related to the moisture content in the ground (hydrogen atoms decrease the energy of the neutrons detected at the sensor).	• estimate of water content in compacted soil • estimate of asphalt content in asphalt concrete • can be quantitative if properly calibrated to site conditions	• limited investigation depth (a few inches) • possible health and safety hazard if operators not properly trained • will detect hydrogen ion (i.e. gas, clay) in non-water bearing stratum
Gamma Density	Instrument is placed on the ground surface and gamma radiation is emitted into the ground. Returning gamma energy is a function of material density (denser materials absorb more gamma energy so less is detected at the sensor)	• estimate of density of soil or asphalt concrete	• limited investigation depth (less than one foot); • investigation depth further limited to a few inches if ground cannot be penetrated • possible health and safety hazard if operators not properly trained

Table 4-7 Borehole Methods of Geophysical Testing

Method	Basic Field Procedures	Applications	Limitations
BOREHOLE METHODS			
Crosshole/ Downhole	Energy sources and geophones are placed in boreholes and/or on ground surface; interval travel times are converted into seismic wave velocity as a function of depth in the borehole	• measurement of wave velocities for seismic site response analysis • depth to water table • correlation of lithologic units with surface seismic • identification of thin layers at depth	• requires one or more boreholes and significant support field equipment
Suspension Logger	Field instrument is placed in a fluid-filled borehole and used to measure P- and S-wave velocities in surrounding soil or rock.	• measurement of wave velocities for seismic site response analysis • correlation of lithologic units with surface seismic • identification of thin layers at depth	• requires borehole and significant support field equipment, which is expensive • borehole must be fluid-filled
Electrical Logging	Field instrument is placed in a borehole. Electrical fields are directly applied or electromagnetically induced into surrounding soil or rock and electrical resistivity is measured.	• estimate of soil/rock permeability or porosity • identification of inorganic contaminant plumes or saltwater intrusion • identification of thin layers at depth	• requires borehole and significant support field equipment, which is expensive • generally cannot operate in a cased borehole • may require fluid-filled borehole • results may be dependent upon drilling mud salinity
Nuclear Logging	Field instrument is placed in a borehole. Surrounding soil or rock is irradiated with neutrons particles and/or gamma energy. Energy and neutrons returning to the instrument are measured and related to rock density, porosity and pore fluid type.	• estimate of soil/rock type, density, porosity, and pore fluid density • identification of thin layers at depth	• requires borehole and significant support field equipment, which is expensive • possible health and safety hazard if operators are not properly trained
Lithology Logging	Field instrument is placed in a borehole; naturally occurring electrical fields and radiation levels are related to soil or rock type	• classification of soil or rock type • identification of thin layers at depth	• requires borehole and significant support field equipment, which is expensive • may require fluid-filled borehole • results are dependent upon site-specific conditions and/or borehole fluid salinity

The following comments apply to highway engineering.

- Seismic Methods: These methods are becoming increasingly popular for highway and general geotechnical engineering practice, as they have the potential to provide quantitative data regarding the shear wave velocity of the subsurface materials. The shear wave velocity is directly related to small-strain material stiffness, which in turn, is often correlated to strength and soil/rock type. As such, these techniques are often used for assessing the vertical stiffness profile in a soil deposit and for assessing the interface between soil and rock.
- Electrical Methods: These methods are usually used when attempting to locate voids or locally distinct materials. With regards to highway applications, these procedures may be applicable for

assessing the potential for karst activity along a potential transmission corridor, or for locating specific underground drums and/or voids. The techniques provide qualitative information only and are usually part of a two- or three-phased investigation program.

- Gravity and Magnetic Methods: These methods are similar to the previously described electrical methods, except that they rely on the correlations between the influence of voids and subsurface anomalies and differences in the earth's micro-gravitational field and/or the magnetic fields, rather than the changes in the electrical fields.
- Near-surface Nuclear Methods: These techniques have been used for several years in the field of soil construction. Through careful calibration, it is possible to reliably assess the moisture content and density of compacted soils. These techniques have gained widespread adoption as reliable quantitative techniques.
- Borehole Methods: Downhole geophysical techniques have been recognized as providing reliable indications of a wide range of soil properties. The downhole/crosshole techniques have proven to provide reliable measure of shear wave velocity. As reported previously, this parameter is directly related to small-strain stiffness and is correlated to strength and soil type. The downhole logging techniques have seen little use in highway construction, but they have been the mainstay for deep geologic characterization in oil exploration and deep geologic characterization. The principal advantage is the ability to obtain several different geophysical tests/ indicators by "stringing" these tools together in a deep boring.

With specific regards to highway construction, a few typical examples where geophysical testing could be used to compliment conventional exploration are as follows:

- Highly Variable Subsurface Conditions: In several geologic settings, the subsurface conditions along a transportation corridor may be expected to be variable. This variability could be from underlying karst development above limestone, alluvial deposits, including buried terrace gravels, across a wide floodplain, buried boulders in a talus slope. For these cases, conventional exploration techniques may be very difficult and if "refusal" is encountered at one depth, there is a strong likelihood that different materials underlie the region. In these cases, a preliminary subsurface characterization profile using geophysical testing could prove advantageous in designing future focused investigations.
- Regional Studies: Along a transmission corridor it may be necessary to assess the depth to (and through) rippable rock. Alternative alignments may or may not be possible, but the cost implications may be significant. Therefore, it is important to obtain a profile related to rock/soil stiffness. Geophysical testing is a logical consideration for this application, as a precursor to invasive investigation.
- Settlement Sensitive Structures: In the case where a settlement-sensitive structure is to be founded on deposits of sands, the in situ modulus of the sand deposit is critical. After assessing the characteristics of the site, it may be helpful to quantify the deformation modulus via geophysical testing at the specific foundation site.

These examples demonstrate that geophysical testing has a potentially important role in the subsurface characterization of soils and rocks. Like the other "tools" described in this document, the particular selection of the appropriate technology is very much a function of the site conditions and the goals of the characterization program. In this document, focus is placed on geophysical testing techniques that can be used to measure soil shear wave velocity, V_s, such as seismic refraction, SASW, and seismic cone penetrometer.

4.7. Soil Borings and Test Pits

4.7.1. Soil Borings

Soil borings are probably the most common method of subsurface exploration in the field.

4.7.1.1. Auger Borings

Rotating an auger while simultaneously advancing it into the ground either hydraulically or mechanically advances auger borings. The auger is advanced to the desired depth and then withdrawn. Samples of cuttings can be removed from the auger; however, the depth of the sample can only be approximated. These samples are disturbed and should be used only for material identification. This method is used to establish soil strata and water table elevations, or to advance to the desired stratum before Standard Penetration Testing (SPT) or undisturbed sampling is performed. However, it cannot be used effectively in soft or loose soils below the water table without casing or drilling mud to hold the hole open. See ASTM D 1452 (AASHTO T 203).

4.7.1.2. Hollow-Stem Auger Borings

A hollow-stem auger consists of a continuous flight auger surrounding a hollow drill stem. The hollow-stem auger is advanced similar to other augers; however, removal of the hollow stem auger is not necessary for sampling. SPT and undisturbed samples are obtained through the hollow drill stem, which acts like a casing to hold the hole open. This increases usage of hollow-stem augers in soft and loose soils. See ASTM D 6151 (AASHTO T 251).

4.7.1.3. Wash Borings

In this method, the boring is advanced by a combination of the chopping action of a light bit and the jetting action of water flowing through the bit. This method of advancing the borehole is used only when precise soil information is not required between sample intervals.

4.7.1.4. Percussion Drilling

In this method, the drill bit advances by power chopping with a limited amount of water in the borehole. Slurry must be periodically removed. The method is not recommended for general exploration because of the difficulty in determining stratum changes and in obtaining undisturbed samples. However, it is useful in penetrating materials not easily penetrated by other methods, such as those containing boulders.

4.7.1.5. Rotary Drilling

A downward pressure applied during rapid rotation advances hollow drill rods with a cutting bit attached to the bottom. The drill bit cuts the material and drilling fluid washes the cuttings from the borehole. This is, in most cases, the fastest method of advancing the borehole and can be used in any type of soil except those containing considerable amounts of large gravel or boulders. Drilling mud or casing can be used to keep the borehole open in soft or loose soils, although the former makes identifying strata change by examining the cuttings difficult.

4.7.1.6. Sealing Boreholes

Upon completion, unless they are used for continuing water level observations, all borings should be backfilled in accordance with applicable local environmental regulations. In many cases this will require grouting. In boreholes for groundwater observations, place casing in tight contact with walls of holes, or fill annular space with sand/gravel.

- Cavernous Limestone. In limestone areas suspected of containing solution channels or cavities, each column location should be investigated. For smaller structures, locate boring or probe at each planned column location. For large structures and area investigation use indirect methods noted below, followed by borings or probes in final column locations, and on close centers (25 ft. under walls or heavily loaded areas). Aerial photographs have been used effectively by experienced geologists for detecting sinkholes and the progress of cavity development by comparing old to new photographs. Geophysical methods are used to detect anomalies in subsurface resistivity, gravity, magnetic field or seismic velocities and to correlate such anomalies with cavity presence[88].

4.7.2. Test Pits

These are the simplest methods of inspecting subsurface soils. Test pits are used to examine and sample soils *in situ*, to determine the depth to groundwater, and to determine the thickness of topsoil. They consist of excavations performed by hand, backhoe, or dozer, and range from shallow manual or machine excavations to deep, sheeted, and braced pits. Hand excavations are often performed with posthole diggers or hand augers. They offer the advantages of speed and ready access for sampling. They are severely hampered by limitations of depth and by the fact they cannot be used in soft or loose soils or below the water table. See Table 4-8 for types, uses, and limitations of test pits and trenches. Hand-cut samples are frequently necessary for highly sensitive, cohesive soils, brittle and weathered rock, and soil formation with honeycomb structure.

4.7.3. Test Trenches

Test trenches are particularly useful for exploration in very heterogeneous deposits such as rubble fills, where borings are either meaningless or not feasible. They are also useful for detection of fault traces in seismicity investigations.

[88] Higginbottom, I.E., The Use of Geophysical Methods in Engineering Geology, Part II, Electrical Resistivity, Magnetic and Gravity Methods, Ground Engineering, Vol. 9, No. 2, 1976, and Millet, R.A. and Morehouse, D.C., Bedrock Verification Program for Davis-Besse Nuclear Power Station, Proceedings of the Specialty Conference on Structural Design of Nuclear Plant Facilities, ASCE, 1973.

Table 4-8 Use, Capabilities and Limitations of Test Pits and Trenches

Exploration Method	General Use	Capabilities	Limitations
Hand-Excavated Test Pits and Shafts	Bulk sampling, in situ testing, visual inspection.	Provides data in inaccessible areas, less mechanical disturbance of surrounding ground.	Expensive, time-consuming, limited to depths above groundwater level.
Backhoe Excavated Test Pits and Trenches	Bulk sampling, in situ testing, visual inspection, excavation rates, depth of bedrock and groundwater.	Fast, economical, generally less than 15 feet deep, can be up to 30 feet deep.	Equipment access, generally limited to depths above groundwater level, limited undisturbed sampling.
Drilled Shafts	Pre-excavation for piles and shafts, landslide investigations, drainage wells.	Fast, more economical than hand excavated, min. 30 inches dia., max. 6 feet dia.	Equipment access, difficult to obtain undisturbed samples, casing obscures visual inspection.
Dozer Cuts	Bedrock characteristics depth of bedrock and groundwater level, rippability, increase depth capability of backhoes, level area for other exploration equipment.	Relatively low cost, exposures for geologic mapping.	Exploration limited to depth above groundwater level.
Trenches for Fault Investigations	Evaluation of presence and activity of faulting and sometimes landslide features.	Definitive location of faulting, subsurface observation up to 30 feet.	Costly, time-consuming, requires shoring, only useful where dateable materials are present, depth limited to zone above groundwater level.

4.8. Sampling

4.8.1. Disturbed and Undisturbed Sampling

- Disturbed samples are primarily used for classification tests and must contain all of the constituents of the soil even though the structure is disturbed.
- Undisturbed samples are taken primarily for laboratory strength and compressibility tests and in those cases where the in-place properties of the soil must be studied.
- Many offshore samplers fall in a special category and are treated separately.

4.8.2. Types of Soil Sampling

Common methods of sampling during field explorations include those listed below. All samples should be properly preserved and carefully transported to the laboratory such that sample integrity is maintained. See ASTM D 4220.

4.8.2.1. Bag Bulk Samples

These are disturbed samples obtained from auger cuttings or test pits. The quantity of the sample depends on the type of testing to be performed, but can range up to 50 lb (25 kg) or more. Testing performed on these samples includes classification, moisture-density, Limerock Bearing Ratio (LBR), and corrosivity tests. A portion of each sample should be placed in a sealed container for moisture content determination.

4.8.2.2. Split-Barrel

Also known as a split-spoon sample, this method is used in conjunction with the Standard Penetration Test. The sampler is a 2" (50.8 mm) (O.D.) split barrel which is driven into the soil with a 140-pound (63.5 kg) hammer dropped 30 inches (760 mm). After it has been driven 18 inches (450 mm), it is withdrawn and the sample removed. The sample should be immediately examined, logged and placed in sample jar for storage. These are disturbed samples and are not suitable for strength or consolidation testing. They are adequate for moisture content, gradation, and Atterberg Limits tests, and valuable for visual identification. See ASTM D 1586.

4.8.2.3. Shelby Tube

This is thin-walled steel tube, usually 3 inches (76.2 mm) (O.D.) by 30 inches (910 mm) in length. It is pushed into the soil with a relatively rapid, smooth stroke and then retracted. This produces a relatively undisturbed sample provided the Shelby tube ends are sealed immediately upon withdrawal. Refer to ASTM D 1587 (AASHTO T 207). This sample is suitable for strength and consolidation tests. This sampling method is unsuitable for hard materials. Good samples must have sufficient cohesion to remain in the tube during withdrawal. Refer to ASTM D 1587 (AASHTO T 207).

4.8.2.4. Piston Samplers

4.8.2.4.1. Stationary

This sampler has the same standard dimensions as the Shelby Tube, above. A piston is positioned at the bottom of the thin-wall tube while the sampler is lowered to the bottom of the hole, thus preventing disturbed materials from entering the tube. The piston is locked in place on top of the soil to be sampled. A sample is obtained by pressing the tube into the soil with a continuous, steady thrust. The stationary piston is held fixed on top of the soil while the sampling tube is advanced. This creates suction while the sampling tube is retrieved thus aiding in retention of the sample. This sampler is suitable for soft to firm clays and silts. Samples are generally less disturbed and have a better recovery ratio than those from the Shelby Tube method.

4.8.2.4.2. Floating

This sampler is similar to the stationary method above, except that the piston is not fixed in position but is free to ride on the top of the sample. The soils being sampled must have adequate strength to cause the piston to remain at a fixed depth as the sampling tube is pushed downward. If the soil is too weak, the piston will tend to move downward with the tube and a sample will not be obtained. This method should therefore be limited to stiff or hard cohesive materials.

4.8.2.4.3. Retractable

This sampler is similar to the stationary sampler, however, after lowering the sampler into position the piston is retracted and locked in place at the top of the sampling tube. A sample is then obtained by pushing the entire assembly downward. This sampler is used for loose or soft soils.

4.8.2.4.4. Hydraulic (Osterberg)

In this sampler, a movable piston is attached to the top of a thin-wall tube. Sampling is accomplished as hydraulic pressure pushes the movable piston downward until it contacts a stationary piston positioned at the top of the soil sample. The distance over which the sampler is pushed is fixed; it cannot be over-pushed. This sampler is used for very soft to firm cohesive soils.

4.8.3. General Requirements for Sampling Program

The number and type of samples to be taken depend on the stratification and material encountered.

- Representative Disturbed Samples. Take representative disturbed samples at vertical intervals of no less than 5 feet and at every change in strata. Recommended procedures for obtaining disturbed samples are contained in ASTM Standard D1586, Penetration Test and Split Barrel Sampling of Soils.
- Undisturbed Samples. The number and spacing of undisturbed samples depend on the anticipated design problems and the necessary testing program. Undisturbed samples should comply with the following criteria:
  - They should contain no visible distortion of strata, or opening or softening of materials;
  - Specific recovery ratio (length of undisturbed sample recovered divided by length of sampling push) should exceed 95%; and
  - They should be taken with a sampler with an area ratio (annular cross-sectional area of sampling tube divided by full area of outside diameter of sampler) less than 15%.
- Obtain undisturbed samples in cohesive soil strata, so that there is at least one representative sample in each boring for each 10 feet depth. Recommended procedures for obtaining undisturbed samples are described in ASTM Standard D1587, Thin-Walled Tube Sampling of Soils. Additional cautions include the following:
  - Caving. Use casing or viscous drilling fluid to advance borehole if there is danger of caving. If groundwater measurements are planned, drilling fluid should be of the revert type.
  - Above Groundwater Table. When sampling above groundwater table, maintain borehole dry whenever possible.
  - Below Groundwater Table. When sampling below groundwater table, maintain borehole full of water or drilling fluid during cleanout, during sampling and sample withdrawal, and while removing cleanout tools. Where continuous samples are required, casing should remain full for the entire drilling and sampling operation.
  - Soft or Loose Soil. Sampling of a soft or loose soil directly below a stiff or compact soil in the same tube should be avoided. Discontinue driving of sample tube when a sudden decrease in resistance occurs.

4.8.4. Undisturbed Samples from Test Pits

Hand trimmed samples may be obtained in test pits, in test trenches, or in surface exposures. Samples so obtained are potentially the least disturbed of all types of samples. The basic procedure consists of trimming out a column of soil the same size or slightly smaller than the container to be used in transportation, sliding the container over the sample, and surrounding the samples with wax. Tight, stiff containers that can be sealed, and are not readily distorted, should be used.

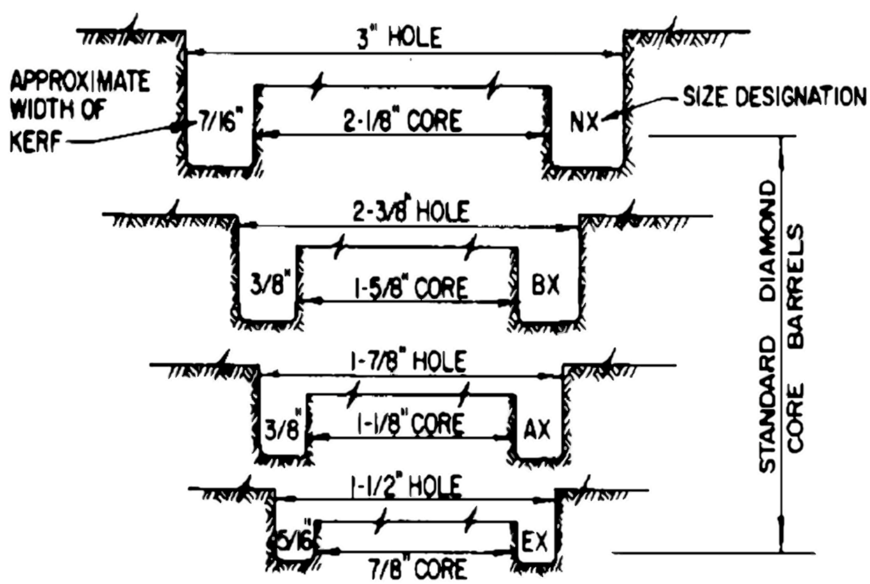
4.8.5. Rock Cores

Rock cores are obtained using core barrels equipped with diamond or tungsten carbide tipped bits. A core barrel is advanced through rock by the application of downward pressure during rotation. Circulating water removes ground-up material from the hole while also cooling the bit. The rate of advance is controlled so as to obtain the maximum possible core recovery. The minimum core barrel to be used shall be HW (2.4 inch {61 mm} diameter), but it is preferable to use a 4" (101.6 mm) diameter core barrel. Refer to ASTM D 2113 (AASHTO T 225).

There are three basic types of core barrels: Single tube, double tube, and triple tube. Single tube core barrels generally provide poor recovery rates in some types of rock. Double tube and triple tube are better in many circumstances and are described below. See also ASTM D 2113 (AASHTO T 225). Refer to ASTM D 5079 for practices of preserving and transporting rock core samples. Rock is sampled with core barrels having either tungsten carbide or diamond core bits as listed or described below or Figure 4-9.

Figure 4-9 Standard Sizes, in inches, for Casings, Rods, Core Barrels, and Holes

Size Symbol							
Casing, Core Barrel	Drill Rod	Casing OD	Casing Bit OD	Core Barrel Bit OD	Drill Rod OD	Approx. Diameter of Core Hole	Approx. Diameter of Core
EX	E	1 13/16	1 27/32	1 7/16	1 5/16	1 1/2	7/8
AX	A	2 1/4	2 5/16	1 27/32	1 5/8	1 7/8	1 1/8
BX	B	2 7/8	2/15/16	2 5/16	1 29/16	2 3/8	1 5/8
NX	N	3 1/2	3 9/16	2 15/16	2 3/8	3	2 1/8



STANDARDS BY NATIONAL BUREAU OF STANDARDS DIAMOND CORE DRILL MANUFACTURERS.

The suitability of cores for structural property tests depends on the quality of individual samples. Specify double or triple tube core barrel for maximum core recovery in weathered, soft, or fractured rock. The percentage of core recovery is an indication of soundness and degree of weathering of rock. Carefully examine core section for reasons for low recovery.

4.8.5.1.1. Double Tube Core Barrel

This core barrel consists of inner and outer tubes equipped with a diamond or tungsten-carbide drill bit. As coring progresses, fluid is introduced downward between the inner and outer tubes to cool the bit and to wash ground-up material to the surface. The inner tube protects the core from the highly erosive action of the drilling fluid. In a rigid type core barrel, both the inner and outer tubes rotate. In a swivel type, the inner tube remains stationary while the outer tube rotates. Several series of swivel type core barrels are available. Barrel sizes vary from EWG or EWM (0.845 inch (21.5 mm) to 6 inch (152.4 mm) I.D.). The larger diameter barrels are used in highly erodible materials, such as Florida limestone, to generally obtain better core recovery. The minimum core barrel to be used shall be HW (2.4 inch (61 mm) I.D.), and it is recommended using 4 inch (101.6 mm) diameter core barrels to better evaluate the Florida limestone properties.

4.8.5.1.2. Triple Tube Core Barrel

Similar to the double tube, above, but has an additional inner liner, consisting of either a clear plastic solid tube or a thin metal split tube, in which the core is retained. This barrel best preserves fractured and poor quality rock cores.

4.8.6. Sampling of Disintegrated Rock Transition Zones

General guidance on sampling of rock with various degrees of disintegration is given in Table 4-9[89].

Table 4-9 Sampling of Disintegrated Rock Zones

Description of Material	*Sampling Method*
Colluvium - Loosely packed, poorly sorted material.	Driven samples or triple tube core barrel. Double tube barrel is required for boulders. Denison Sampler can be used if no boulders are present.
Structureless residual soil - The soil shows none of the fabric of the rock from which it is derived.	Driven samples or triple tube core barrel. Dennison Sampler can be used. Hand-cut samples are best.
Decomposed rock containing rounded boulders that may be much harder than surrounding material.	Driven samples or triple tube core barrel. Double tube barrel is required to sample boulders.
Decomposed rock containing angular boulders separated by thin seams of friable material.	Double tube core barrel with triple tube barrel in weak seams.
Slightly decomposed rock - Friable material, if present, is limited to narrow seams.	Double tube core barrel.

4.8.7. Offshore Sampling

For water depths less than about 60 feet, land type soil boring equipment can be used on small jack-up platforms, small barges or barrel floats. Floating equipment requires suitable anchoring and is limited to calm sea conditions. For deeper water or more extreme seas, larger drill ships are required to obtain

[89] Modified from Brenner, R.P. and Phillipson, H.B., Sampling of Residual Soils in Hong Kong, Proceedings of the International Symposium of Soil Sampling, Singapore, 1979.

quality undisturbed samples. See Table 4-10 for common underwater samplers. Numerous types of oceanographic samplers, both open-tube and piston types are available for use from shipboard. These depend upon free-fall penetration and thus are limited in depth of exploration. The quality of samples obtained by most oceanographic samplers is not high because of their large length to diameter ratio.[90]

Table 4-10 Common Underwater Samplers

Sampler	Size of Sample	Length of Sample	Water Depth Limitations	Method of Penetration	Remarks
Petersen Dredge	Grab	± 6" depth.	To 200' and more with additional weight.	Clam shell jaw.	Reliable grab sampler, intact samples may be obtained with jaws that precisely mate.
Open Barrel Gravity Corer	2.5" to 6" diameter	Core barrels length from 6' to 30'.	No limit on depth but required weight, amount of line or size of vessel may control.	Spooled freely off the winch drum.	
Phleger Corer	About 1.5" diameter	Core barrels available in 12", 24" and 36" length.	From 25' to 200'.	Free fall from 10' to 20' above bottom.	Relatively lightweight core for upper 1' to 3' of bottom sediments. Samples usually not suitable for strength tests.
Piston Gravity Corer	Standard corer has 2.5" barrel	Standard barrel is 10'. Additional 10' sections can be added.	No depth limit except that available weight, amount of line, or size of vessel may control.	Free fall from calibrated height above bottom such that piston does not penetrate sediments.	Capable of obtaining samples suitable for strength tests with experienced crew, but samples may be seriously disturbed.
Vibratory Corer	Sample is 3.5" diameter	20' standard, can be lengthened to 40'.	Minimum depth limited by draft of support vessel. Maximum depth about 200'.	Pneumatic impacting vibratory hammer.	Samples are disturbed because of vibration and large area ratio. Samples not suitable for strength testing. Penetration resistance can be measured, continuous representative samples in marine soils are obtained.

[90] For detailed information on underwater sampling equipment, see ASTM STP 501, Underwater Soil Sampling, Testing and Construction Control, 1972, and Lee, H.J. and Clausner, J.E., Seafloor Soil Sampling and Geotechnical Parameter Determination - Handbook, Civil Engineering Laboratory, Department of the Navy, August 1979.

4.9. Penetration Resistance Tests

The most common test is the Standard Penetration Test (SPT), which measures resistance to the penetration of a standard sampler in borings. The method is rapid, and when tests are properly conducted in the field, they yield useful data, although there are many factors that can affect the results. A more controlled test is the cone penetrometer test in which a cone shaped tip is jacked from the surface of the ground to provide a continuous resistance record.

4.9.1. Standard Penetration Test (SPT)

This test is probably the most widely used field test in the United States. It has the advantages of simplicity, the availability of a wide variety of correlations for its data, and the fact that a sample is obtainable with each test.

4.9.1.1. Procedure

The test is covered under ASTM Standard D1586, which requires the use of a standard 2" (O.D.) split barrel sampler, driven by a 140-pound (63.6 kg) hammer dropping 30" (760 mm) in free fall. The procedure is generalized as follows:

- Clean the boring of all loose material, and material disturbed by drilling.
- Insert sampler, verifying the sampler reaches the same depth as was drilled.
- Obtain a consistent 30" free-fall drop of the hammer with two wraps of a rope around the cathead on the drill rig. (Cables attached to the hoisting drum should not be used because it is difficult to obtain free fall.)
- The sampler is advanced a total of 18 inches (450 mm).
- The number of blows required to advance the sampler for each of three 6" (150 mm) increments is recorded. The sum of the number of blows for the second and third increments is called the Standard Penetration Value, or more commonly, N-value (blows per foot {300 mm}).

The SPT values should not be used indiscriminately. They are sensitive to the fluctuations in individual drilling practices and equipment. Studies have also indicated that the results are more reliable in sands than clays. Although extensive use of this test in subsurface exploration is recommended, it should always be augmented by other field and laboratory tests, particularly when dealing with clays. The type of hammer (safety or automatic) shall be noted on the boring logs, since this will affect the actual input driving energy.

A method to measure the energy during the SPT has been developed (ASTM D 4633). Since there is a wide variability of performance in SPT hammers, this method is useful to evaluate an individual hammer's performance. The SPT installation procedure is similar to pile driving because it is governed by stress wave propagation. As a result, if force and velocity measurements are obtained during a test, the energy transmitted can be determined.

4.9.1.2. Corrections

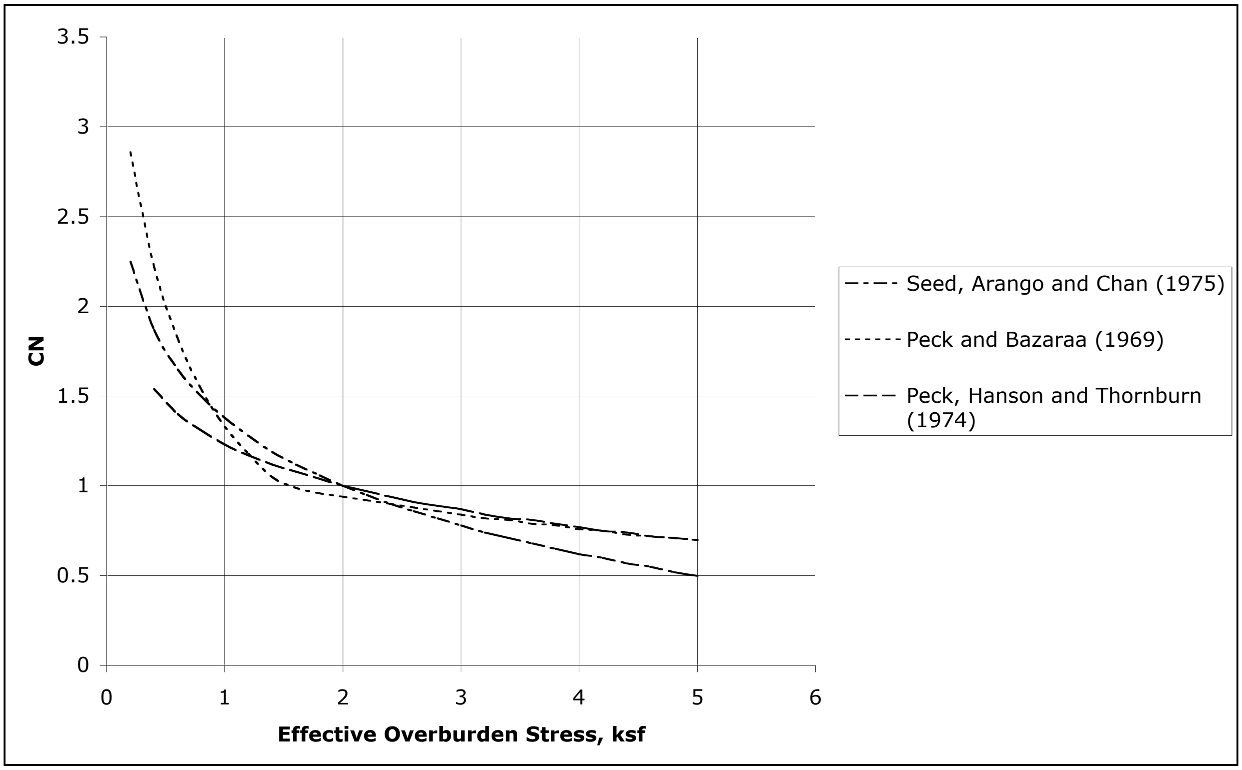
SPT values should be corrected for two factors: the overburden pressure and the efficiency of the hammer.

- Overburden. For STP values, correct N for overburden using:

Equation 4-1: $$(N_1)_{60} = C_N N$$

where C_N is a correction factor based on the effective overburden stress. Figure 4-10 shows values of C_N for a range of overburden stresses. Using this correction results in a value of N that would have been measured if the effective overburden stress had been 2 ksf.

Figure 4-10 Correlation Between C_N and Effective Overburden Pressure



- Hammer efficiency. Prior to 1980, the efficiency of the hammer was not well recognized as influencing the blow count and was usually not considered in analysis. Historically, SPT tests in the U.S. have been performed with machines with a mechanical efficiency of around 60%. Other types of testing equipment (especially the newer automatic hammers) have different efficiencies. Since N is also sensitive to the energy supplied by the equipment, N1 should be further corrected to the value at 60% of the input energy, $(N_1)_{60}$. The combined correction is:

Equation 4-2: $$(N_1)_{60} = C_N \frac{e_{hammer}}{60} N_1$$

Where e_{hammer} = efficiency of the SPT hammer used, percent. Table 4-11 shows representative efficiencies for SPT procedures.

Table 4-11 Energy Ratios for SPT Procedures Country Hammer Type Hammer Release Estimated Rod Energy (%)

Country	*Type of Ram*	*Type of Ram Lifting and Release Mechanism*	*Efficiency, Percent*
Japan	Donut	Free-fall	78
	Donut	Rope & Pulley with special throw release	67

U.S.A.	Safety	Rope & Pulley	60
	Donut	Rope & Pulley	45
Argentina	Donut	Rope & Pulley	45
China	Donut	Free-fall	60
	Donut	Rope and Pulley	50

4.9.1.3. Correlations

Because the Standard Penetration Test is relatively simple to run, it has given rise to many correlations with many different soil properties. These are especially useful in situations where undisturbed samples are unavailable, which is frequently the case with deep foundations. Some of these correlations are discussed below.

4.9.1.3.1. Compactness and Consistency

Table 4-12 shows the basic relationship of SPT results (both N_{60} and automatic hammer) results with compactness for granular soils and consistency for cohesive soils. Although this type of table is usual for rough estimates of soil properties, when possible more accurate correlations should be used; some of these are shown in the following sections.

Table 4-12 Relative Density or Consistency of Soils as a Function of SPT N Values

Nomenclature	***Relative Density, Percent***	***Internal Friction Angle φ, degrees***	***Moist Unit Weight, pcf***	***Safety Hammer SPT N_{60} Value (Blow/Foot {300 mm})***	***Automatic Hammer SPT N-Value (Blow/Foot {300 mm})***
		Cohesionless Soils			
Very Loose	0-15%	< 28	<100	Less than 4	Less than 3
Loose	15-35%	28-30	95-125	4 – 10	3 – 7
Medium Dense	35-65%	30-36	110-130	10 – 30	7 – 21
Dense	65-85%	36-41	110-140	30 – 50	21 – 35
Very Dense	85-100%	> 41	> 130	> 50	> 35
Nomenclature	***Hand Manipulation Characteristics***	***Estimated Unconfined Compression Strength q_u, ksf***	***Moist Unit Weight, pcf***	***Safety Hammer SPT N_{60} Value (Blow/Foot {300 mm})***	***Automatic Hammer SPT N-Value (Blow/Foot {300 mm})***
		Cohesive Soils			

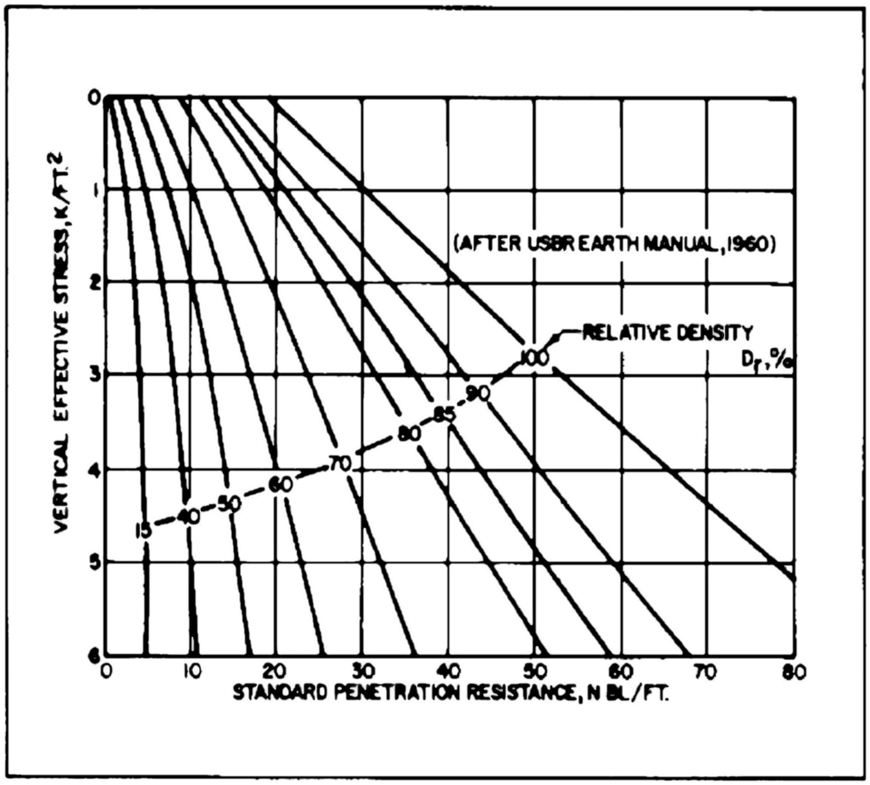
Very Soft	***Extruded between fingers when squeezed***	< 0.50	100-120	< 2	< 1
Soft	***Moulded by light finger pressure***	0.5 - 1	100-120	2 – 4	1 – 3
Firm or Medium	***Moulded by strong finger pressure***	1 - 2	110-130	4 – 8	3 – 6
Stiff	***Readily indented by thumb but penetrated with great effort***	2 - 4	120-140	8 – 15	6 – 11
Very Stiff	***Readily indented by thumbnail***	4 - 8	120-140	15 – 30	11 – 21
Hard	***Indented with difficulty by thumbnail***	> 8	> 130	> 30	> 21

4.9.1.3.2. Relative Density of Granular (but fine grained) Deposits

If the test is a true standard test, the "N" value is influenced by the effective vertical stress at the level where "N" is measured, density of the soil, stress history, gradation and other factors.[91] The Gibbs & Holtz correlation of Figure 4-11 is commonly used to estimate the relative density from SPT.

[91] Marcuson, W.F. III, and Bieganouski, W.A., SPT and Relative Density in Coarse Sands, Journal of geotechnical engineering Division, ASCE, Vol. 103, No. GT 11, 1977.

Figure 4-11 Correlations between Relative Density and Standard Penetration Resistance in Accordance with Gibbs and Holtz[92]

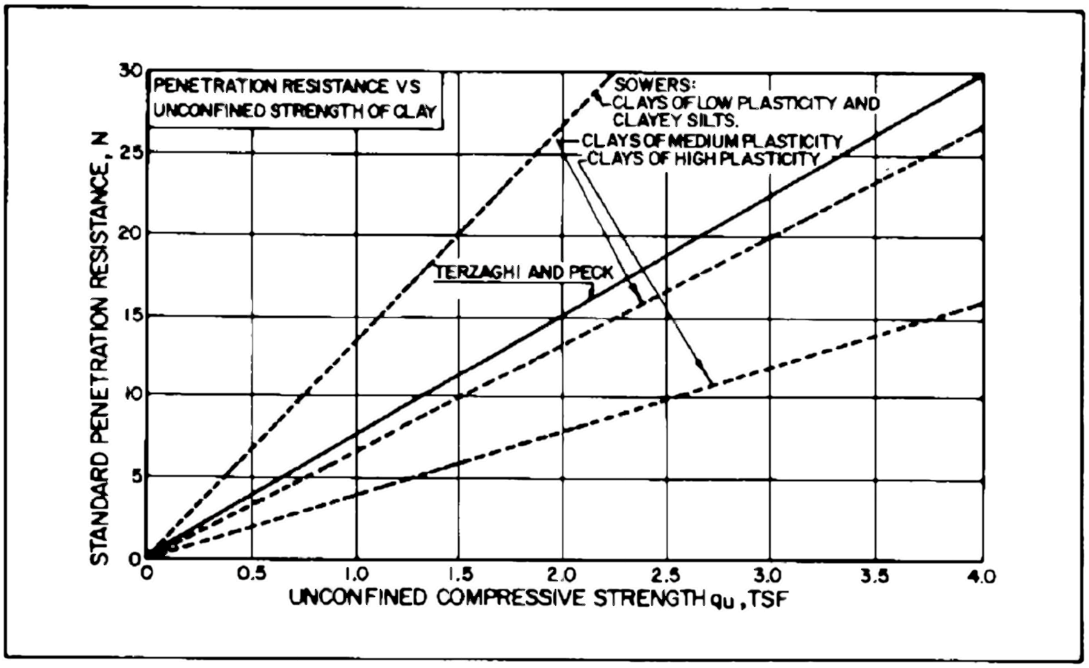


4.9.1.3.3. Undrained Shear Strength

A crude estimate for the undrained shear strength can be made using Figure 4-12. Correlations are not meaningful for medium to soft clays where effects of disturbance are excessive.

[92] Lacroix, Y. and Horn, H.M., Direct Determination and Indirect Evaluation of Relative Density and Earthwork Construction Projects, ASTM STP 523, 1973.

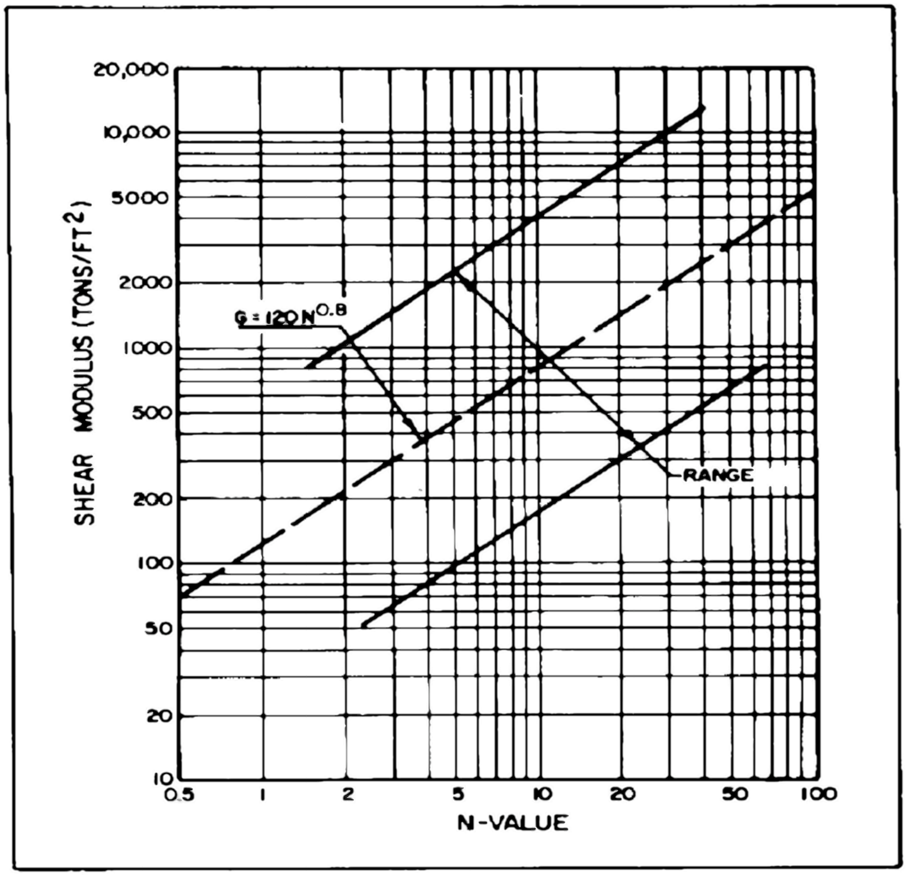
Figure 4-12 Correlations of Standard Penetration Resistance with Unconfined Strength of Clay



4.9.1.3.4. *Shear Modulus at Very Small Strains*

A crude estimate of the shear modulus at small strains for sandy and cohesive soils can be obtained from the statistical relationships in Figure 4-13.

Figure 4-13 Shear Modulus vs. N Values (SPT) at Very Small Strains[93]

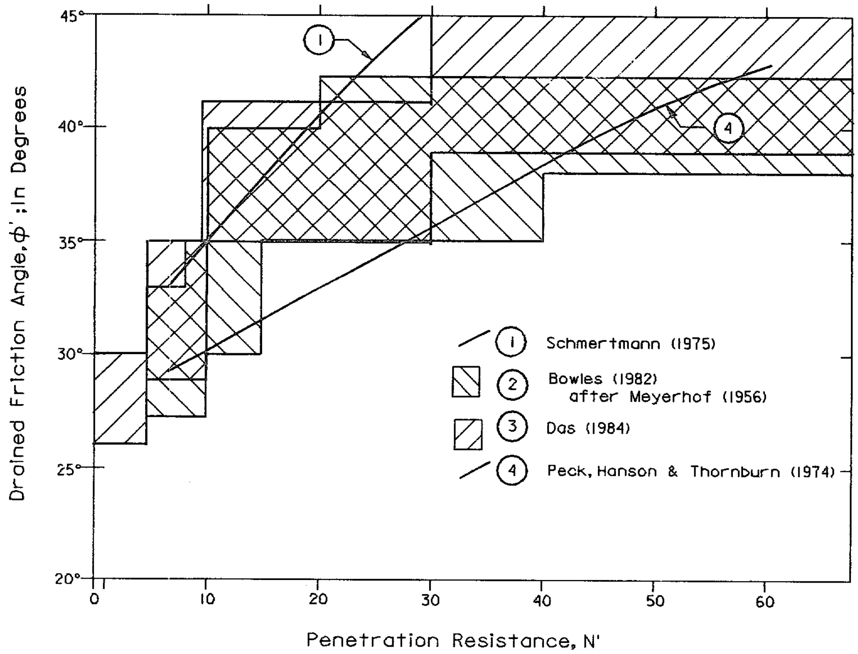


4.9.1.3.5. Drained Friction Angle ϕ'

The drained friction angle ϕ' can be estimated from N' using Figure 4-14. This is used mostly with retaining walls where the drained friction angle is significant for long-term behavior of the wall.

93 Ohsaki, Y., and Iwasaki, R., On Dynamic Shear Moduli and Poisson's Ratios of Soil Deposits, Soils and Foundations Vol. 13, No. 4, 1973.

Figure 4-14 ϕ' vs. N' for Granular Materials



4.9.1.4. Limitations

Except where confirmed by specific structural property tests, these relationships are suitable for estimates only. Blow counts are affected by operational procedures, by the presence of gravel, or cementation. They do not reflect fractures or slickensides in clay, which may be very important to strength characteristics. The standard penetration test results (N values) are influenced by operational procedures as illustrated in Table 4-13.

Table 4-13 Procedures That May Affect the Measured "N" Values[94]

Faulty Procedure	*Description*
Inadequate cleaning of the borehole (This may also prevent sample recovery.)	SPT is only partially made in original soil. Sludge may be trapped in the sampler and compressed as the sampler is driven, increasing the blow count.
Not seating the sampler spoon on undisturbed material	Incorrect "N" values obtained.
Driving of the sample spoon above the bottom of the casing	"N" values are increased in sands and reduced in cohesive soils.
Failure to maintain sufficient hydrostatic head in boring	The water table in the borehole must be at least equal to the piezometric level in the sand, otherwise the sand at the bottom of the borehole may be transformed into a loose state.

[94] Canadian Geotechnical Society, Properties of Soil and Rock, Canadian Foundation Engineering Manual, Part I, Canadian Geotechnical Society, 1978

Faulty Procedure	*Description*
Attitude of operators perhaps the mood of operator and time of drilling.	Blow counts for the same soil using the same rig can vary, depending on who is operating the rig, and
Overdrive sampler sampler	Higher blow counts usually result from overdriven sampler.
Sampler plugged by gravel	Higher blow counts result when gravel plugs sampler, resistance of loose sand could be highly overestimated.
Plugged casing	High "N" values may be recorded for loose sand when sampling below groundwater table. Hydrostatic pressure causes sand to rise and plug casing.
Overwashing ahead of casing	Low blow count may result for dense sand since overwashing loosens sand.
Drilling method	Drilling technique (e.g., cased holes vs. mud stabilized holes) may result in different "N" values for the same soil.
Not using the standard hammer drop	Energy delivered per blow is not uniform. European countries have adopted an automatic trip hammer that is now coming into use in North America.
Free fall of the drive weight is not attained	Using more than 1-1/2 turns of rope around the drum and/or using wire cable will restrict the fall of the drive weight.
Not using correct weight	Driller frequently supplies drive hammers with weights varying from the standard by as much as10 lbs.
Weight does not strike the drive cap concentrically	Impact energy is reduced, increasing "N" values.
Not using a guide rod	Incorrect "N" value obtained.
Not using a good tip on the sampling spoon	If the tip is damaged and reduces the opening or increases the end area the" N" value can be increased.
Use of drill rods heavier than standard	With heavier rods more energy is absorbed by the rods causing an increase in the blow count.
Not recording blow counts and penetration accurately	Incorrect "N" values obtained.
Incorrect drilling procedures	The SPT was originally developed from wash boring techniques. Drilling procedures that seriously disturb the soil will affect the "N" value, e.g. drilling with cable tool equipment.
Using drill holes that are too large	Holes greater than 10 cm (4 in) in diameter are not recommended. Use of larger diameters may result in decreases in the blow count.
Inadequate supervision	Frequently a sampler will be impeded by gravel or cobbles causing a sudden increase in blow count; an inexperienced observer does not recognize this. (Accurate recording of drilling, sampling and depth is always required.)
Improper logging of soils	Not describing the sample correctly.

Faulty Procedure	*Description*
Using too large a pump	Too high a pump capacity will loosen the soil at the base of the hole causing a decrease in blow count.

4.9.2. Cone Penetrometer Test (CPT)

4.9.2.1. Test Description

The Cone Penetrometer Test is a quasi-static penetration test in which a cylindrical rod with a conical point is advanced through the soil at a constant rate and the resistance to penetration is measured. A series of tests performed at varying depths at one location is commonly called a sounding.

Several types of penetrometers are in use, including mechanical (mantle) cone, mechanical friction-cone, electric cone, electric friction-cone, and piezocone penetrometers. Cone penetrometers measure the resistance to penetration at the tip of the penetrometer, or the end-bearing component of resistance. Friction-cone penetrometers are equipped with a friction sleeve, which provides the added capability of measuring the side friction component of resistance. Mechanical penetrometers have telescoping tips allowing measurements to be taken incrementally, generally at intervals of 8 inches (200 mm) or less. Electric (or electronic) penetrometers use electric force transducers to obtain continuous measurements with depth. Piezocone penetrometers are electric penetrometers, which are also capable of measuring pore water pressures during penetration.

For all types of penetrometers, cone dimensions of a 60-degree tip angle and a 1.55 in^2 (10 cm^2) projected end area are standard. Friction sleeve outside diameter is the same as the base of the cone. Penetration rates should be between 0.4 to 0.8 in/sec (10 to 20 mm/sec). Tests shall be performed in accordance with ASTM D 3441 (which includes mechanical cones) and ASTM D 5778 (which includes piezocones).

The penetrometer data is plotted showing the end-bearing resistance, the friction resistance and the friction ratio (friction resistance divided by end bearing resistance) as functions of depth. Pore pressures, if measured, can also be plotted with depth. The results should also be presented in tabular form indicating the interpreted results of the raw data. See Figure 4-15**,** Figure 4-16**,** and Figure 4-17 (Note: the log for a standard cone penetration test would only include the first three plots: tip resistance, local friction, and friction ratio; shown in Figure 4-16).

Figure 4-15 Typical Log from Mechanical Friction-Cone

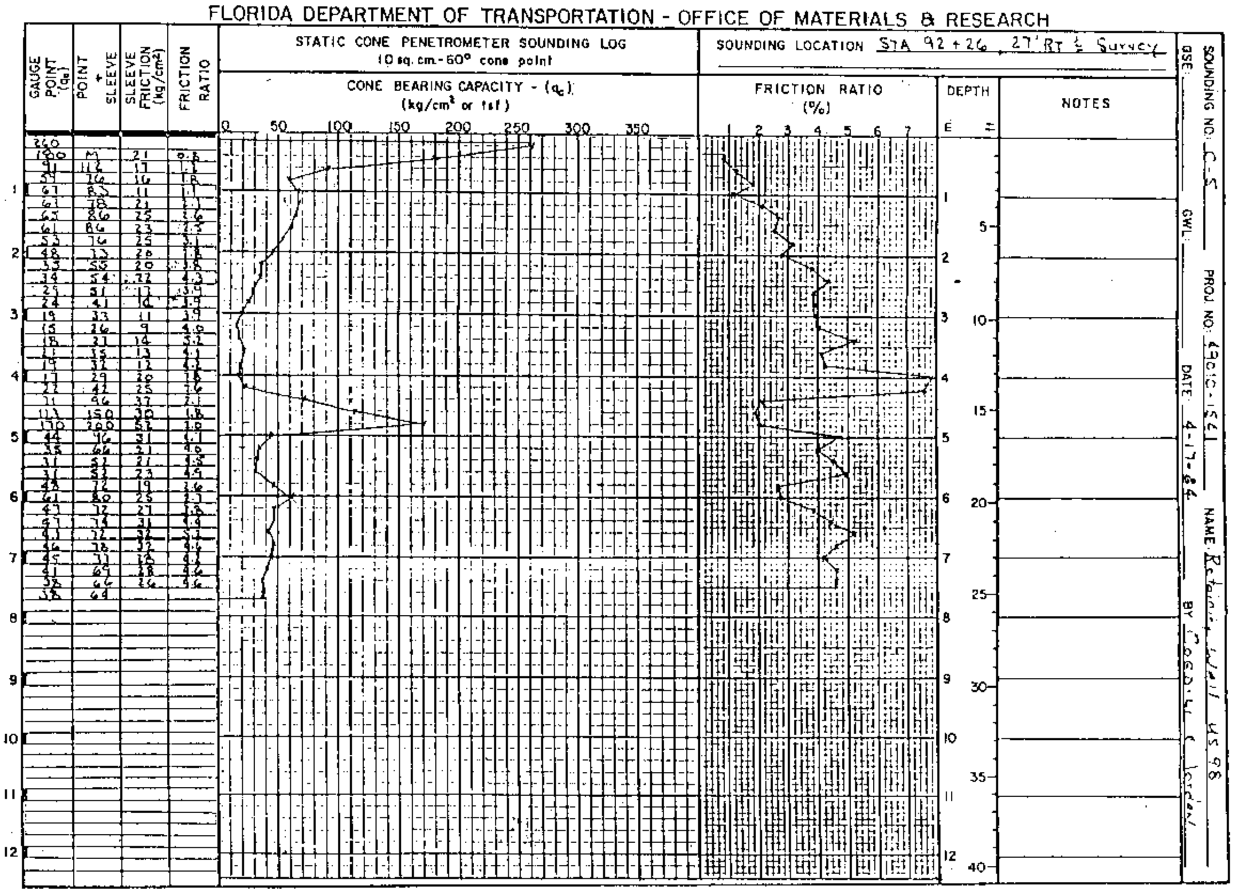


Figure 4-16 Typical Log from Electric Piezocone

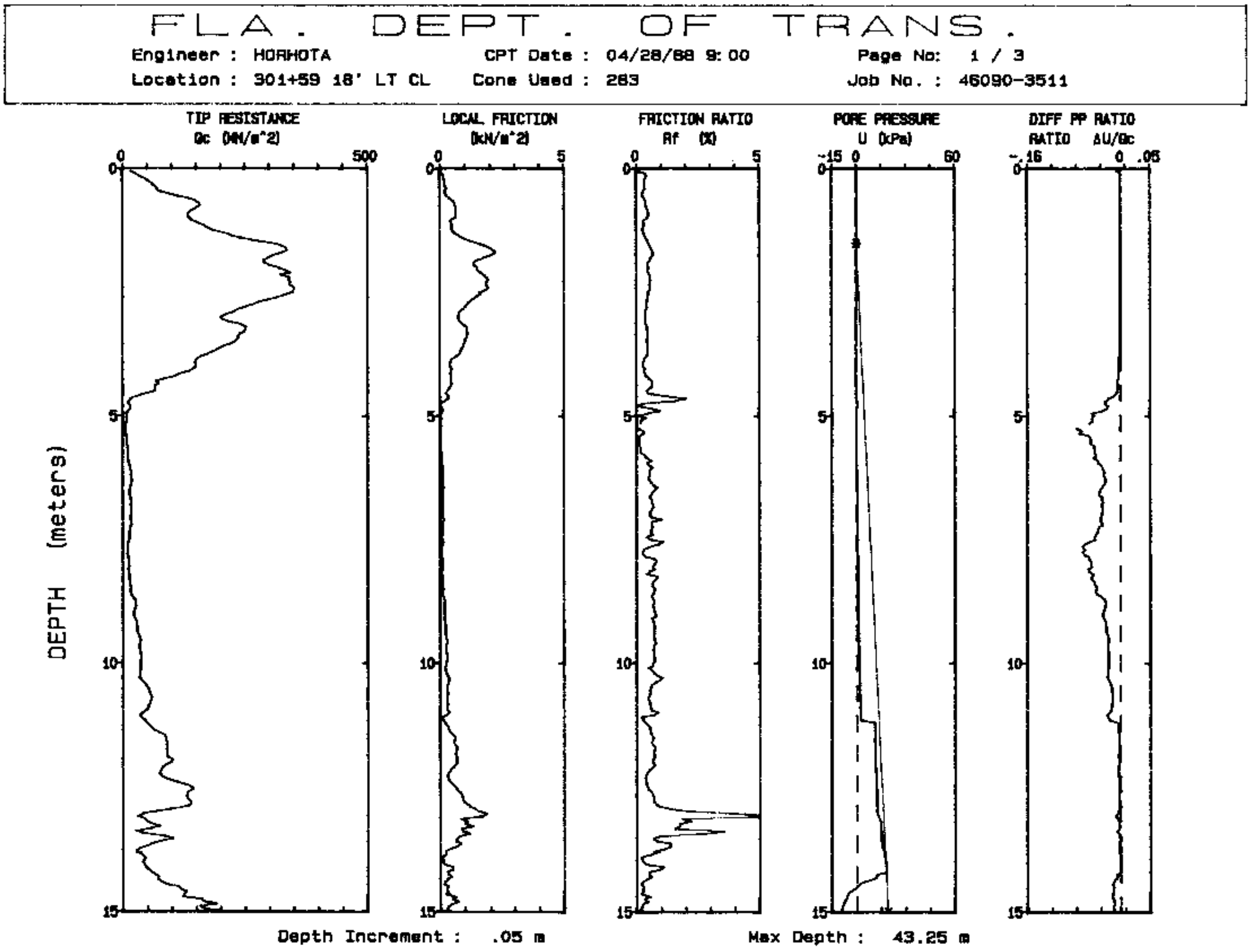


Figure 4-17 Typical Interpreted Output from Electric Cone Penetrometer

FLORIDA D.O.T MATERIALS OFFICE

CPT DATE :04/28/88 9:00 ENGINEER :BLANTON
LOCATION :301+59 6 m LT CL Cone Used :283
Job No. :46090-3511 Water table (meters) : 1.5
Tot. Unit Wt. (avg) : 510 N/m^3

DEPTH (meters)	Qc (avg) (MN/m^2)	Fs (avg) (kN/m^2)	Rf (avg) (%)	SIGV' (MPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	Su (MPa)
0.50	53.24	0.16	0.31	0.04	sand to silty sand	>90	>48	13	UNDEFINED
1.00	139.74	0.56	0.40	0.13	sand	>90	>48	27	UNDEFINED
1.50	205.64	0.71	0.35	0.22	sand	>90	>48	39	UNDEFINED
2.00	312.59	1.77	0.57	0.28	gravelly sand to sand	>90	>48	50	UNDEFINED
2.50	341.26	1.77	0.52	0.31	gravelly sand to sand	>90	>48	>50	UNDEFINED
3.00	262.08	1.08	0.41	0.35	gravelly sand to sand	>90	>48	42	UNDEFINED
3.50	236.04	1.00	0.42	0.38	sand	>90	>48	45	UNDEFINED
4.00	173.89	0.67	0.39	0.42	sand	>90	46-48	33	UNDEFINED
4.50	92.91	0.41	0.44	0.45	sand to silty sand	70-80	44-46	22	UNDEFINED
5.00	17.01	0.13	0.79	0.49	sandy silt to clayey silt	UNDFND	UNDFD	7	1.07
5.50	6.64	0.01	0.21	0.52	sensitive fine grained	UNDFND	UNDFD	3	.38
6.00	10.38	0.03	0.32	0.55	sandy silt to clayey silt	UNDFND	UNDFD	4	.62
6.50	16.33	0.10	0.60	0.59	sandy silt to clayey silt	UNDFND	UNDFD	6	1.01
7.00	15.86	0.10	0.61	0.62	sandy silt to clayey silt	UNDFND	UNDFD	6	.97
7.50	14.86	0.09	0.63	0.66	sandy silt to clayey silt	UNDFND	UNDFD	6	.90
8.00	10.37	0.06	0.61	0.69	sandy silt to clayey silt	UNDFND	UNDFD	4	.60
8.50	13.54	0.09	0.67	0.73	sandy silt to clayey silt	UNDFND	UNDFD	5	.80
9.00	22.86	0.16	0.70	0.76	silty sand to sandy silt	<40	34-36	7	UNDEFINED
9.50	29.15	0.21	0.71	0.80	silty sand to sandy silt	<40	36-38	9	UNDEFINED
10.00	35.88	0.26	0.72	0.83	silty sand to sandy silt	<40	36-38	11	UNDEFINED
10.50	39.31	0.28	0.71	0.87	silty sand to sandy silt	40-50	36-38	13	UNDEFINED
11.00	53.59	0.30	0.56	0.90	sand to silty sand	50-60	38-40	13	UNDEFINED
11.50	58.47	0.30	0.52	0.94	sand to silty sand	50-60	38-40	14	UNDEFINED
12.00	92.18	0.64	0.69	0.97	sand to silty sand	60-70	40-42	22	UNDEFINED
12.50	94.25	0.44	0.47	1.01	sand to silty sand	60-70	40-42	23	UNDEFINED
13.00	125.46	1.04	0.83	1.04	sand to silty sand	70-80	40-42	30	UNDEFINED
13.50	50.89	1.15	2.26	1.08	sandy silt to clayey silt	UNDFND	UNDFD	19	3.24
14.00	51.81	0.39	0.76	1.11	silty sand to sandy silt	40-50	36-38	17	UNDEFINED
14.50	68.95	0.32	0.46	1.15	sand to silty sand	50-60	38-40	17	UNDEFINED
15.00	154.16	0.41	0.27	1.18	sand	70-80	42-44	30	UNDEFINED
15.50	214.47	0.46	0.22	1.22	gravelly sand to sand	80-90	42-44	34	UNDEFINED
16.00	239.03	0.43	0.18	1.25	gravelly sand to sand	80-90	42-44	38	UNDEFINED
16.50	168.43	0.21	0.13	1.29	sand	70-80	42-44	32	UNDEFINED
17.00	102.13	0.13	0.13	1.32	sand	60-70	38-40	20	UNDEFINED
17.50	101.49	0.22	0.22	1.36	sand	60-70	38-40	19	UNDEFINED
18.00	171.24	0.28	0.16	1.39	sand	70-80	40-42	33	UNDEFINED
18.50	174.32	0.23	0.13	1.43	sand	70-80	40-42	33	UNDEFINED
19.00	191.14	0.25	0.13	1.46	sand	70-80	42-44	37	UNDEFINED

Dr - All sands (Jamiolkowski et al. 1985) PHI - Robertson and Campanella 1983 Su: Nk= 15

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****

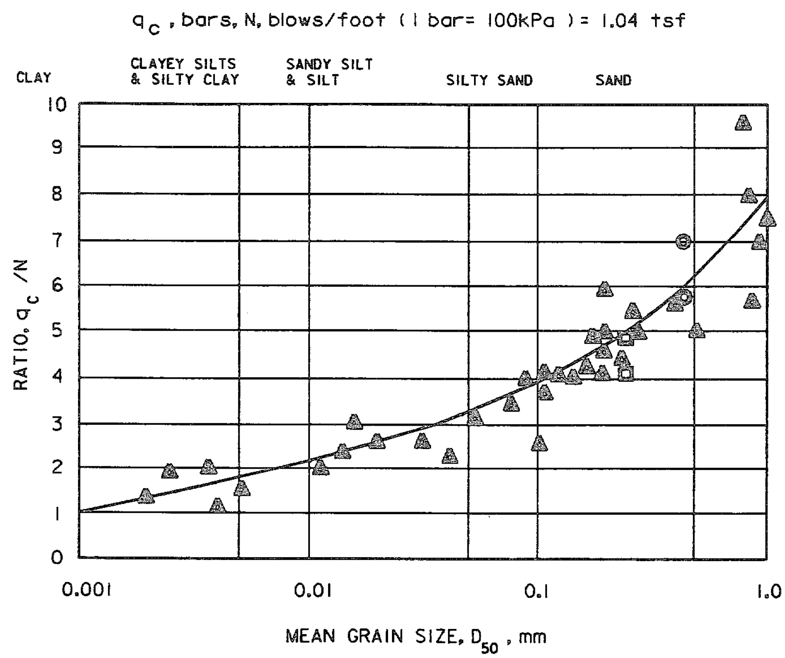
The friction ratio plot can be analyzed to determine soil type. Many correlations of the cone test results to other soil parameters have been made, and design methods are available for spread footings and piles. The penetrometer can be used in sands or clays, but not in rock or other extremely dense soils. Generally, soil samples are not obtained with soundings, so penetrometer exploration should always be augmented by SPT borings or other borings with soil samples taken.

The piezocone penetrometer can also be used to measure the dissipation rate of the excessive pore water pressure. This type of test is useful for subsoils, such as fibrous peat or muck that are very sensitive to sampling techniques. The cone should be equipped with a pressure transducer that is capable of measuring the induced water pressure. To perform this test, the cone will be advanced into the subsoil at a standard rate of 0.8 inch/sec (20 mm/sec). Pore water pressures will be measured immediately and at several time intervals thereafter. Use the recorded data to plot a pore pressure versus log-time graph. Using this graph one can directly calculates the pore water pressure dissipation rate or rate of settlement of the soil.

4.9.2.2. Correlations

The ratio (q_c/N) is typically in the range of 2 to 6 and is related to median grain size (see Figure 4-18).

Figure 4-18 q_c/N versus D_{50}[95]



The undrained strength of fine-grained soils may be estimated by using a modification of bearing capacity theory:

Equation 4-3: $q_u = \dfrac{q_c - p_o}{N_k}$

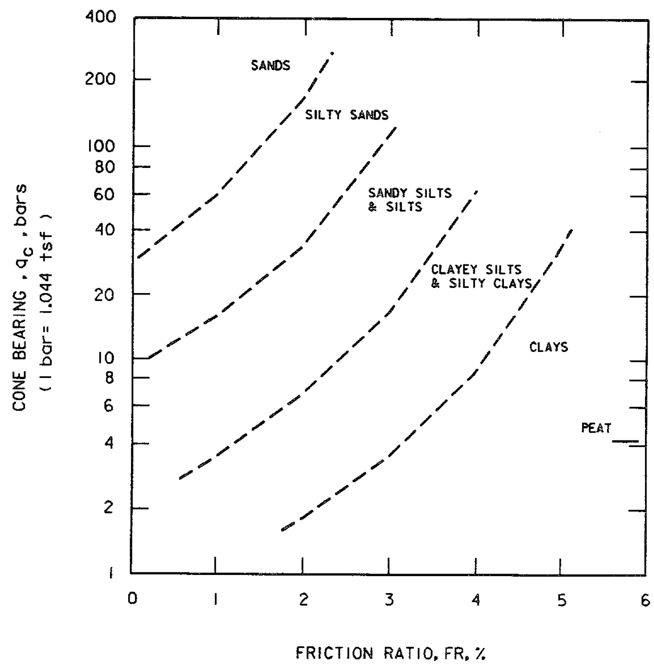
Where

- q_c = unit point resistance of cone penetrometer
- p_o = the in situ total overburden pressure
- N_k = empirical cone factor typically in the range of 10 to 20 The N_k value should be based on local experience and correlation to laboratory tests.

Cone penetration tests also may be used to infer soil classification to supplement physical sampling. Figure 4-19 indicates probable soil type as a function of cone resistance and friction ratio. Cone penetration tests may produce erratic results in gravelly soils.

[95] Robertson, P. K., and Campanella, R. G. 1983. "Interpretation of Cone Penetration Tests; Parts I and II," Canadian Geotechnical Journal, Vol 20, No. 4, pp 718-745.

Figure 4-19 Soil classification from cone penetrometer



4.9.3. Dynamic Cone Penetrometer Test

This test is similar to the cone penetrometer test except, instead of being pushed at a constant rate, the cone is driven into the soil. The number of blows required to advance the cone in 6" (150 mm) increments is recorded. A single test generally consists of two increments. Tests can be performed continuously to the depth desired with an expendable cone, which is left in the ground upon drill rod withdrawal, or they can be performed at specified intervals by using a retractable cone and advancing the hole by auger or other means between tests. Samples are not obtained.

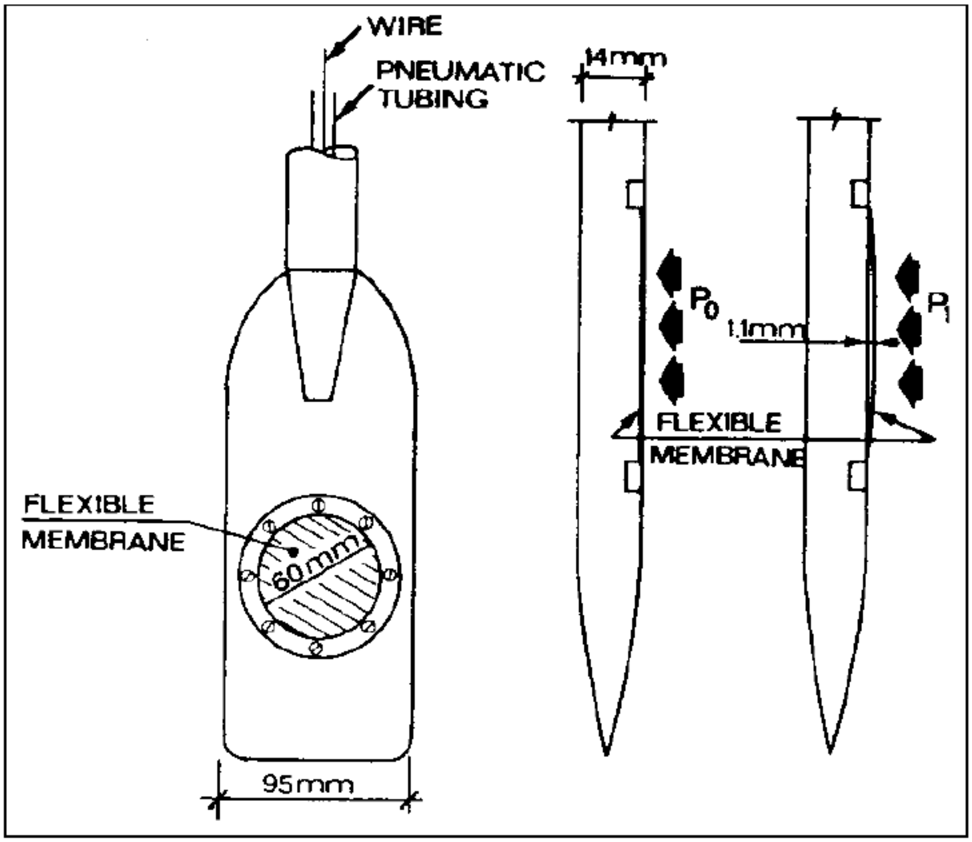
Blow counts can generally be used to identify material type and relative density. In granular soils, blow counts from the second 6" (150 mm) increment tend to be larger than for the first increment. In cohesive soils, the blow counts from the two increments tend to be about the same. While correlations between blow counts and engineering properties of the soil exist, they are not as widely accepted as those for the SPT.

4.9.4. Dilatometer Test (DMT)

The dilatometer is a 3.75" (95 mm) wide and 0.55" (14 mm) thick stainless steel blade with a thin 2.4" (60 mm) diameter expandable metal membrane on one side. While the membrane is flush with the blade surface, the blade is either pushed or driven into the soil using a penetrometer or drilling rig. Rods carry pneumatic and electrical lines from the membrane to the surface. At depth intervals of 8 inch (200 mm), the pressurized gas expands the membrane and both the pressure required to begin membrane movement and that required to expand the membrane into the soil 0.04 inches (1.1 mm) are measured. Additionally, upon venting the pressure corresponding to the return of the membrane to its original position may be recorded (see Figure 4-20, Figure 4-21, and Figure 4-22).

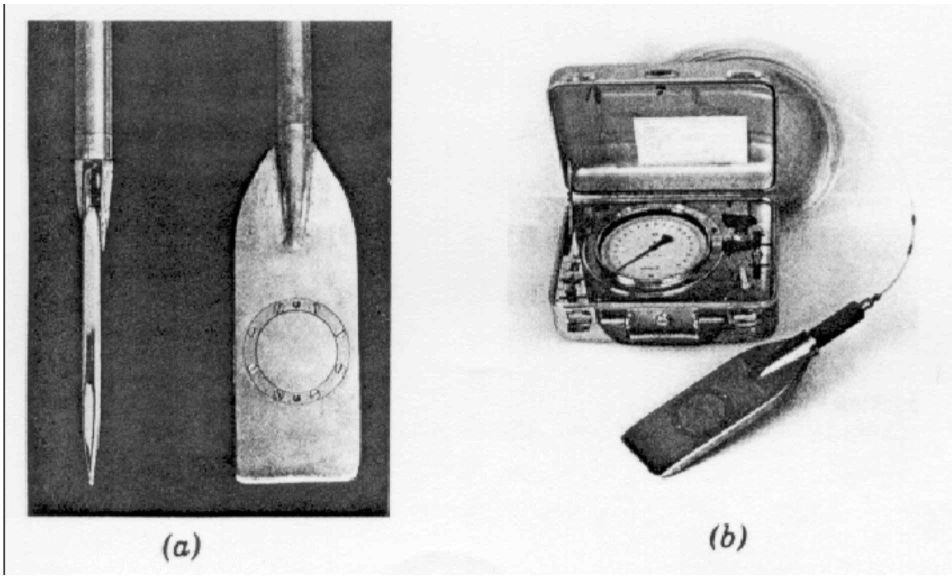
Through developed correlations, information can be deduced concerning material type, pore water pressure, in-situ horizontal and vertical stresses, void ratio or relative density, modulus, shear strength parameters, and consolidation parameters. Compared to the pressuremeter, the flat dilatometer has the advantage of reduced soil disturbance during penetration.

Figure 4-20 Schematic of the Marchetti Flat Dilatometer[96]



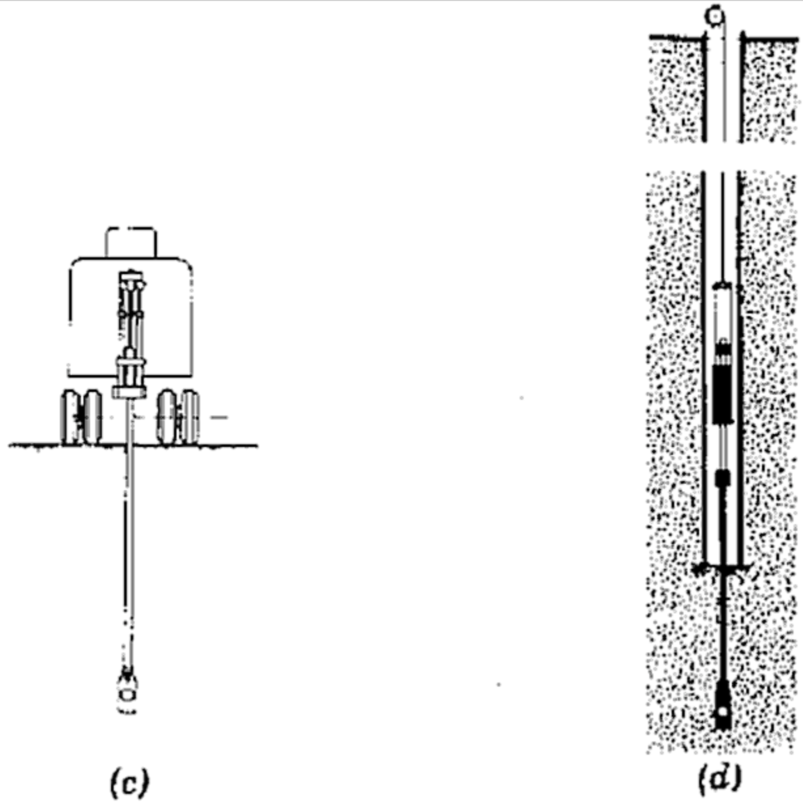
[96] Baldi, G., Bellotti R., Ghionna, V., Jamiolkowski, M., Marchetti, S. and Pasqualini, E. Flat Dilatometer Tests in Calibration Chambers, Use of Insitu Tests in geotechnical engineering, ASCE Specialty Conference, Geotechnical Special Publication No. 6, 1986.

Figure 4-21 Dilatometer[97]



[97] Marchetti, Silvano, In-Situ Tests by Flat Dilatometer, Journal of the geotechnical engineering Division, ASCE, Vol. 106, No. GT3, March 1980.

Figure 4-22 Dilatometer (Continued)



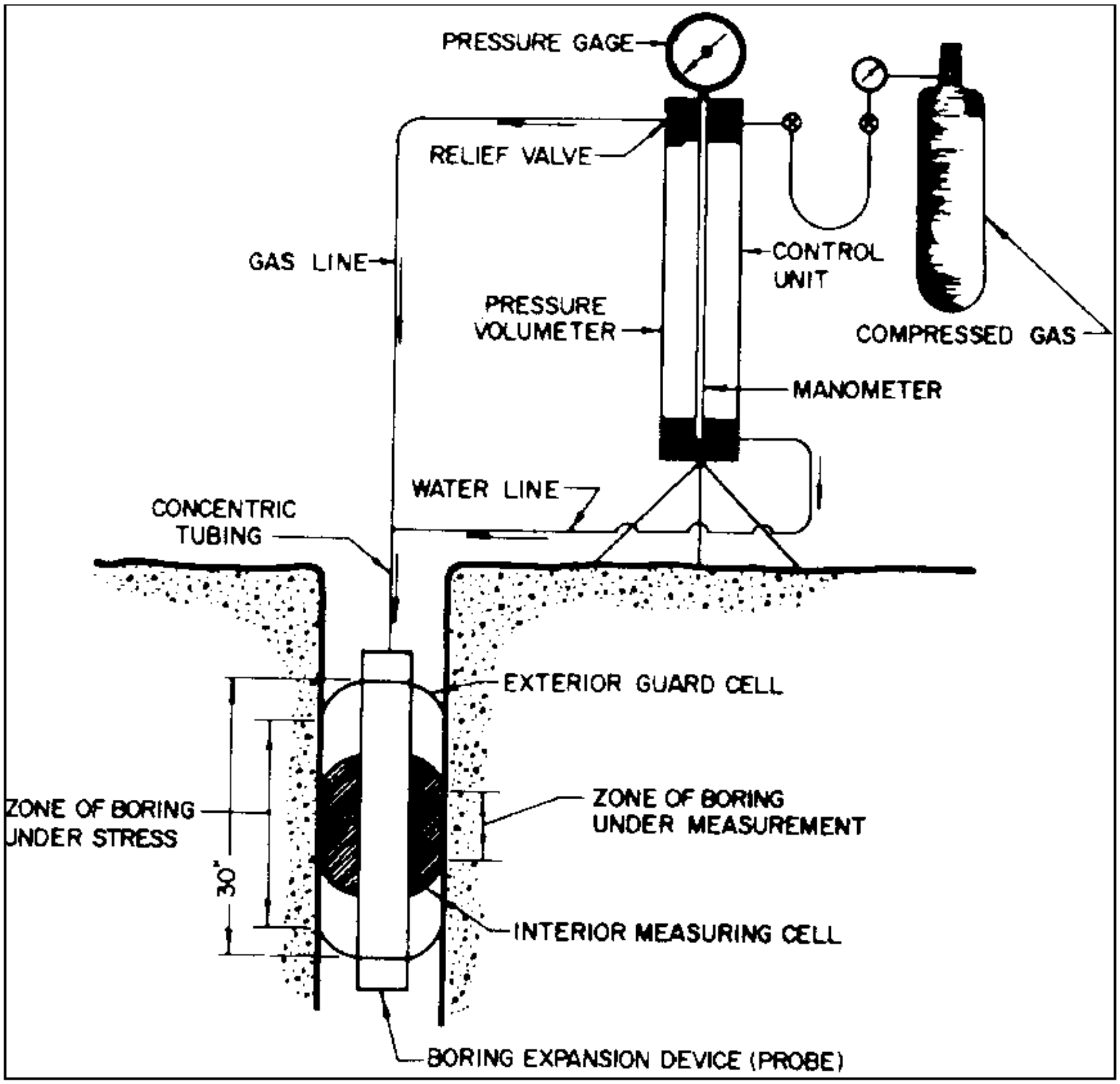
Flat Dilatometer: (a) Side View and Front View; (b) Blade, Control Unit and Cable; (c) Dilatometer Being Jacked into Ground; (d) Dilatometer Being Driven by Down-The-Hole Wireline Hammer

4.9.5. Pressuremeter Test (PMT)

This test is performed with a cylindrical probe placed at the desired depth in a borehole. The Menard type pressuremeter requires pre-drilling of the borehole; the self-boring type pressuremeter advances the hole itself, thus reducing soil disturbance. The Menard probe contains three flexible rubber membranes (see Figure 4-23). The middle membrane provides measurements, while the outer two are guard cells to reduce the influence of end effects on the measurements. When in place, the guard cell membranes are inflated by pressurized gas while the middle membrane is inflated with water by means of pressurized gas. The pressure in all the cells is incremented and decremented by the same amount. The measured volume change of the middle membrane is plotted against applied pressure. Tests shall be performed in accordance with ASTM D 4719. Studies have shown that the guard cells can be eliminated without sacrificing the accuracy of the test data provided the probe is sufficiently long.

Furthermore, pumped air can be substituted for the pressurized gas used to inflate the membrane with water. The TAXAM pressuremeter is an example of this type.

Figure 4-23 Menard Pressuremeter Equipment



Test results are normally used to directly calculate bearing capacity and settlements, but the test can be used to estimate strength parameters. The undrained strength of fine-grained materials is given by:

Equation 4-4: $$q_u = \frac{p_1 - p'_{ho}}{2K_b}$$

Where

- p_1 = limit pressure
- p'_{ho} = effective at-rest horizontal pressure
- K_b = a coefficient typically in the range of 2.5 to 3.5 for most clays.

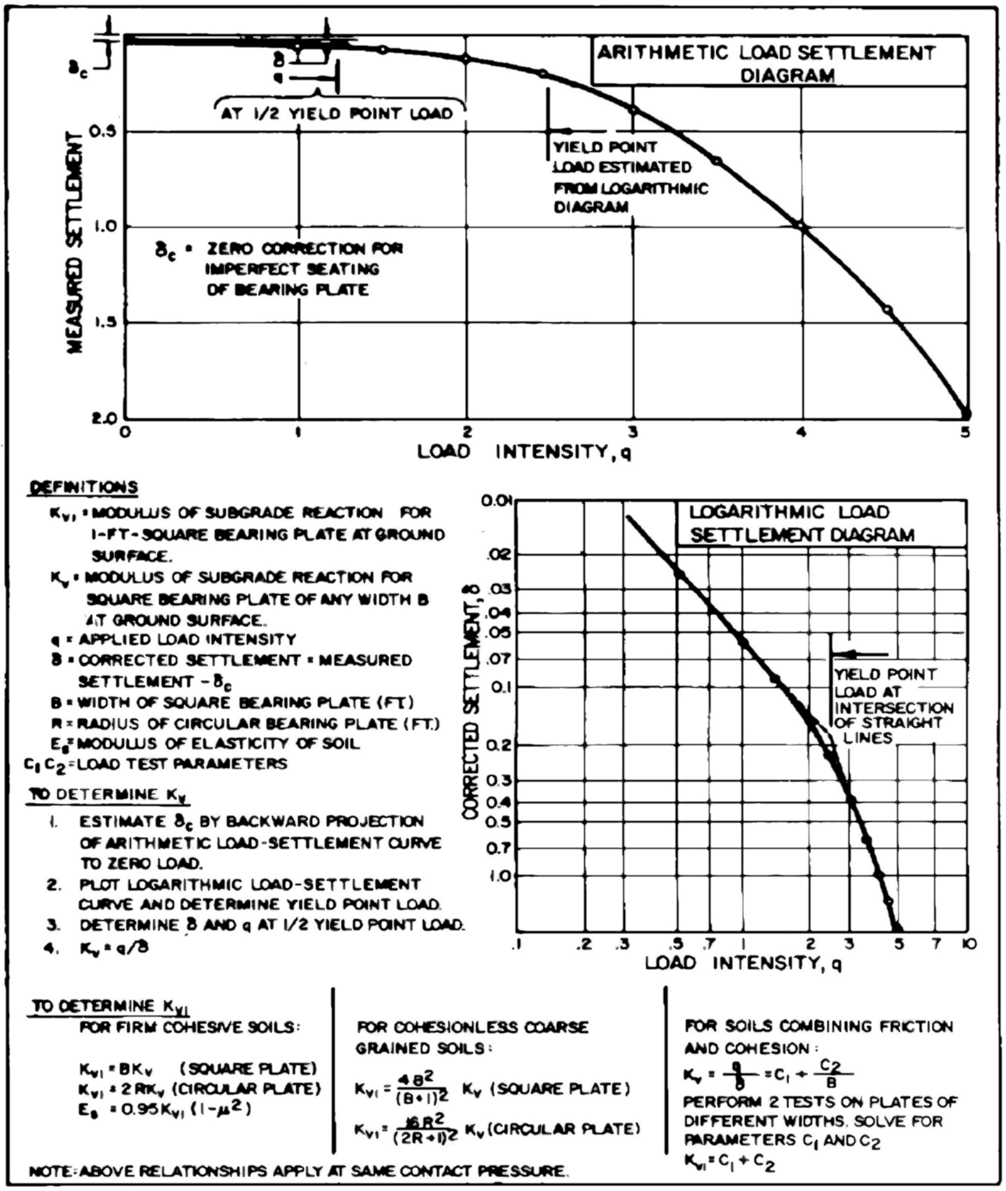
Correlation with laboratory tests and local experience is recommended. The pressuremeter test is very sensitive to borehole disturbance and the data may be difficult to interpret for some soils.

4.9.6. Plate Bearing Test

The plate-bearing test can be used as an indicator of compressibility and as a supplement to other compressibility data.

- Procedure. For ordinary tests for foundation studies, use procedure of ASTM Standard D1194, Test for Bearing Capacity of Soil for Static Load on Spread Footings, except that dial gages reading to 0.001" should be substituted. Tests are utilized to estimate the modulus of subgrade reaction and settlements of spread foundations. Results obtained have no relation to deep-seated settlement from volume change under load of entire foundation.
- Analysis of Test Results. (See Figure 4-24.) Determine yield point pressure for logarithmic plot of load versus settlement. Convert modulus of subgrade reaction determined from test K_v, to the property K_v for use in computing immediate settlement (Chapter 5). In general, tests should be conducted with groundwater saturation conditions simulating those anticipated under the actual structure.

Figure 4-24 Analysis of Plate Bearing Tests



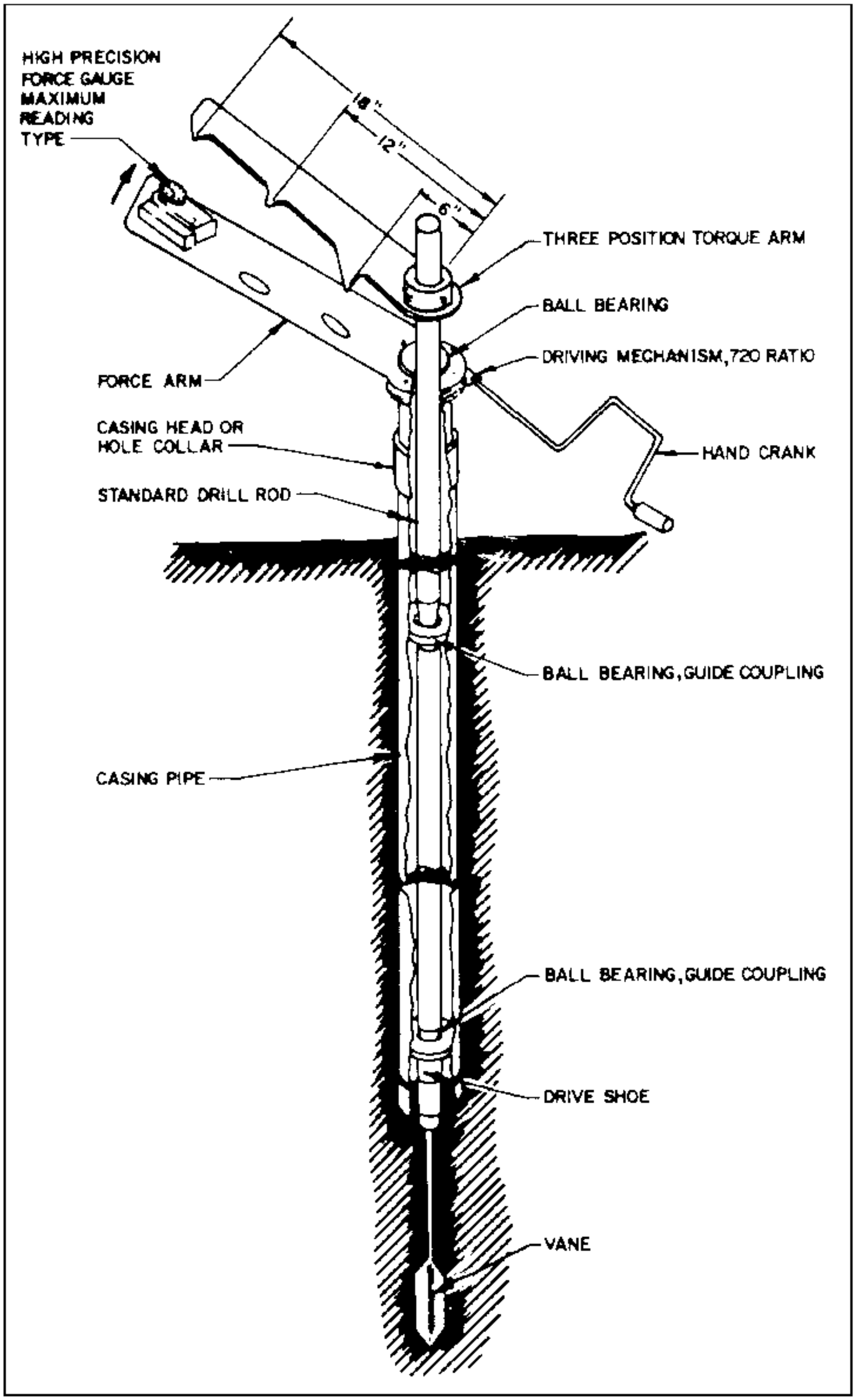
Data from the plate load test is applicable to material only in the immediate zone (say to a depth of two plate diameters) of the plate and should not be extrapolated unless material at greater depth is essentially the same.

4.9.7. Field Vane Test

This test consists of advancing a four-bladed vane into cohesive soil to the desired depth and applying a measured torque at a constant rate until the soil fails in shear along a cylindrical surface. (See Figure 4-25) The torque measured at failure provides the undrained shear strength of the soil. A second test run immediately after remolding at the same depth provides the remolded strength of the soil and thus information on soil sensitivity. Tests shall be performed in accordance with ASTM D 2573.

This method is commonly used for measuring shear strength in soft clays and organic deposits. It should not be used in stiff and hard clays. Results can be affected by the presence of gravel, shells, roots, or sand layers. Shear strength may be overestimated in highly plastic clays and a correction factor should be applied.

Figure 4-25 Vane Shear Test Equipment



4.9.8. Pocket Penetrometer

Used for obtaining the shear strength of cohesive, non-gravely soils on field exploration or construction sites. Commercial penetrometers are available which read unconfined compressive strength directly. The tool is used as an aid to obtaining uniform classification of soils. It does not replace other field tests or laboratory tests.

4.9.9. Torvane Shear Device

Used for obtaining rapid approximations of shear strength of cohesive, non-gravelly soils on field exploration. Can be used on ends of Shelby tubes, penetration samples, block samples from test pits or sides of test pits. The device is used in uniform soils and does not replace laboratory tests.

4.9.10. Percolation Test

The percolation test is used to ascertain the vertical percolation rate of unsaturated soil, i.e., the rate at which the water moves through near surface soils.

The percolation test method most commonly used, unless there are specified local requirements, is the test developed by the Robert A. Taft Sanitary Engineering Centre[98]. This test consists of digging a 4 to 12 inch (100 to 300 mm) diameter hole to the stratum for which information is required, cleaning and backfilling the bottom with coarse sand or gravel, filling the hole with water and providing a soaking period of sufficient length to achieve saturation. During the soaking period, water is added as necessary to prevent loss of all water. The percolation rate is then obtained by filling the hole to a prescribed water level (usually 6 inches) and measuring the drop in water level over a set time (usually 30 minutes). The times required for soaking and for measuring the percolation rate vary with the soil type; local practice should be consulted for specific requirements. Test holes are often kept filled with water for at least four hours; preferably overnight, before the test is conducted. In soils that swell, the soaking period should be at least 24 hours to obtain valid test results.

Results of this test are generally used in evaluating site suitability for septic system drainage fields.

4.9.11. Infiltration Test

The infiltration rate of a soil is the maximum rate at which water can enter the soil from the surface under specified conditions. The most common test in Florida uses a double-ring infiltrometer. Two open cylinders, approximately 20 inch (500 mm) high and 12 to 24 inch (300 to 600 mm) in diameter, are driven concentrically into the ground. The outer ring is driven to a depth of about 6 inch (150 mm), the inner ring to a depth of 2 to 4 inch (50 and 100 mm). Both are partially filled with water. As the water filtrates into the soil, measured volumes are added to keep the water levels constant. The volumes of water added to the inner ring and to the annular space during a specific time interval, equivalent to the amounts, which have infiltrated the soil. These are converted into infiltration rates, expressed in units of length per unit time, usually inches (millimetres) per hour. The infiltration rate is taken as the maximum infiltration velocity occurring over a period of several hours. In the case of differing velocities for the inner ring and the annular space, the maximum velocity from the inner ring should be used. The time required to run the test is dependent upon soil type. Tests shall be performed in accordance with ASTM D 3385.

Drainage engineers in evaluating runoff, ditch or swale infiltration use information from this test.

[98] HUD, Public Health Service Health Manual of Septic Tank Practice, NTIS PB 21822

4.9.12. Permeability Tests

Permeability, also known as hydraulic conductivity, is the measure of the rate of flow of water through soils, usually measured when the soil is saturated. Field permeability tests measure the coefficient of permeability (hydraulic conductivity) of in-place materials. The area and length factors are often combined in a "shape factor" or "conductivity coefficient." Measurement of permeability is highly sensitive to both natural and test conditions. The difficulties inherent in field permeability testing require that great care be taken to minimize sources of error and to correctly interpret, and compensate for, deviations from ideal test conditions.

Permeability differs from infiltration or percolation rates in that permeability values are corrected for the hydraulic boundary conditions, including the hydraulic gradient, and thus is representative of a specific soil property.

Many types of field permeability tests can be performed. In geotechnical exploration, equilibrium tests are the most common. These include constant and variable head gravity tests and pressure (Packer) tests conducted in single borings. In a few geotechnical investigations, and commonly in water resource or environmental studies, non-equilibrium "aquifer" or "pump" tests are conducted (a well is pumped at a constant rate for an extended period of time). See Figure 4-26 for analysis of observations and Table 4-14 for computation of permeability from variable head tests.

Figure 4-26 Analysis of Permeability by Variable Head Tests

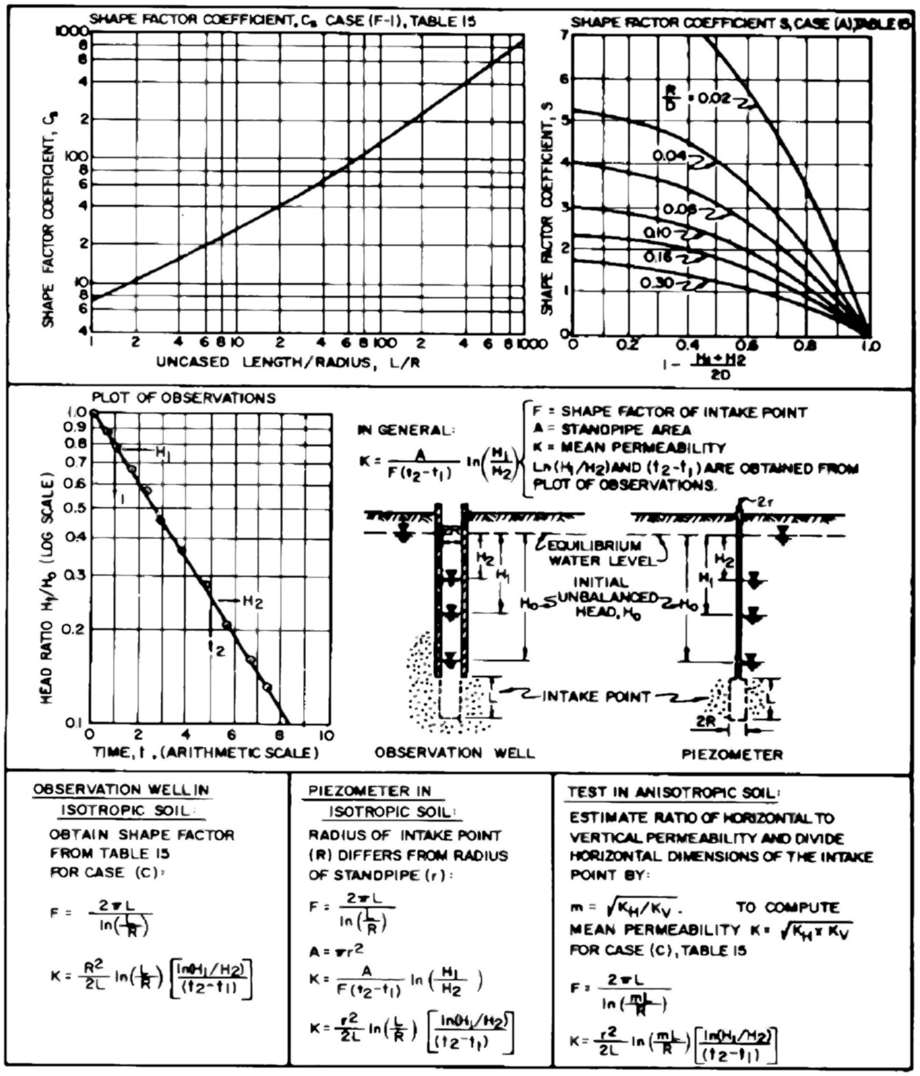
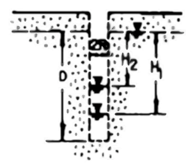
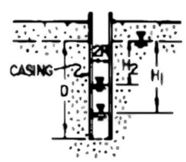
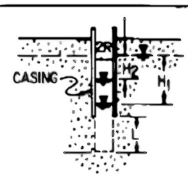
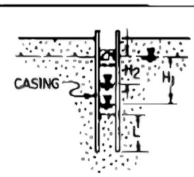
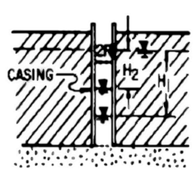
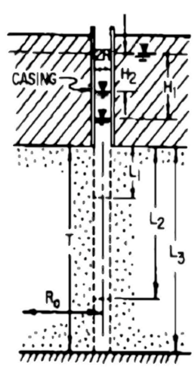


Table 4-14 Shape Factors for Computation of Permeability from Variable Head Tests

	CONDITION	DIAGRAM	SHAPE FACTOR, F	PERMEABILITY, K BY VARIABLE HEAD TEST	APPLICABILITY
OBSERVATION WELL OR PIEZOMETER IN SATURATED ISOTROPIC STRATUM OF INFINITE DEPTH				(FOR OBSERVATION WELL OF CONSTANT CROSS SECTION)	
	(A) UNCASED HOLE		$F = 16\pi DSR$	$K = \frac{R}{16DS} \times \frac{(H_2 - H_1)}{(t_2 - t_1)}$ FOR $\frac{D}{R} < 50$	SIMPLEST METHOD FOR PERMEABILITY DETERMINATION. NOT APPLICABLE IN STRATIFIED SOILS. FOR VALUES OF S, SEE FIGURE 13.
	(B) CASED HOLE, SOIL FLUSH WITH BOTTOM.		$F = \frac{11R}{2}$	$K = \frac{2\pi R}{11(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$ FOR $6'' \leq D \leq 60''$	USED FOR PERMEABILITY DETERMINATION AT SHALLOW DEPTHS BELOW THE WATER TABLE. MAY YIELD UNRELIABLE RESULTS IN FALLING HEAD TEST WITH SILTING OF BOTTOM OF HOLE.
	(C) CASED HOLE, UNCASED OR PERFORATED EXTENSION OF LENGTH "L".		$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$	$K = \frac{R^2}{2L(t_2 - t_1)} \ln\left(\frac{L}{R}\right) \ln\left(\frac{H_1}{H_2}\right)$ FOR $\frac{L}{R} > 8$	USED FOR PERMEABILITY DETERMINATIONS AT GREATER DEPTHS BELOW WATER TABLE.
	(D) CASED HOLE, COLUMN OF SOIL INSIDE CASING TO HEIGHT "L"		$F = \frac{11\pi R^2}{2\pi R + 11L}$	$K = \frac{2\pi R + 11L}{11(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	PRINCIPAL USE IS FOR PERMEABILITY IN VERTICAL DIRECTION IN ANISOTROPIC SOILS.
OBSERVATION WELL OR PIEZOMETER IN AQUIFER WITH IMPERVIOUS UPPER LAYER				(FOR OBSERVATION WELL OF CONSTANT CROSS SECTION)	
	(E) CASED HOLE, OPENING FLUSH WITH UPPER BOUNDARY OF AQUIFER OF INFINITE DEPTH.		$F = 4R$	$K = \frac{\pi R}{4(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	USED FOR PERMEABILITY DETERMINATION WHEN SURFACE IMPERVIOUS LAYER IS RELATIVELY THIN. MAY YIELD UNRELIABLE RESULTS IN FALLING HEAD TEST WITH SILTING OF BOTTOM OF HOLE.
	(F) CASED HOLE, UNCASED OR PERFORATED EXTENSION INTO AQUIFER OF FINITE THICKNESS: (1) $\frac{L_1}{T} \leq 0.2$ (2) $0.2 < \frac{L_2}{T} < 0.85$ (3) $\frac{L_3}{T} = 1.00$ NOTE: R_0 EQUALS EFFECTIVE RADIUS TO SOURCE AT CONSTANT HEAD.		(1) $F = C_s R$	$K = \frac{\pi R}{C_s(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	USED FOR PERMEABILITY DETERMINATIONS AT DEPTHS GREATER THAN ABOUT 5FT. FOR VALUES OF C_s, SEE FIGURE 13.
			(2) $F = \frac{2\pi L_2}{\ln(L_2/R)}$	$K = \frac{R^2 \ln\left(\frac{L_2}{R}\right)}{2L_2(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$ FOR $\frac{L}{R} \geq 8$	USED FOR PERMEABILITY DETERMINATIONS AT GREATER DEPTHS AND FOR FINE GRAINED SOILS USING POROUS INTAKE POINT OF PIEZOMETER.
			(3) $F = \frac{2\pi L_3}{\ln\left(\frac{R_0}{R}\right)}$	$K = \frac{R^2 \ln\left(\frac{R_0}{R}\right)}{2L_3(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	ASSUME VALUE OF $\frac{R_0}{R}$ = 200 FOR ESTIMATES UNLESS OBSERVATION WELLS ARE MADE TO DETERMINE ACTUAL VALUE OF R_0.

4.9.12.1. Seepage Test

These tests can be constant head, falling head, or rising head tests. The constant head test is the most generally applicable and, in areas of unknown permeability, should be performed first. The falling head and rising head methods are used in areas where the permeability is low enough to permit accurate measurement of the change in water level. Results are used in the design of exfiltration systems. The more commonly performed tests include:

4.9.12.1.1. Constant Head Test

This is the most generally applicable permeability test. It may be difficult to perform in materials of either very high or very low permeability since the flow of water may be difficult to maintain or to measure.

4.9.12.1.2. Rising Head Test

In a saturated zone with sufficiently permeable materials, this test is more accurate than a constant or a falling head test. Plugging of the pores by fines or by air bubbles is less apt to occur in a rising head test. In an unsaturated zone, the rising head test is inapplicable.

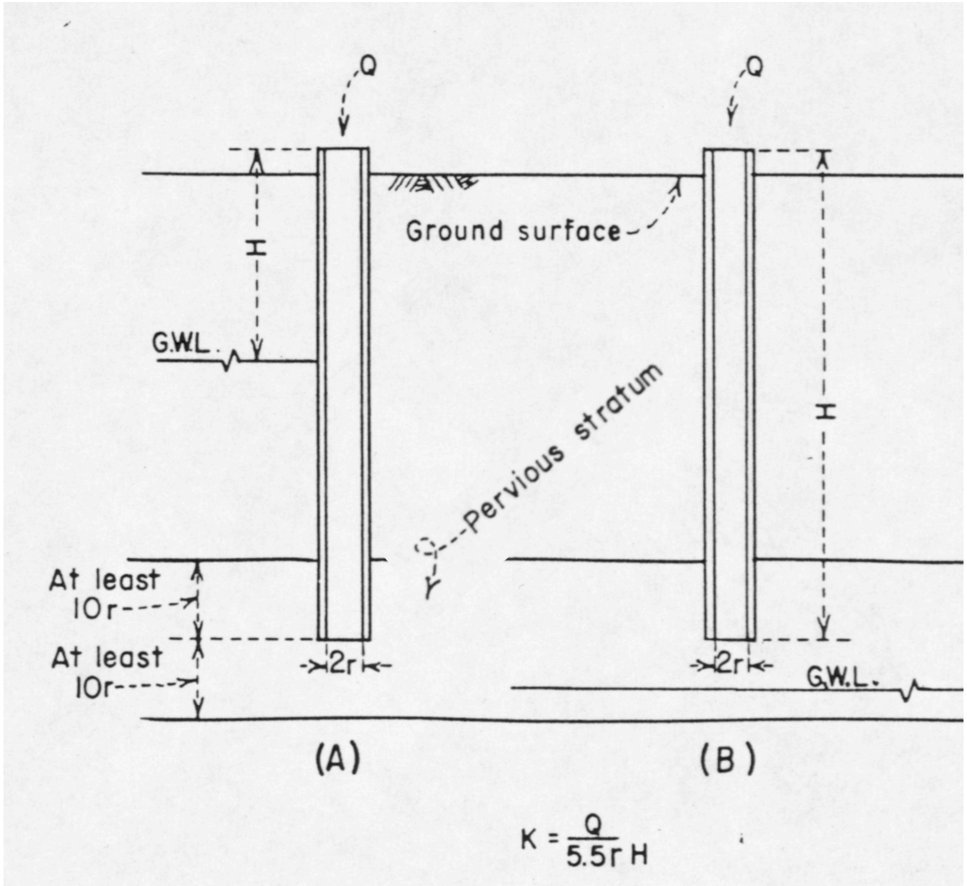
4.9.12.1.3. Falling Head Test

In zones where the flow rates are very high or very low, this test may be more accurate than a constant head test. In an area of unknown permeability, the constant head test should be attempted before a falling head test.

4.9.12.1.4. Open-End Borehole Test

This test can be conducted as either a constant head or a variable head test. An open-end pipe or casing is installed to the desired depth within a uniform soil. The pipe/casing is then cleaned out flush with the bottom of the pipe/casing while the hole is kept filled with water. Clear water is added through a metering system to maintain gravity flow at a constant head until measurements indicate a steady-state flow is achieved. The permeability is calculated from the rate of steady-state flow, height of head and radius of pipe (see Figure 4-27).

Figure 4-27 Open-end Borehole Test[99]



4.9.12.1.5. Exfiltration Test

This test is performed as a constant head test. A 7" (175 mm) diameter (or larger) hole is augered to a standard depth of 10 feet (3 meters). Approximately 0.125 ft^3 (0.0035 m^3) of 0.5" (13 mm) diameter gravel is poured to the bottom of the hole to prevent scour. A 6" (150 mm) diameter (or larger), 9-feet (2.75 meter) long casing which is perforated with 0.5-inch (12.7 mm) holes on 2" (51 mm) centers over the bottom 6.0 feet (1.8 m) is then lowered into the hole. Water is added and the amount required to maintain a constant water level over specified time intervals is recorded.

4.9.12.2. Pumping Test

Pumping tests are used in large-scale investigations to more accurately measure the permeability of an area. The results are used in the design of dewatering systems and other situations where the effects of a change in the water table are to be analysed.

Pumping tests require a test hole and at least one observation well, although several observation wells at varying distances from the test hole are preferable. As water is pumped from the test hole, water level changes within each observation well and corresponding times is recorded. Pumping is continued at a constant rate until the water level within each observation well remains constant. Permeability

[99] US Department of Interior, Bureau of Reclamation, Earth Manual, US Government Printing Office, Washington, D.C, 1994.

calculations are made based on the rate of pumping, the measured draw down, and the configuration of the test hole and observation wells. Refer to ASTM D 4050.

4.9.12.3. Gravity and Pressure Tests

In a boring, gravity and pressure tests are appropriate. The segment of the boring tested is usually 5 to 10 feet, but may be larger. A large number of tests must be conducted to achieve an overall view of the seepage characteristics of the materials. The zone of influence of each test is small, usually a few feet or perhaps a few inches. These methods can detect changes in permeability over relatively short distances in a boring, which conventional pump or aquifer tests cannot. Exploration boring (as opposed to "well") methods are therefore useful in geotechnical investigations where inhomogeneity and anisotropy may be of critical importance. Results from pressure tests using packers in fractured rock may provide an indication of static heads, inflow capacities, and fracture deformation characteristics, but conventional interpretation methods do not give a true permeability in the sense that it is measured in porous media.

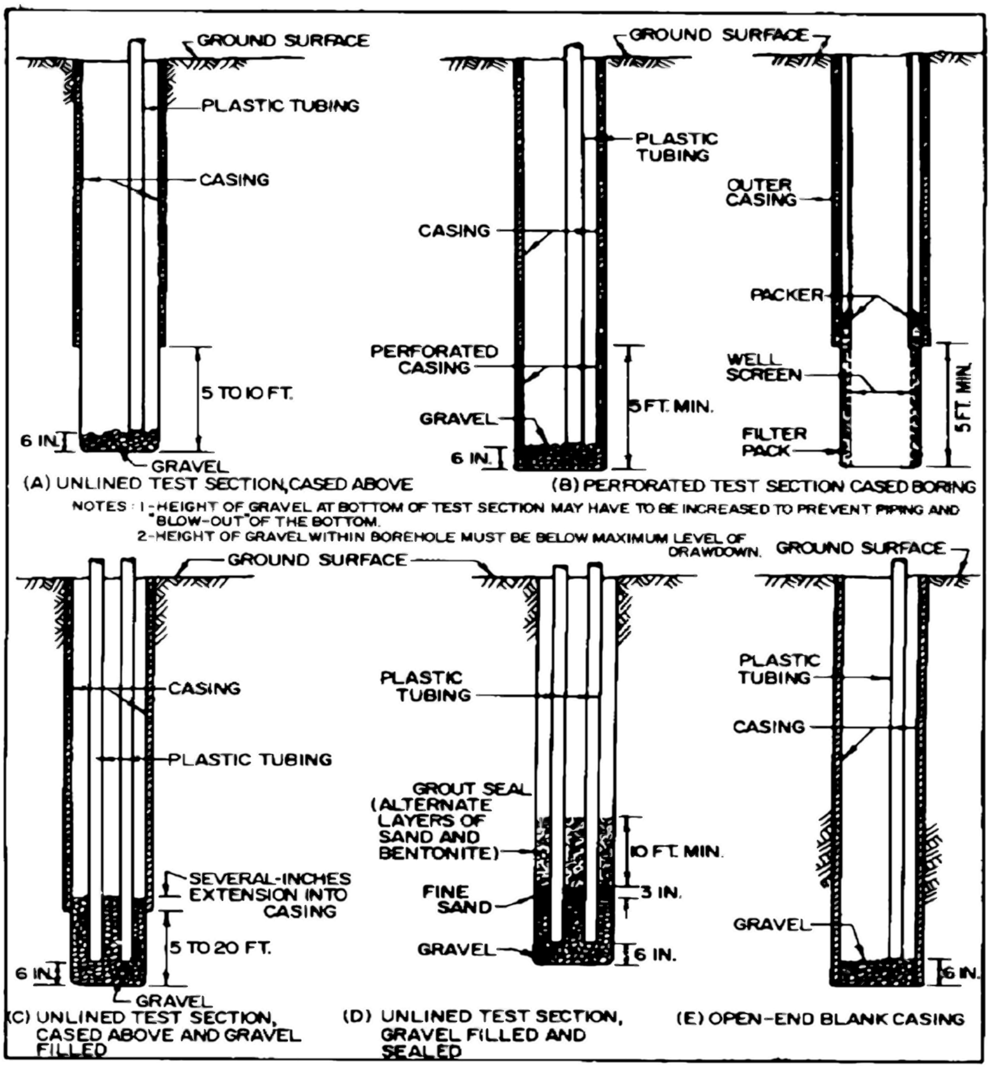
4.9.12.4. Factors Affecting Tests

The following five physical characteristics influence the performance and applicability of permeability tests:

- Position of the water level,
- Type of material - rock or soil,
- Depth of the test zone,
- Permeability of the test zone, and
- Heterogeneity and anistropy of the test zone.

To account for these it is necessary to isolate the test zone. Methods for doing so are shown in Figure 4-28.

Figure 4-28 Test Zone Isolation Methods



4.9.13. Environmental Corrosion Tests

These tests are carried out on soil and water at structure locations, on structural backfill materials and on subsurface materials along drainage alignments to determine the corrosion classification to be considered during design. For structures, materials are classified as slightly, moderately, or extremely aggressive, depending on their pH, resistivity, chloride content, and sulfate content. For roadway drainage systems, test results for each stratum are presented for use in determining alternate culvert materials. Testing shall be performed in the field and/or the laboratory according to the standard procedures listed in Table 3-2.

4.9.14. Grout Plug Pull-out Test

This test is performed when the design of drilled shafts in rock is anticipated. However, the values obtained from this test should be used carefully. Research has indicated that the results are overly conservative.

A 4” (100 mm) diameter (minimum) by 30” (760 mm) long core hole is made to the desired depth in rock. A high strength steel bar with a bottom plate and a reinforcing cage over the length to be grouted is lowered to the bottom of the hole. Sufficient grout is poured into the hole to form a grout plug approximately 2 feet (600 mm) long. After curing, a centre hole jack is used to incrementally apply a tension load to the plug with the intent of inducing a shear failure at the grout - limestone interface. The plug is extracted, the failure surface examined, and the actual plug dimensions measured.

The ultimate shear strength of the grout-limestone interface is determined by dividing the failure load by the plug perimeter area. This value can be used to estimate the skin friction of the rock-socketed portion of the drilled shaft.

4.10. Groundwater Measurements

The groundwater level should be measured at the depth at which water is first encountered as well as at the level at which it stabilizes after drilling. If necessary, the boring should be kept open with perforated casing until stabilization occurs. On many projects, seasonal groundwater fluctuation is of importance and converting the borings to standpipe piezometers can make long-term measurements.

4.10.1. *Piezometers*

The pore water pressure should be checked often during embankment construction. After the fill is in place, it can be monitored at a decreasing frequency. The data should be plotted (as pressure or feet (meters) of head) as a function of time. A good practice is to plot pore water pressure, settlement, and embankment elevation on the same time-scale plot for comparison.

4.10.2. Typical Installation

Piezometers are used to measure the amount of water pressure within the saturated pores of a specific zone of soil. The critical levels to which the excess pore pressure will increase prior to failure can be estimated during design. During construction, the piezometers are used to monitor the pore water pressure build-up. After construction, the dissipation of the excess pore water pressure over time is used as a guide to consolidation rate. Thus, piezometers can be used to control the rate of fill placement during embankment construction over soft soils.

The three basic components of a piezometer installation are:

1. Tip. A piezometer tip consisting of a perforated section, well screen, porous tube, or other similar feature and, in fine-grained or unstable materials, a surrounding zone of filter sands;
2. Standpipe. Watertight standpipe or measurement conduit, of the smallest practical diameter, attached to the tip and extending to the surface of the ground;
3. Seals. A seal or seals consisting of cement grout, bentonite slurry, or other similarly impermeable material placed between the standpipe and the boring walls to isolate the zone to be monitored.

The vertical location, i.e., depth and elevation of each item must be accurately measured and recorded.

Piezometers should be placed prior to construction in the strata in which problems are most likely to develop. If the problem stratum is more than 10 feet (3 m) thick, more than one piezometer should be placed, at varying depths. The junction box should be located at a convenient location but outside the construction area if possible.

4.10.3. Piezometer Types

All systems, except the open well, have a porous filter element that is placed in the ground. The most common types used for groundwater measurements are described in Table 4-15.

Table 4-15 Groundwater or Piezometric Level Monitoring Devices

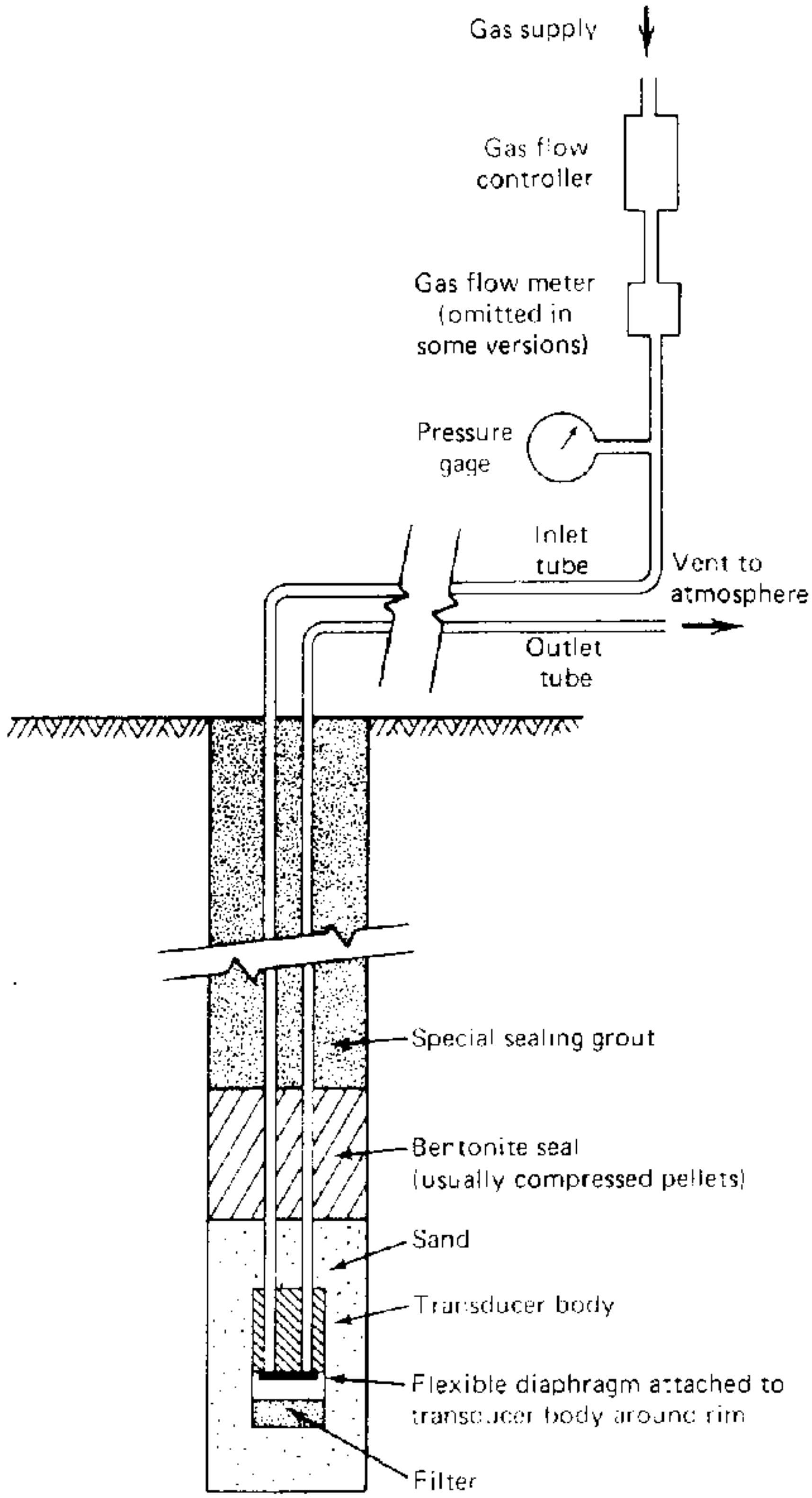
Instrument	*Advantages*	*Disadvantages*

Standpipe piezometer or wellpoint.	Simple. Reliable. Long experience record. No elaborate terminal point needed.	Slow response time. Freezing problems.
Pneumatic piezometer.	Level of terminal independent of tip level. Rapid response.	Must prevent humid air from entering tubing.
Electric piezometer	Level of terminal independent of tip level. Rapid response. High sensitivity. Suitable for automatic readout.	Expensive. Temperature correction may be required. Errors due to zero drift can arise.

4.10.3.1. Pneumatic Piezometer

The simplest type of piezometer is an open standpipe extending through the fill, but its use may be limited by the response time lag inherent in all open standpipe piezometers. More useful is the pneumatic piezometer, which consists of a sensor body with a flexible diaphragm attached. This sensor is installed in the ground and attached to a junction box with twin tubes. The junction box outlet can be connected to a readout unit. Pressurized gas is applied to the inlet tube. As the applied gas pressure equals and then exceeds the pore water pressure, the diaphragm deflects allowing gas to vent through the outlet tube. The gas supply is then turned off and the diaphragm returns to its original position when the pressure in the inlet tube equals the pore water pressure. This pressure is recorded (see Figure 4-29). Refer to AASHTO T 252. Also available are vibrating wire and electrical resistance piezometers, but use of the electrical resistance piezometers is generally limited to applications where dynamic responses are to be measured.

Figure 4-29 Typical Pneumatic Piezometer[104]



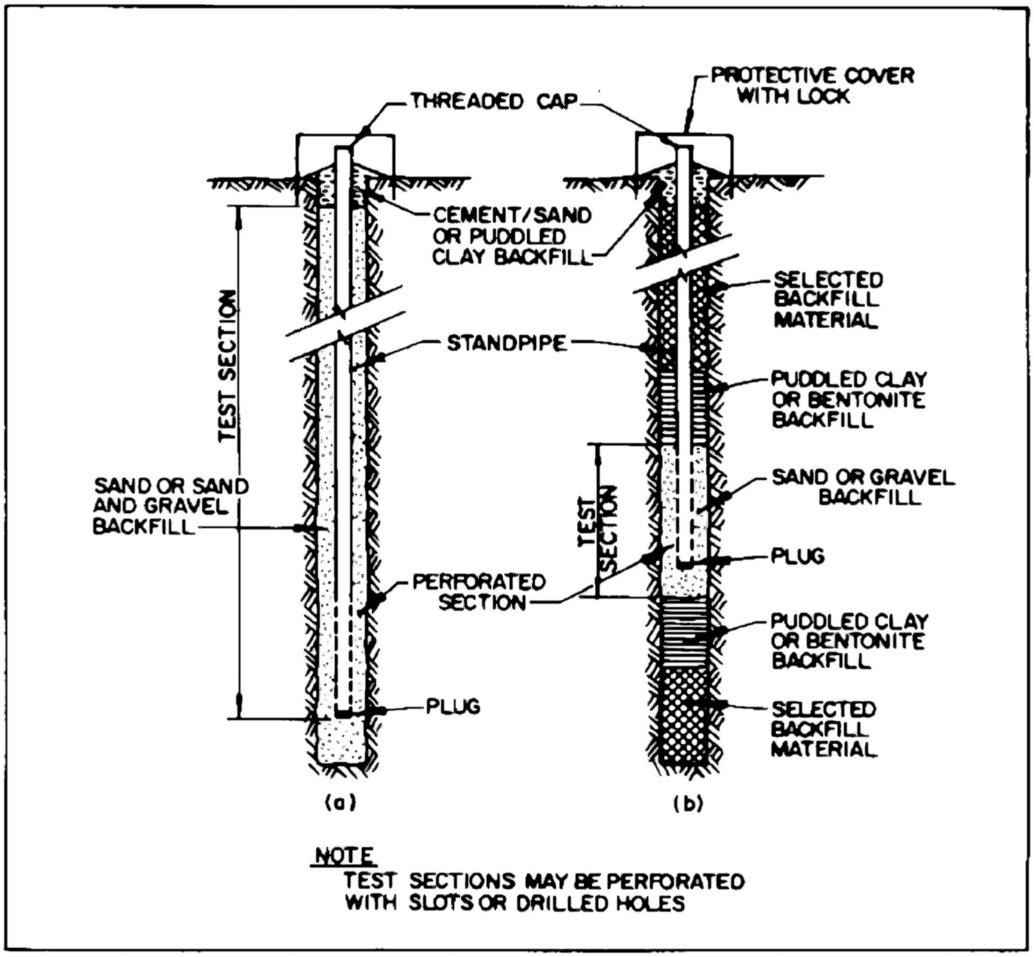
4.10.3.2. Open Well

The most common groundwater recording technique is to measure water level in an open boring as shown in Figure 4-30(a). A disadvantage is that different layers of soil may be under different hydrostatic pressures and therefore the groundwater level recorded may be inaccurate and misleading. Thus, this system is useful only for relatively homogeneous deposits.

4.10.3.3. Open Standpipe Piezometer

Most of the disadvantages of the open borehole can be overcome by installing an open standpipe piezometer in the borehole as shown in Figure 4-30(b). This system is effective in isolating substrata of interest.

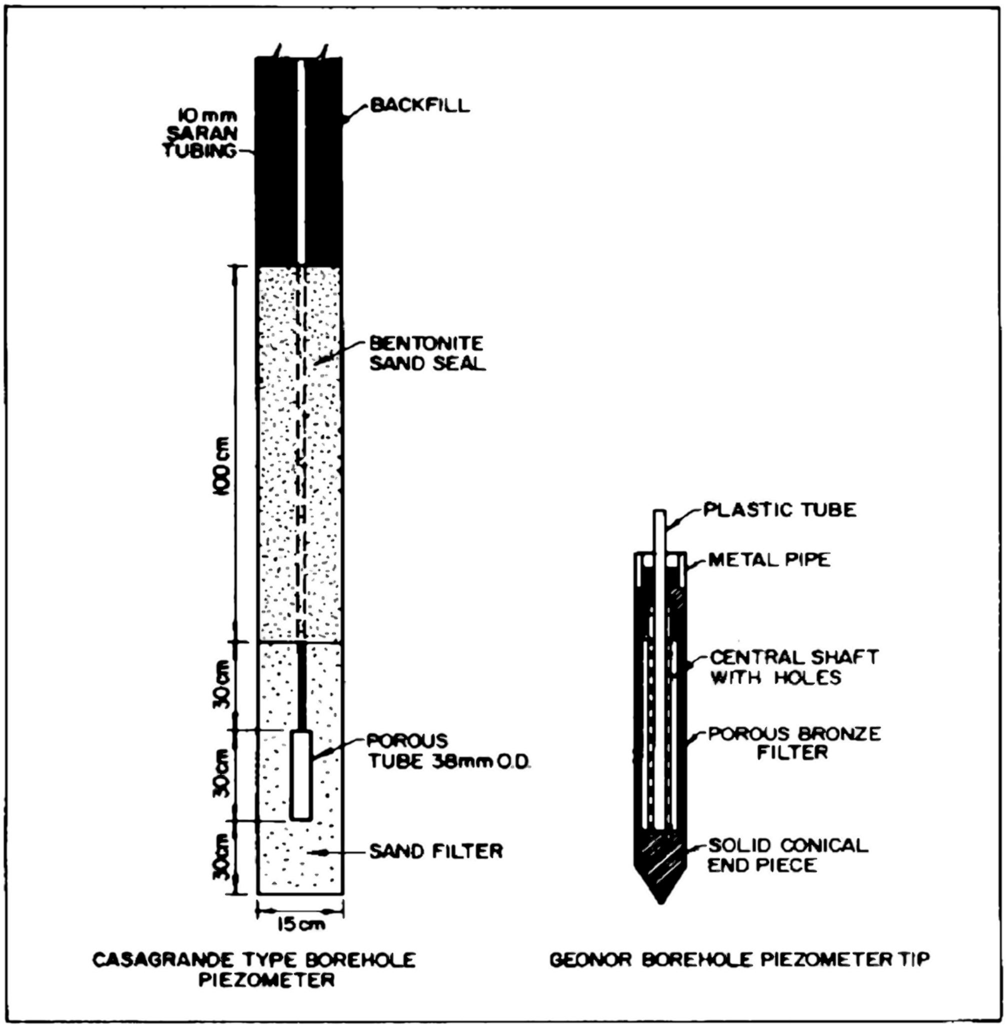
Figure 4-30 Open Standpipe Piezometers



4.10.3.4. Porous Element Piezometer

As shown in Figure 4-31, a porous element is connected to the riser pipe that is of small diameter to reduce the equalization time. The most common tip is the nonmetallic ceramic stone (Casagrande Type). The ceramic tip is subject to damage and for that reason porous metal tips or other tips of the same dimension are now available. Pores are about 50 microns size, so that the tip can be used in direct contact with fine-grained soils.

Figure 4-31 Porous Element Piezometers



4.10.3.5. Other Types

Other piezometers used for special investigations include electrical, air pneumatic, oil pneumatic and water pressure types.

4.10.4. Multiple Installations

Several piezometers may be installed in a single boring with an impervious seal separating the measuring zones. However, if measurements are needed in zones with 10 feet or less of vertical separation, it is generally best to install piezometers in separate borings.

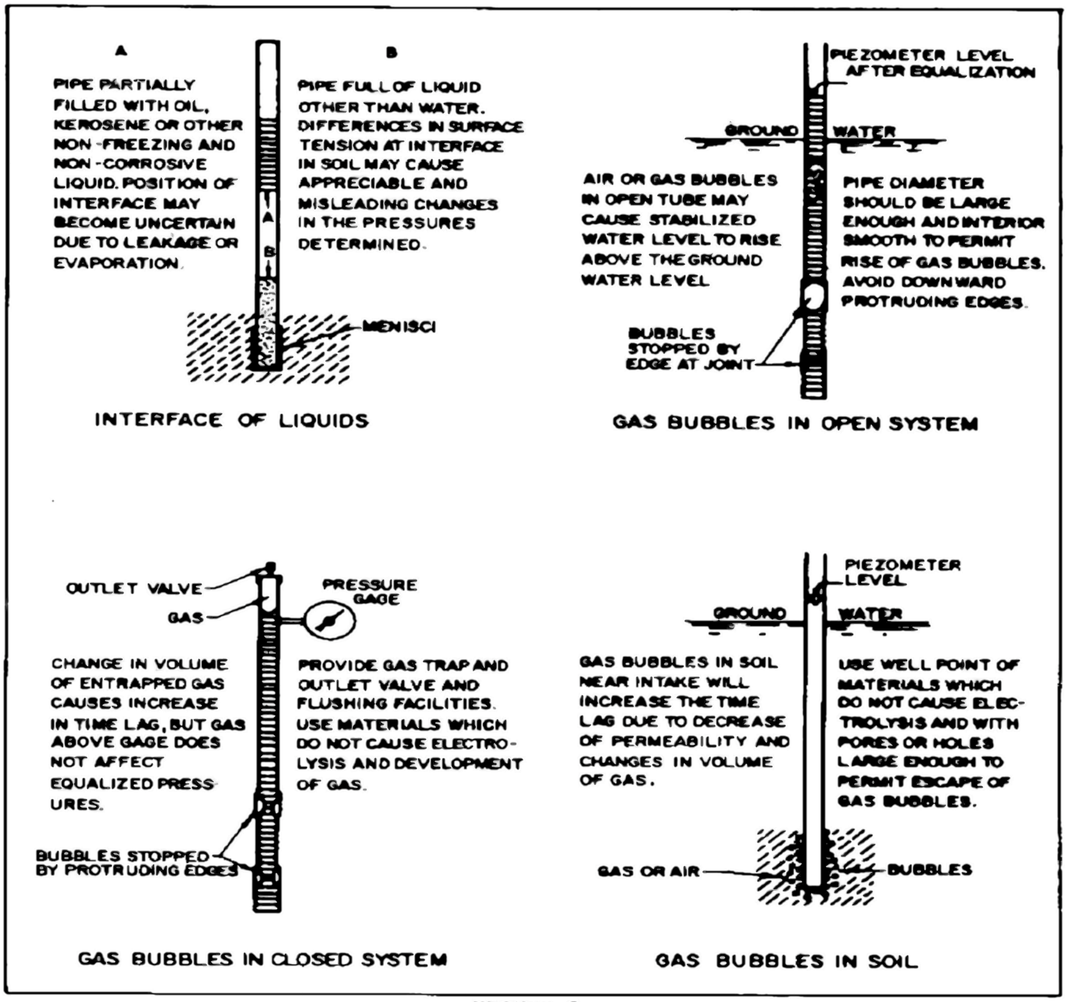
4.10.5. Measurements

Water levels can be measured to within 0.5 inch, using several devices, including the plumb bob, cloth or metal surveyors' tapes coated with chalk, or commercially available electrical indicators for use in small tubes.

4.10.6. Sources of Error

Major sources of error are due to gas bubbles and tube blockage. Some are shown in Figure 4-32. The magnitude of errors can be controlled by proper piezometer selection, installation, and de-airing techniques.

Figure 4-32 Sources of Error and Corrective Methods in Groundwater Pressure Measurements



4.11. Measurement of Soil and Rock Properties in situ

A great number of tools and methods have been devised for measuring *in situ* engineering properties of soil and rock. The most common tools, the split spoon sampler and the cone penetrometer, have been previously discussed. This section describes other methods commonly used in exploration programs or during construction control.

4.11.1. In-Place Density

In-place soil density can be measured on the surface by displacement methods to obtain volume and weight, and by nuclear density meters. Density at depth can be measured only in certain soils by the drive cylinder (sampling tube) method.

- Displacement Methods. Direct methods of measuring include sand displacement and water balloon methods.[100] The sand displacement and water balloon methods are the most widely used methods because of their applicability to a wide range of material types and good performance. The sand displacement method (ASTM Standard D1556, Density of Soil in Place by the Sand Cone Method) is the most frequently used surface test and is the reference test for all other methods. A procedure for the water or rubber balloon method is given in ASTM Standard D2167, Density of Soil in Place by the Rubber Balloon Method.
- Drive-Cylinder Method. The drive cylinder (ASTM Standard D2937, Density of Soil in Place by the Drive-Cylinder Method) is useful for obtaining subsurface samples from which the density can be ascertained, but it is limited to moist, cohesive soils containing little or no gravel and moist, fine sands that exhibit apparent cohesion.
- Nuclear Moisture-Density Method. Use ASTM Standard D2922, Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth). Before nuclear density methods are used on the job, results must be compared with density and water contents determined by displacement methods. Based on this comparison, correlations may be required to the factory calibration curves or a new calibration curve may have to be developed.[101]

4.11.2. Detection of Combustible Gases

Methane and other combustible gases may be present in areas near sanitary landfills, or at sites near or over peat bogs, marshes and swamp deposits. Commercially available indicators are used to detect combustible gases or vapours and sample air in borings above the water table. The detector indicates the concentration of gases as a percentage of the lower explosive limit from 0 to 100 on the gage. The lower explosive limit represents the leanest mixture that will explode when ignited. The gage scale between 60% and 100% is coloured red to indicate very dangerous concentrations. If concentrations are judged serious, all possibilities of spark generation (e.g., pile driving, especially mandrel driven shells) should be precluded, and a venting system or vented crawl space should be considered. The system could be constructed as follows:

a) Place a 6” layer of crushed stone (3/4” size) below the floor slab; a polyethylene vapour barrier should overlie the crushed stone.
b) Install 4” diameter perforated pipe in the stone layer below the slab; the top of the pipe should be immediately below the bottom of the slab.
c) The pipes should be located such that gas rising vertically to the underside of the floor slab does not have to travel more than 25 feet laterally through the stone to reach a pipe.
d) The pipes can be connected to a single, non-perforated pipe of 6” diameter, and vented to the atmosphere at roof level[102].

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[100] ASTM STP 523, Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils, 1972

[101] Safety regulations pertaining to the use of nuclear gages are contained in U.S. Corps of Engineers, Radiological Safety, ER385-1-80.

[102] Further details on gas detection and venting can be found in Noble, G., Sanitary Landfill Design Handbook, Technamic Publishing Co., Westport, CT., 1976, and United States Environmental Protection Agency (EPA), Process Design Manual, Municipal Sludge Landfills, EPA-625 11-78-010, SW 705, 1978.

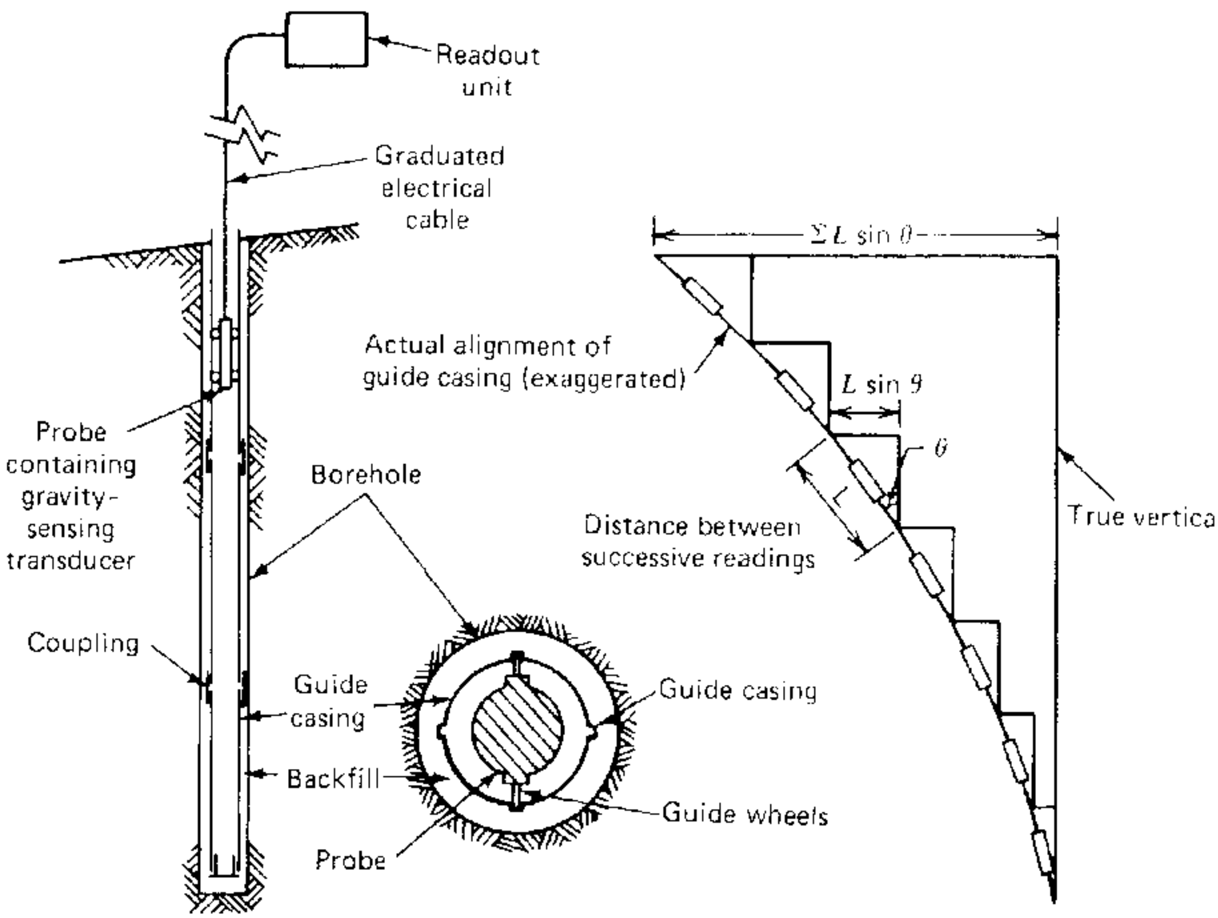
4.12. Field Instrumentation

Field instrumentation is used to measure load and displacement and to monitor changes during and after construction. This allows verification of design assumptions and performance monitoring, which could indicate the need for implementation of contingency plans or design changes.[103]

4.12.1. *Inclinometers (Slope Indicators)*

These instruments are used to monitor embankment or cut slope stability. An inclinometer casing consists of a grooved metal or plastic tube that is installed in a borehole. The bottom of the tube must be in rock or dense material, which will not experience any movement, thereby achieving a stable point of fixity. A sensing probe is lowered down the tube and deflection of the tube is measured. Successive readings can be plotted to provide the engineer with information about the rate of subsurface movement with depth (see Figure 4-33). Refer to ASTM D 4622 (AASHTO T 254).

Figure 4-33 Principle of Inclinometer Operation[104]



[103] For additional guidance on planning and performing geotechnical monitoring see Dunnicliff, C.J., Geotechnical Instrumentation for Monitoring Field Performance, National Cooperative Highway Research Program, Synthesis of Highway Practice, Transportation Research Board, to be published 1981, and Dunnicliff, C.J., Equipment for Field Deformation Measurements, Proceedings of the Fourth Panamerican Conference, SMFE, Vol. II, San Juan, Puerto Rico, January 1973.

[104] Dunnicliff, John, Geotechnical Instrumentation for Monitoring Field Performance, Wiley-Interscience, New York, 1993.

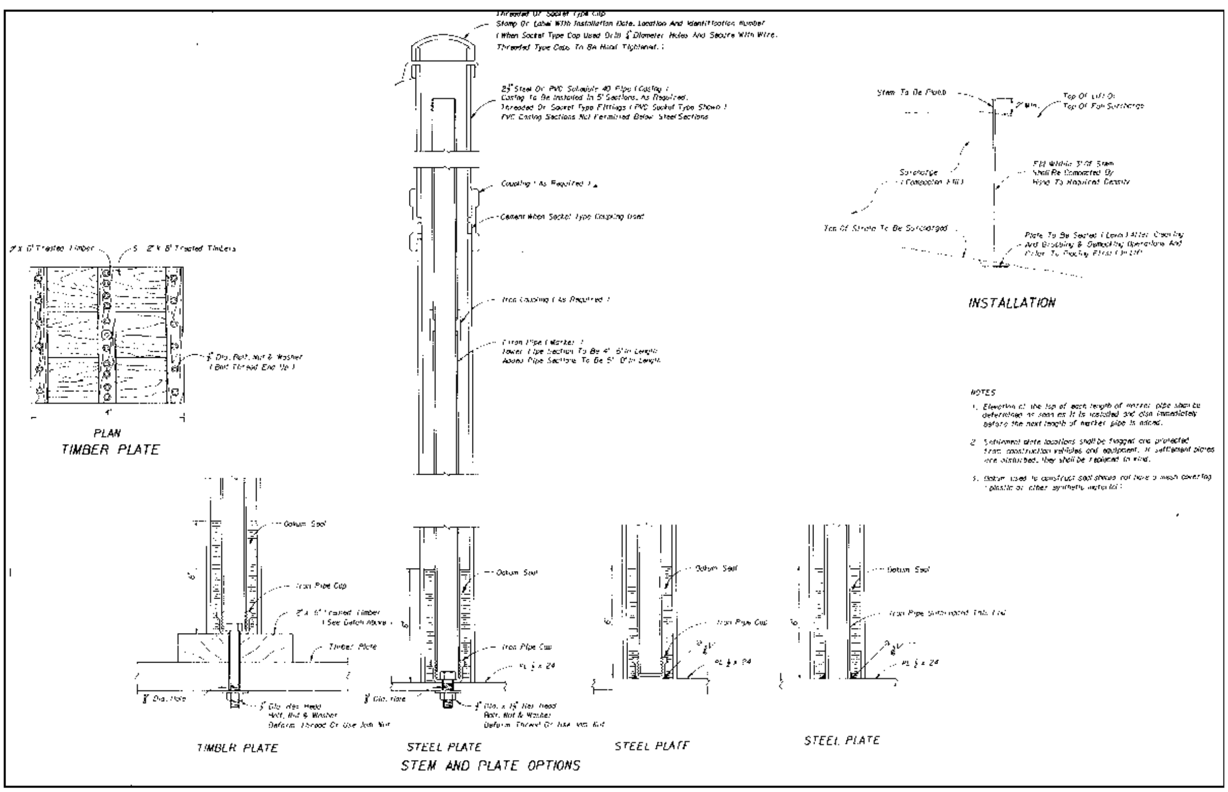
Care must be taken when installing the casing so that spiralling of the casing does not occur because of poor installation techniques. This will result in the orientation of the grooves at depth being different than at the surface. This can be checked with a spiral-checking sensor, and the data adjusted with most new computerized data reduction routines. Also, the space between the borehole wall and the casing should be backfilled with a firm grout, sand, or gravel. For installation in highly compressible soils, use of telescoping couplings should be used to prevent damage of the casing.

To monitor embankment construction, inclinometers should be placed at or near the toes of slopes of high-fill embankments where slope stability or lateral squeeze is considered a potential problem. The casing should penetrate the strata in which problems are anticipated. Readings should be taken often during embankment construction. Fill operations should be halted if any sudden increase in movement rate is detected.

4.12.2. *Settlement Indicators*

Settlement instruments simply record the amount and rate of the settlement under a load; they are most commonly used on projects with high fill embankments where significant settlement is predicted. The simplest form is the settlement platform or plate, which consists of a square wooden platform or steel plate placed on the existing ground surface prior to embankment construction. A reference rod and protecting pipe are attached to the platform. As fill operations progress, additional rods and pipes are added. (See Figure 4-34). Settlement is evaluated by periodically measuring the elevation of the top of the reference rod. Benchmarks used for reference datum shall be known to be stable and remote from all possible vertical movement. It is recommended to use multiple benchmarks and to survey between them at regular intervals.

Figure 4-34 Typical Settlement Platform Design



Settlement platforms should be placed at those points under the embankment where maximum settlement is predicted. On large jobs two or more per embankment are common. The platform elevation must be recorded before embankment construction begins. This is imperative, as all future readings will be compared with the initial reading. Readings thereafter should be taken periodically until the embankment and surcharge (if any) are completed, then at a reduced frequency. The settlement data should be plotted as a function of time. The geotechnical engineer should analyse this data to determine when the rate of settlement has slowed sufficiently for construction to continue.

A disadvantage to the use of settlement platforms is the potential for damage to the reference rod by construction equipment. An alternative to settlement plates is probe extensometers in which a probe lowered down a compressible pipe can identify points along the pipe either mechanically or electrically, and thereby, the distance between these points can be determined. Surveying at the top of the pipe needs to be performed to get absolute elevations if the pipe is not seated into an incompressible soil layer. This method allows a settlement profile within the compressible soil layer to be obtained. Care must be taken during installation and grouting the pipe in the borehole so that it is allowed to settle in the same fashion as the surrounding soil.

4.12.3. *Tiltmeters*

Tiltmeters measure the inclination of discreet parts of structures from the norm. They are most commonly used to monitor tilting of bridge abutments and decks or retaining walls, and can also be used to monitor rotational failure surfaces in landslides. Types range from a simple plumb line to more sophisticated equipment.

4.12.4. *Monitoring Wells*

A monitoring or observation well is used to monitor groundwater levels or to provide ready access for sampling to detect groundwater contamination. It consists of a perforated section of pipe or well point attached to a riser pipe, installed in a sand-filled borehole.

Monitoring wells should also be installed in conjunction with piezometers to provide a base reference necessary for calculating changes in pore pressure. The monitoring well should be placed in an unimpacted area of construction to reflect the true static water table elevation.

4.12.5. *Vibration Monitoring*

It is sometimes desirable to monitor the ground vibrations induced by blasting, pile driving, construction equipment, or traffic. This is especially critical when construction is in close proximity to sensitive structures or equipment, which may become damaged if subjected to excessive vibration.

A vibration-monitoring unit typically consists of a recording control unit, one or more geophones, and connecting cables. Sound sensors to detect noise levels are also available. Geophones and/or sound sensors are placed at locations where data on vibration levels is desired. Peak particle velocities, principle frequencies, peak sound pressure levels, and actual waveforms can be recorded. Results are compared with pre-established vibration-limiting criteria, which are based on structure conditions, equipment sensitivity, or human tolerance.

4.12.6. Special Instrumentation

Earth pressure cells and strain gauges fall into this category of special instruments. They are not normally used in monitoring construction projects but only in research and special projects. These instruments require experienced personnel to install and interpret the data.

4.13. Soil Profile Development

The mark of successfully accomplishing a subsurface investigation is the ability to draw a soil profile of the project site complete with soil types and necessary design properties. The soil profile is a visual display of subsurface conditions as interpreted from all foregoing explorations and testing. Uncertainties in its development usually indicate additional exploration or testing is required.

In the optimum situation, the soil profile is developed in stages. First, a rough profile is established from the drillers logs by the soil engineer. The object is to discover any obvious gaps or question marks while the drill crew is still at the site so that additional work can be performed immediately. Once a crew has left the site, a delay of months may occur before their schedule permits reoccupying the site (not to mention the additional cost to the State, and aggravation to the drill crew to reoccupy a remote site). The drilling inspector or crew chief should be required to call the soils engineer when progression of the last scheduled boring has begun, to request further instructions for supplemental borings.

When all borings are completed and laboratory visuals and moisture content data received, the initial soil profile should be revised. Definite soil layer boundaries and accurate soil descriptions should be established for soil deposits. Too often, the engineer will overcomplicate a simple profile by noting minute variations between adjacent soil samples. This can be avoided by:

1. Reviewing the geological site history, i.e., if the soil map denotes a lakebed deposit overlying a glacial till deposit, do not subdivide the lakebed deposit because adjacent samples have differing amounts of silt and clay. Realize before breaking down the soil profile that probably only two layers exist and variations are to be expected within each. Important variations such as average thickness of silt and clay varves can be noted adjacent to the visual description of the layer.
2. Remembering that the soil samples examined are only a minute portion of the soil underlying the site and must be considered in relation to not only adjacent samples, but also adjacent borings.

A few simple rules should be followed at this stage to properly interpret the available data:

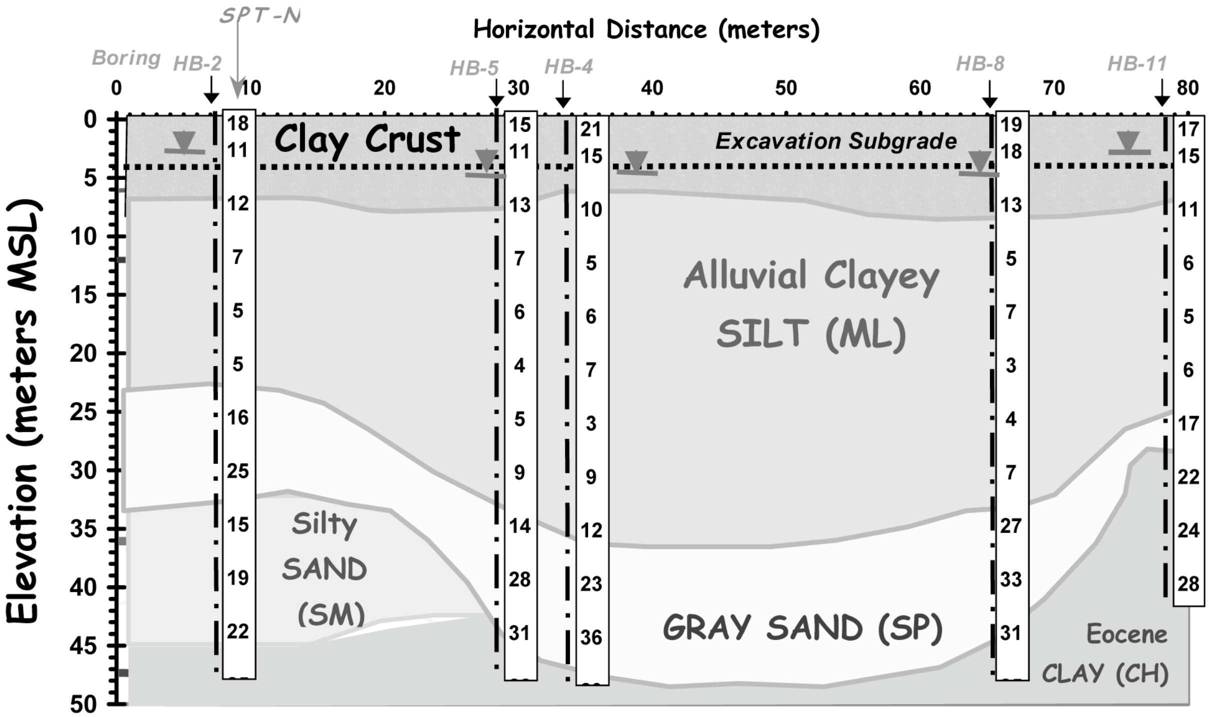
- Review the U.S.D.A. County Soil Map and determine major deposits expected at the site.
- Examine the subsurface log containing standard penetration test results and the laboratory visual descriptions with accompanying moisture contents.
- Personally review representative soil samples to check laboratory identification and to calibrate your interpretation with the laboratory technicians who performed the visual.
- Establish rational mechanics for drawing the soil profile.
    - Use a vertical scale of 1” equals 10 feet or 20 feet; generally, any smaller scale tends to squeeze data and prevent interpretation.
    - Use a horizontal scale equal to the vertical, if possible, to simulate actual relationships. However, the total length should be kept within 36 inches to permit review in a single glance.

When the soil layer boundaries and descriptions have been established, determine the extent and details of laboratory testing. Consolidation and triaxial tests are expensive. Do not casually read the drillers log and randomly select certain samples for testing. Plan the test program intelligently from the soil profile. Identify major soil deposits and assign appropriate tests.

The final soil profile is the foundation engineer's best interpretation of all available subsurface data. The final soil profile should include the average physical properties of the soil deposits, i.e., unit weight, shear strength, etc., in addition to a visual description of each deposit observed water level, and special items such as boulders or artesian pressure. Successful development of this subsurface profile will allow the foundation engineer to advance his design with confidence.

A sample profile is shown in Figure 4-35.

Figure 4-35 Sample Soil Profile



§ 5. Distribution of Stresses

This section covers the analysis of stress conditions at a point, stresses beneath structures and embankments, and empirical methods for estimating loads on buried pipes, conduits, shafts, and tunnels.

Stresses in earth masses are analyzed using two basic and different assumptions. One assumes elastic conditions, and the other assumes full mobilization of shear strength (plastic equilibrium). Elastic solutions apply to problems for which shear failure is unlikely. If the safety factor against shear failure exceeds about 3, stresses are roughly equal to values computed from elastic theory. Plastic equilibrium applies in problems of foundation or slope stability and wall pressures where shear strength may be completely mobilized.

5.1. Overview of Stresses in Soils

5.1.1. Types of Soil Stresses

The normal stress at any orientation in saturated soil mass equals the sum of two elements: (a) pore water pressure carried by fluid in void spaces, and (b) effective stress carried by the grain skeleton of the soil. Analysis of a soil system (e.g., settlement, stability analyses) is performed in terms of either total stresses or effective stresses. The choice between the two analysis methods is governed by the properties of the surrounding soils, pore water behavior, and the method of loading.

5.1.1.1. Pore Water Pressure

Pore water pressure may consist of:

- Hydrostatic pressure,
- Capillary pressure,
- Seepage, or
- Pressure resulting from applied loads to soils that drain slowly.

5.1.1.2. Total Stress

The overburden pressure plus any applied loads produces the total stress at any point.

5.1.1.3. Effective Stress

Effective stress equals the total stress minus the pore water pressure, or the total force in the soil grains divided by the gross cross-sectional area over which the force acts. Under an applied load, the total stress in a saturated soil sample is composed of the intergranular stress and pore pressure (neutral stress). As the pore water has no strength and is incompressible, only the intergranular stress is effective in resisting shear or limiting compression of the soil sample. Therefore, the intergranular contact stress is called the effective stress.

In general, soil deposits below the water table should be considered saturated. The total stress at that depth may be found by

Equation 5-1: $\sigma'_{oz} = \gamma z - u_w$

Where

- σ'_{oz} = initial vertical effective stress at depth z, ksf
- γ = saturated unit weight of soil mass at depth z, ksf

- z = depth, ft
- u_w = pore water pressure, ksf.

u_w usually is the hydrostatic pressure $\gamma_w z_w$ where γ_w is the unit weight of water, 0.062 ksf, and z_w is the height of a column of water above depth z. γz is the total overburden pressure $\sigma_{oz.}$

The consolidation process demonstrates the very important principle of effective stress. When pore water drains from soil during consolidation, the area of contact between soil grains increases, which increases the level of effective stress. In practice, stage construction of embankments is used to permit increase of effective stress in the foundation soil before subsequent fill load is added. In such operations effective stress increase is frequently monitored with piezometers to insure the next stage of embankment can be safely placed.

5.1.1.4. Overburden Pressure

The laboratory testing required to solve soil-related problems involves simulating conditions naturally existing in the ground. Soils existing distances below ground are affected by the weight of the soil above that depth. The influence of this weight, known generally as overburden pressure, causes a state of stress to exist that is unique at that depth, for that soil. When a soil sample is removed from the ground, that state of stress is relieved as all weight confining the sample has been removed. In testing, it is important to re-establish the *in situ* stress conditions and to study changes in soil properties when additional stresses representing the expected design loading are applied. As previously mentioned, the effective stress (grain to grain contact) is the controlling factor in shear and consolidation. Therefore, the designer should try to duplicate the effective stress condition during most testing.

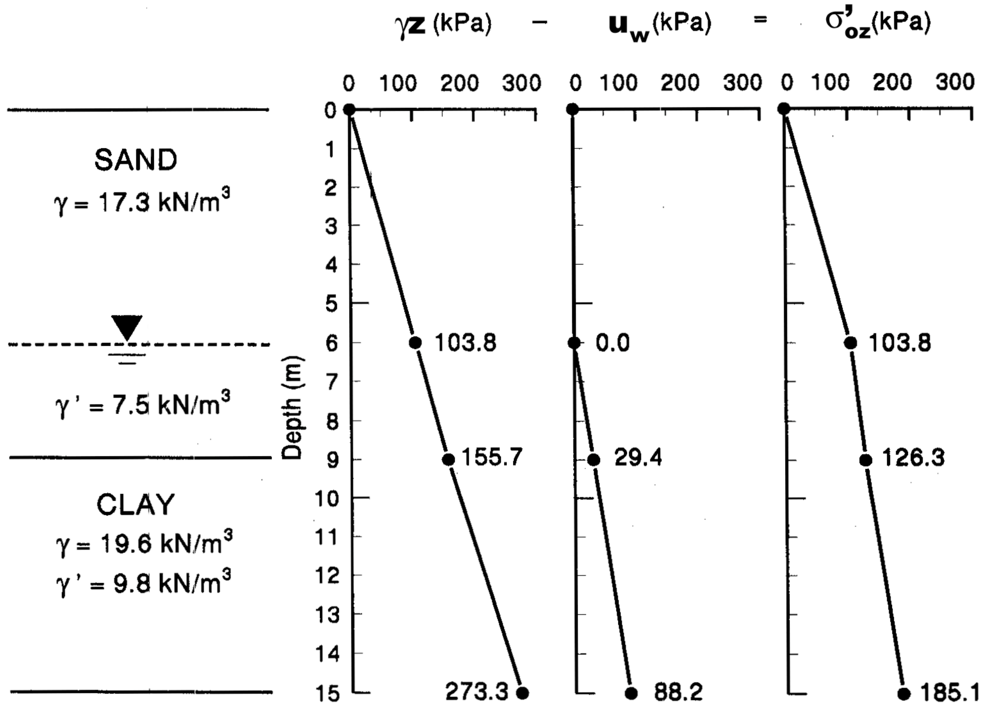
The test stresses are estimated from either the total or effective overburden pressure. The engineers' first task is determining the total and effective overburden pressure variation with depth. This relatively simple job involves determining the average total unit weight (density) for each soil layer in the soil profile, and determining the depth of the water table. Unit weight may be accurately determined from density tests on undisturbed samples or estimated from standard penetration values and soil visuals. Water table is usually recorded on boring logs. The total overburden pressure (P_T) is found by multiplying the total unit weights of each soil layer by the layer thickness and continuously summing the results with depth.

The effective overburden pressure (P_o) at any depth is determined by accumulating the weights of all layers above that depth as follows:

- Soils above the water table - multiply the total unit weight by the thickness of each respective soil layer above the desired depth.
- Soils below the water table - reduce the total unit weights by the weight of water (62.4 pcf or 9.81 kN/m^3), i.e., use effective unit weights and multiply by the thickness of each respective soil layer between the water table and the desired depth.

A plot of effective overburden pressure versus depth is called a P_o diagram and is used throughout all aspects of foundation testing and analysis. An example of this is shown in Figure 5-1.

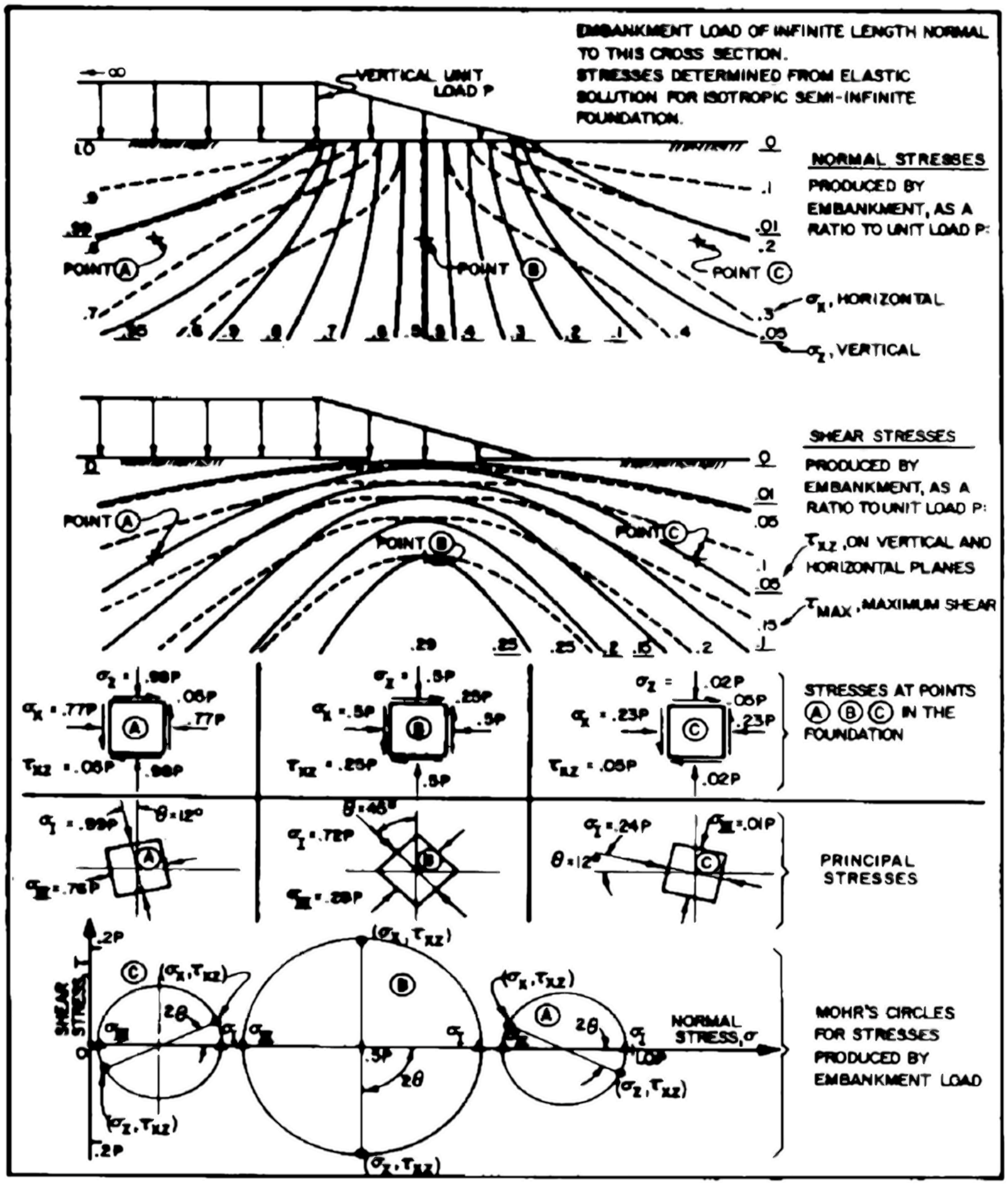
Figure 5-1 Overburden (P_o) Diagram Example



5.1.2. Mohr's Circle of Stress

If normal and shear stresses at one orientation on an element in an earth mass are known, stresses at all other orientations may be determined from Mohr's circle. Examples of stress transformation are given in Figure 5-2. The use of Mohr's circle for plastic equilibrium is illustrated by analysis of triaxial shear test results.

Figure 5-2 Examples of Stress Conditions at a Point



5.1.3. Preconsolidation Stress

The preconsolidation stress or maximum effective past pressure σ'_p experienced by a foundation soil is a principle factor in determining the magnitude of settlement of a structure supported by the soil. σ'_p is the maximum effective stress to which the *in situ* soil has been consolidated by a previous loading; it is the boundary between recompression and virgin consolidation, which are described in 7.3.

Pressures applied to the foundation soil that exceed the maximum past pressure experienced by the soil may cause substantial settlement. Structures should be designed, if practical, with loads that maintain soil pressures less than the maximum past pressure.

5.1.3.1. Geological evidence of a preconsolidation stress

Stresses are induced in the soil mass by history such as surcharge loads from soil later eroded away by natural causes, lowering of the groundwater table and desiccation by drying from the surface.

a) Temporary groundwater levels and lakes may have existed causing loads and overconsolidation compared with existing effective stresses.

b) Desiccation of surface soil, particularly cyclic desiccation due to repeated wetting and drying, creates significant microscale stresses which in turn cause significant preconsolidation effects. Such effects include low void ratios as well as fissures and fractures, high density, high strength and high maximum past pressures measured in consolidation tests.

c) A high preconsolidation stress may be anticipated if

Equation 5-2: $$\frac{N}{15\sigma_{oz}} > \frac{1}{4}$$

where N is the blow count from standard penetration test (SPT) results and σ_{oz} (tons/square foot or tsf) is the total overburden pressure at depth z.

5.1.3.2. Evaluation from maximum past thickness

Local geologic records and publications when available should be reviewed to estimate the maximum past thickness of geologic formations from erosion events, when and amount of material removed, glacial loads, and crustal tilt.

a) The minimum local depth can sometimes be determined from transvalley geologic profiles if carried sufficiently into abutment areas to be beyond the influence of valley erosion effects.

b) The maximum past pressure at a point in an *in situ* soil is estimated by multiplying the unit wet soil weight (approximately 0.06 tsf) by the total estimated past thickness of the overlying soil at that point.

Results of the cone penetration test (CPT) may be used to evaluate the thickness of overburden soil removed by erosion if the cone tip resistance q_c increases linearly with depth[105]. The line of q_c versus depth is extrapolated back above the existing surface of the soil to the elevation where q_c is zero assuming the original cohesion is zero.

The difference in elevation where q_c is zero and the existing elevation is the depth of overburden removed by erosion. This depth times the unit wet weight γ is the total maximum past pressure σ_p. The cohesion for many clays is not zero, but contributes to a q_c approaching one tsf. Extrapolating the line above the existing ground surface to $q_c = 1$ tsf produces a more conservative depth of overburden clay soil. This latter estimate of overburden depth is recommended.

5.1.3.3. Evaluation from overconsolidation ratio

The preconsolidation stress σ'_p may be evaluated from the overconsolidation ratio

[105] Schmertmann, J. H. 1978. Guidelines for Cone Penetration Test Performance and Design, Report No. FHWA-TS-78-209. Available from US Department of Transportation, Federal Highway Administration, Office of Research and Development, Washington, DC 20590. Refer to Figure 7.

Equation 5-3: $OCR = \dfrac{\sigma'_p}{\sigma'_{oz}}$

Where

- OCR = overconsolidation ratio
- σ'_p = preconsolidation stress
- σ'_{oz} =the effective vertical overburden pressure at depth z.

a) The overconsolidation ratio has been related empirically with the coefficient of earth pressure at rest K_o, $\sigma'_{hz}/\sigma'_{oz}$, and the plasticity index PI. σ'_{hz} is the effective horizontal pressure at rest at depth z. Normally consolidated soil is defined as soil with OCR = 1. Overconsolidated soil is defined as soil with OCR > 1.

b) The results of pressuremeter tests (PMT) may be used to evaluate the effective horizontal earth pressure σ'_{hz}. K_o may be evaluated if the effective vertical overburden pressure σ'_{oz} at depth z is known and the OCR estimated as above.

5.1.3.4. Laboratory tests

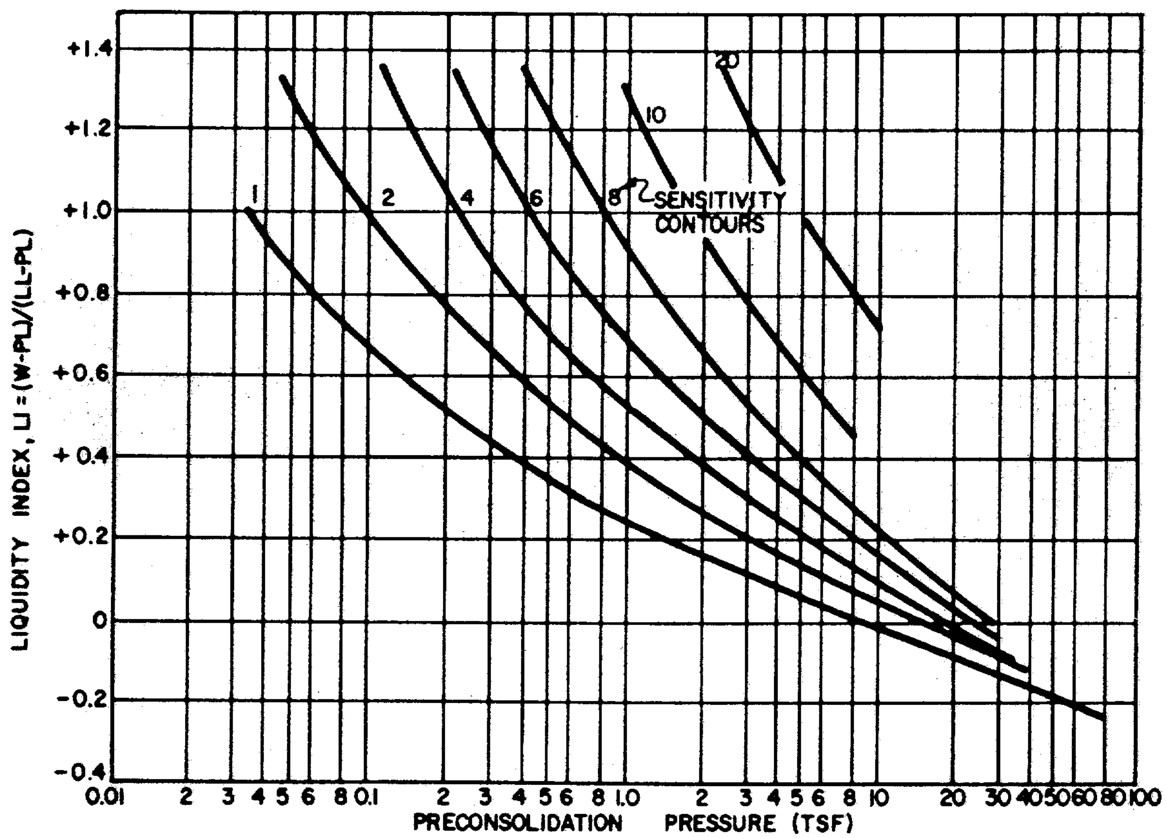
The preconsolidation stress may be calculated from results of consolidation tests on undisturbed soil specimens, 7.7.7.1.

5.1.3.5. Empirical Relationships

A high preconsolidation stress may be anticipated if the natural water content is near the plastic limit PL or below or if $C_u/\sigma_{oz} > 0.3$ where C_u is the undrained shear strength.

Figure 5-3 shows a relationship between preconsolidation pressure and liquidity index. For samples with natural moisture at the liquid limit (liquidity index of 1), preconsolidation ranges between about 0.1 and 0.8 tsf depending on soil sensitivity. For natural moisture at the plastic limit (liquidity index equal to zero), preconsolidation ranges from about 12 to 25 tsf.

Figure 5-3 Preconsolidation Stress as a function of Liquidity Index LI and clay sensitivity (ratio of undisturbed to remolded shear strength)



W = WATER CONTENT, PERCENT
PL = PLASTIC LIMIT, PERCENT
LL = LIQUID LIMIT, PERCENT

Alternately estimate the preconsolidation pressure from:

Equation 5-4: $$\sigma'_p = \frac{C_u}{0.11 + 0.0037PI} = \left(\frac{q_u}{2}\right)\frac{1}{0.11 + 0.0037PI}$$

Where

- q_u = unconfined compressive strength
- σ'_p = preconsolidation stress
- C_u = undrained shear strength
- PI = the soil plasticity index, percent

5.2. Stresses beneath Structures and Embankments

5.2.1. Effect of Foundation Stiffness

The distribution of stress in soil depends on the contact pressure between the foundation and soil, which is a function of the relative stiffness K_R between the soil and the foundation[106]

Equation 5-5: $$K_R = \frac{E_f\left(1-\nu_s^2\right)}{E_s}\left[\frac{D_f}{R}\right]^3$$

Where

- E_f = Young's modulus of foundation, tsf
- ν_s = Poisson's ratio of foundation soil
- E_s = Young's modulus of foundation soil, tsf
- D_f = thickness of foundation, ft
- R = radius of foundation, ft

A uniformly loaded flexible foundation where stiffness $K_R < 0.1$ causes a uniform contact pressure; whereas, a uniformly loaded rigid foundation where $K_R > 10$ causes a highly nonuniform contact pressure distribution, Figure 5-19.

5.2.1.1. Embankments

Earth embankments are flexible and normally in full contact with the supporting soil.

5.2.1.2. Foundations for structures

Foundations such as large mats and footings with sufficient stiffness ($K_R > 0.1$) may not always be in complete contact with the soil.

5.2.2. Limiting Contact Pressures

Contact pressures are limited to maximum pressures defined as the bearing capacity.

5.2.3. Other Factors Influencing Contact Pressure

The magnitude of loading, depth of applied loads, size, shape, and method of load application such as static or dynamic applied loads also influence the distribution of contact pressures.

5.2.4. Semi-Infinite, Elastic Foundations

5.2.4.1. Pressure Bulb of Stressed Soil

The pressure bulb is a common term that represents the volume of soil or zone below a foundation within which the foundation load induces appreciable stress. The stress level at a particular point of soil beneath a foundation may be estimated by the theory of elasticity.

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[106] Brown, P. T. 1969. "Numerical Analysis of Uniformly Loaded Circular Rafts on Deep Elastic Foundations," Geotechnique, Vol 19, pp 399-404, The Institution of Civil Engineers. Available from Thomas Telford Ltd., 1-7 Great George Street, Westminster, London, SW1P 3AA, England.

5.2.4.1.1. Applicability of the theory of elasticity

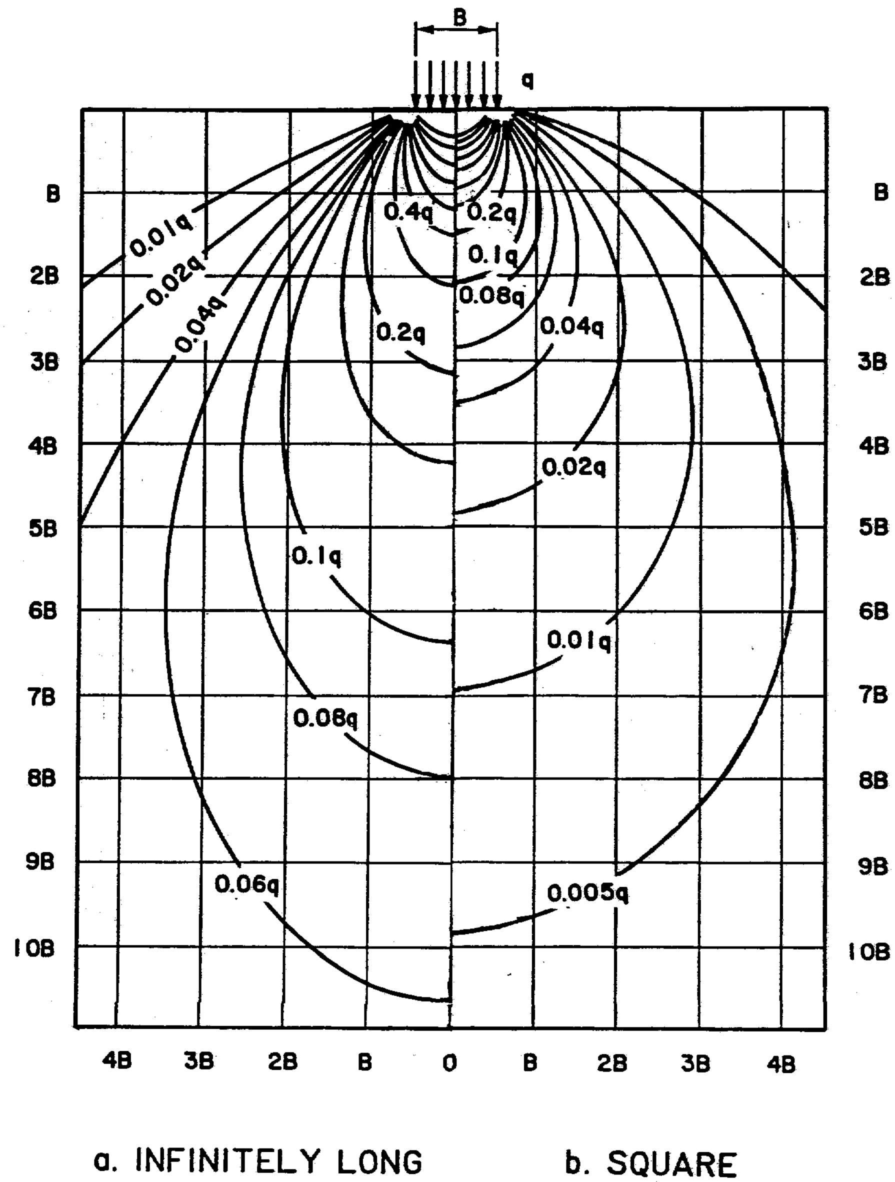
Earth masses and foundation boundary conditions correspond approximately with the theory of plasticity.

5.2.4.1.2. Stress distribution

Various laboratory, prototype, and full scale field tests of pressure cell measurements in response to applied surface loads on homogeneous soil show that the measured soil vertical stress distribution corresponds reasonably well to analytical models predicted by linear elastic analysis for similar boundary conditions.

a) The Boussinesq method is commonly used to estimate the stress distribution in soil. This distribution indicates that the stressed zone decreases toward the edge of the foundation and becomes negligible (less than 10% of the stress intensity) at depths of about 6 times the width of an infinite strip or 2 times the width of a square foundation, Figure 5-4.

Figure 5-4 Contours of equal vertical stress beneath a foundation in semi-infinite elastic solid by the Boussinesq solution



b) The recommended depth of analysis is at least twice the least width of the footing or mat foundation, four times the width of infinite strips or embankments, or the depth of incompressible soil, whichever comes first.

c) The distribution of vertical stress in material overlain by a much stiffer layer is more nearly determined by considering the entire mass as homogeneous rather than a layered elastic system.

5.2.4.2. Assumed Conditions

The following solutions assume the following conditions:

- ∞ Elasticity;

- Continuity;
- Static equilibrium;
- Completely flexible loads so that the pressures on the foundation surface are equal to the applied load intensity;
- Negligible shearing stresses between an embankment and its foundation.

For loads of infinite length or where the length is at least 5 times the width, the stress distribution can be considered plane strain, i.e., deformation occurs only in planes perpendicular to the long axis of the load. In this case, stresses depend only on direction and intensity of load and the location of points being investigated and are not affected by elastic properties.

5.2.4.3. Boussinesq Solution

The Boussinesq solution is based on the assumption of a weightless half space free of initial stress and deformation. The modulus of elasticity is assumed constant and the principle of linear superposition is assumed valid.

Stresses based on Boussinesq assumptions can be solved in one of two ways: 1) using equations, and b) using charts. Which solution is used depends upon the complexity of the problem and the availability of the chart and/or equation. In either case the stress at a given point in the soil based on Boussinesq assumptions is given by the equation

Equation 5-6: $\Delta\sigma_v = I_\sigma q$

Where

- $\Delta\sigma_v$ = Increase in vertical stress due to the load.
- I_σ = the influence factor, either from an equation for from a chart
- q = the bearing pressure of the foundation

Although Boussinesq theory can be used to compute both horizontal and shear stresses in the soil, except in special cases as noted below, most soils are not sufficiently elastic for these stresses to be meaningful.

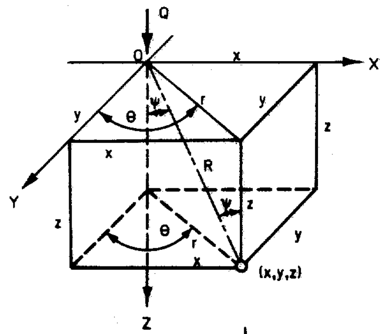
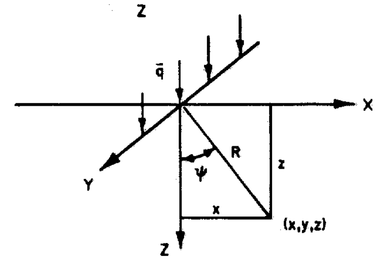
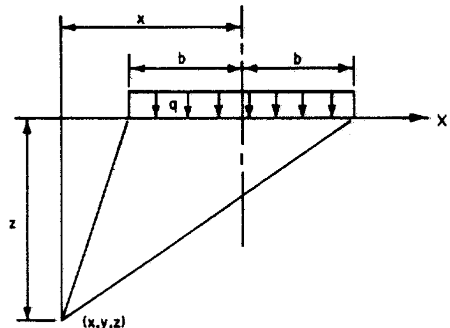
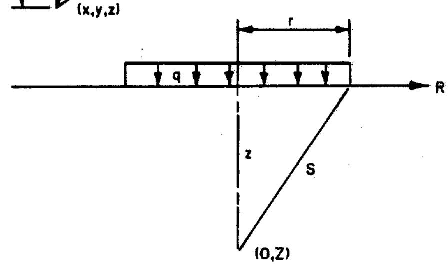
5.2.4.3.1. Equations for strip and area loads

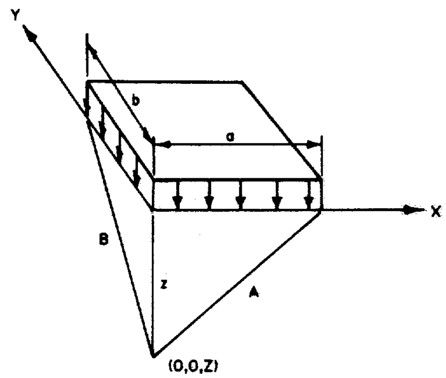
The Boussinesq solutions for increase in vertical stress $\Delta\sigma_z$ shown in Figure 5-5 for strip and area loads apply to elastic and nonplastic materials where deformations are continuous and unloading/reloading do not occur.[107]

[107] Refer to Lysmer, J. and Duncan, J. M. 1969. "Stress and Deflections in Foundations and Pavements," Berkeley, CA. Available from Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, CA 94720, for equations of stress distributions beneath other foundation shapes.

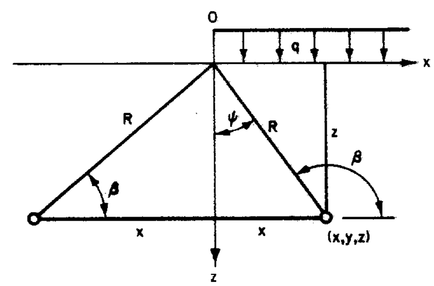
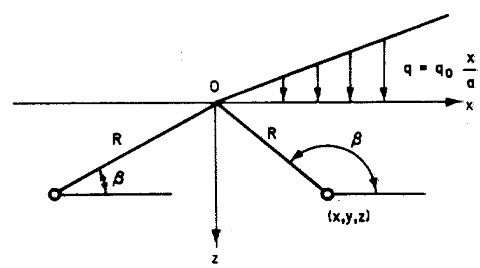
Figure 5-5 Boussinesq Solutions for Increases in Vertical Stresses $\Delta\sigma_v$ Beneath a Foundation

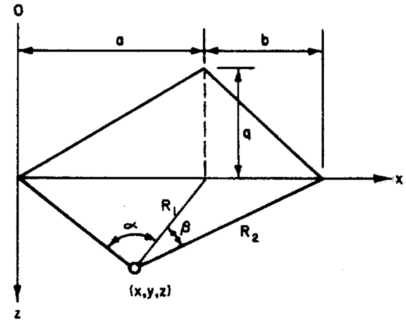
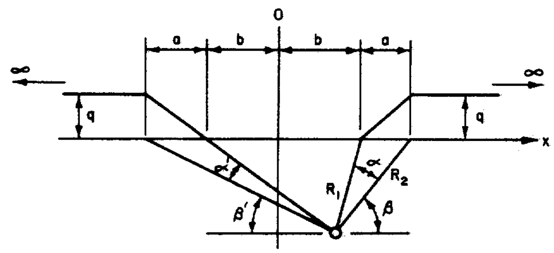
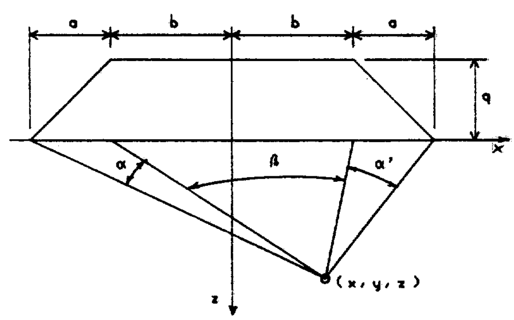
a. Uniform Loads

TYPE OF LOAD NORMAL TO SURFACE	EQUATION FOR $\Delta\sigma_z$	COORDINATE SYSTEM
POINT	$\frac{3Q}{2\pi R^2}\cos^3\psi$ Q = NORMAL LOAD, TONS $r^2 = x^2 + y^2$ $R^2 = r^2 + z^2$	
LINE	$\frac{2\bar{q}z^3}{\pi R^4}$ $\bar{q}$ = NORMAL LOAD, TONS/FT $R^2 = x^2 + z^2$	
STRIP	$\frac{q}{\pi}(\alpha + \sin\alpha\cos(\alpha + 2\beta))$ q = CONTACT PRESSURE, TSF $\alpha = \tan^{-1}\left(\frac{x+b}{z}\right) - \beta$, RADIANS $\beta = \tan^{-1}\left(\frac{x-b}{z}\right)$, RADIANS	
AREA (POINT UNDER CENTER CIRCULAR AREA)	$qr^2 \frac{(S^2 + 2z^2)}{2S^4}$ $S^2 = r^2 + z^2$ q = CONTACT PRESSURE, TSF	

TYPE OF LOAD NORMAL TO SURFACE	EQUATION FOR $\Delta\sigma_z$	COORDINATE SYSTEM
(POINT UNDER CORNER RECTANGULAR AREA)	$\frac{q}{2\pi}\left[\tan^{-1}\frac{ab}{zC} + \frac{abz}{C}\left(\frac{1}{A^2}+\frac{1}{B^2}\right)\right]$ $A^2 = a^2 + z^2$ $B^2 = b^2 + z^2$ $C = (a^2 + b^2 + z^2)^{1/2}$ q = CONTACT PRESSURE, TSF	

b. Nonuniform Strip Loads

TYPE OF LOAD NORMAL TO SURFACE	EQUATION FOR $\Delta\sigma_z$	COORDINATE SYSTEM
UNIFORM ON HALF INFINITE SPACE	$\frac{q}{\pi}\left(\beta + \frac{xz}{R^2}\right)$ q = CONTACT PRESSURE, TSF β = ANGLE IN RADIANS	
LINEAR INCREASE	$\frac{q_0}{\pi a}(x\beta + z)$	

TYPE OF LOAD NORMAL TO SURFACE	EQUATION FOR $\Delta\sigma_z$	COORDINATE SYSTEM
TRIANGULAR	$\frac{q}{\pi}\left[\frac{x}{a}\alpha + \frac{(a+b-x)}{b}\beta\right]$	
CONTRATRAPEZOIDAL	$\frac{q}{\pi a}\left[a(\beta+\beta') - b(\alpha+\alpha') + x(\alpha-\alpha')\right]$	
TRAPEZOIDAL	$\frac{q}{\pi a}\left[a(\alpha+\beta+\alpha')+b(\alpha+\alpha')+x(\alpha-\alpha')\right]$	

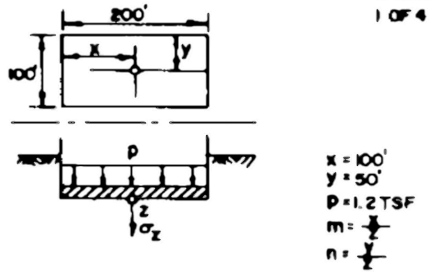
5.2.4.3.2. Graphical solutions

Due to the complexity of the Boussinesq equations, chart solutions for the influence factor are common. These are presented in the following paragraphs. Some solved examples are given in Figure 5-6.

Figure 5-6 Examples of Computation of Vertical Stress

RECTANGULAR MAT FOUNDATION

REFER TO FIGURE 4 FOR INFLUENCE VALUES I FOR VERTICAL STRESS UNDER CORNER OF UNIFORMLY LOADED RECTANGULAR AREA. I FOR 1 OF 4 ADJOINING RECTANGULAR AREAS.

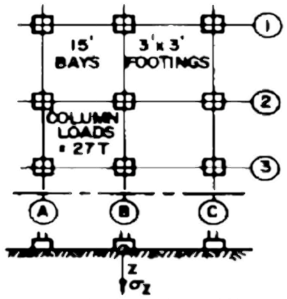


Z FT.	m	n	I 1 RECT.	4I 4 RECT.	$\sigma_z = 4Ip$ TSF
5	20	10	0.250	1.000	1.200
10	10	5	0.249	0.996	1.195
20	5	2.5	0.244	0.976	1.171
30	3.33	1.67	0.232	0.928	1.113
40	2.50	1.25	0.216	0.864	1.037
60	1.67	0.83	0.182	0.728	0.874
80	1.25	0.63	0.149	0.596	0.715
100	1.00	0.50	0.126	0.504	0.605

UNIT AREA LOAD (INCL. MAT) = 2 TSF, UNIT LOAD EXCAVATED = .8 TSF
NET AREA LOAD = 1.2 TSF. DETERMINE PROFILE OF APPLIED STRESS BENEATH CENTER OF MAT

SEPARATE COLUMN FOOTINGS

REFER TO FIGURE 5 FOR INFLUENCE VALUES I FOR VERTICAL STRESS OUTSIDE OF UNIFORMLY LOADED CIRCULAR AREA. SQUARE FOOTING ASSUMED EQUIVALENT TO CIRCULAR AREA.



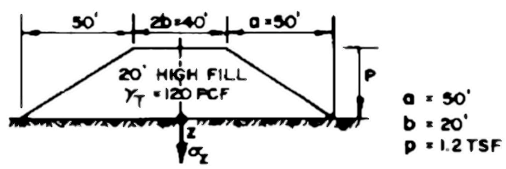
EQUIVALENT r:
$r = (9/\pi)^{1/2} = 1.695'$
p = 3.0 TSF
x DISTANCE:
COL. B2 = 0 (I_1)
COLS. A2, C2, B1, B3 = 15' (I_2)
COLS. A1, A3, C1, C3 = 21.2' (I_3)

Z FT.	z/r	I_1 x/r=0	$I_2/4$ x/r=8.9	$I_3/4$ x/r=12.5	ΣI	$\sigma_z = \Sigma I \cdot p$ TSF
2	1.18	0.64	0	0	0.640	1.720
4	2.36	0.22	0	0	0.220	0.660
6	3.54	0.110	0	0	0.110	0.330
10	5.90	0.042	0.002	0	0.050	0.150
15	8.85	0.019	0.003	0.001	0.035	0.105
20	11.80	0.011	0.004	0.002	0.035	0.105
25	14.74	0.007	0.003	0.002	0.027	0.081

DETERMINE PROFILE OF APPLIED STRESS BENEATH B2.

EMBANKMENT LOAD

REFER TO FIGURE 6 FOR INFLUENCE VALUES I FOR VERTICAL STRESS UNDER EMBANKMENT LOAD OF INFINITE LENGTH. I FOR 1/2 OF EMBANKMENT LOAD.



Z FT.	a/z	b/z	I	2I	$\sigma_z = 2I \cdot p$ TSF
5	10	4	0.500	1.000	1.200
10	5	2	0.496	0.992	1.190
20	2.5	1.0	0.475	0.950	1.140
30	1.67	0.67	0.444	0.888	1.066
40	1.25	0.50	0.410	0.820	0.984
60	0.83	0.33	0.344	0.688	0.826
80	0.63	0.25	0.294	0.588	0.706
100	0.50	0.20	0.252	0.504	0.605

DETERMINE PROFILE OF APPLIED STRESS σ_z BENEATH ℄ OF EMBANKMENT OF INFINITE LENGTH.

5.2.4.3.2.1. Square, Rectangular and Strip Foundations

Quick estimates of the influence factor for square and strip foundations may be obtained by using of Figure 5-4. For more accurate computations, use Figure 5-7.

Figure 5-7 Influence factor I_σ for the increase in vertical stress beneath a corner of a uniformly loaded rectangular area for the Boussinesq stress distribution[108]

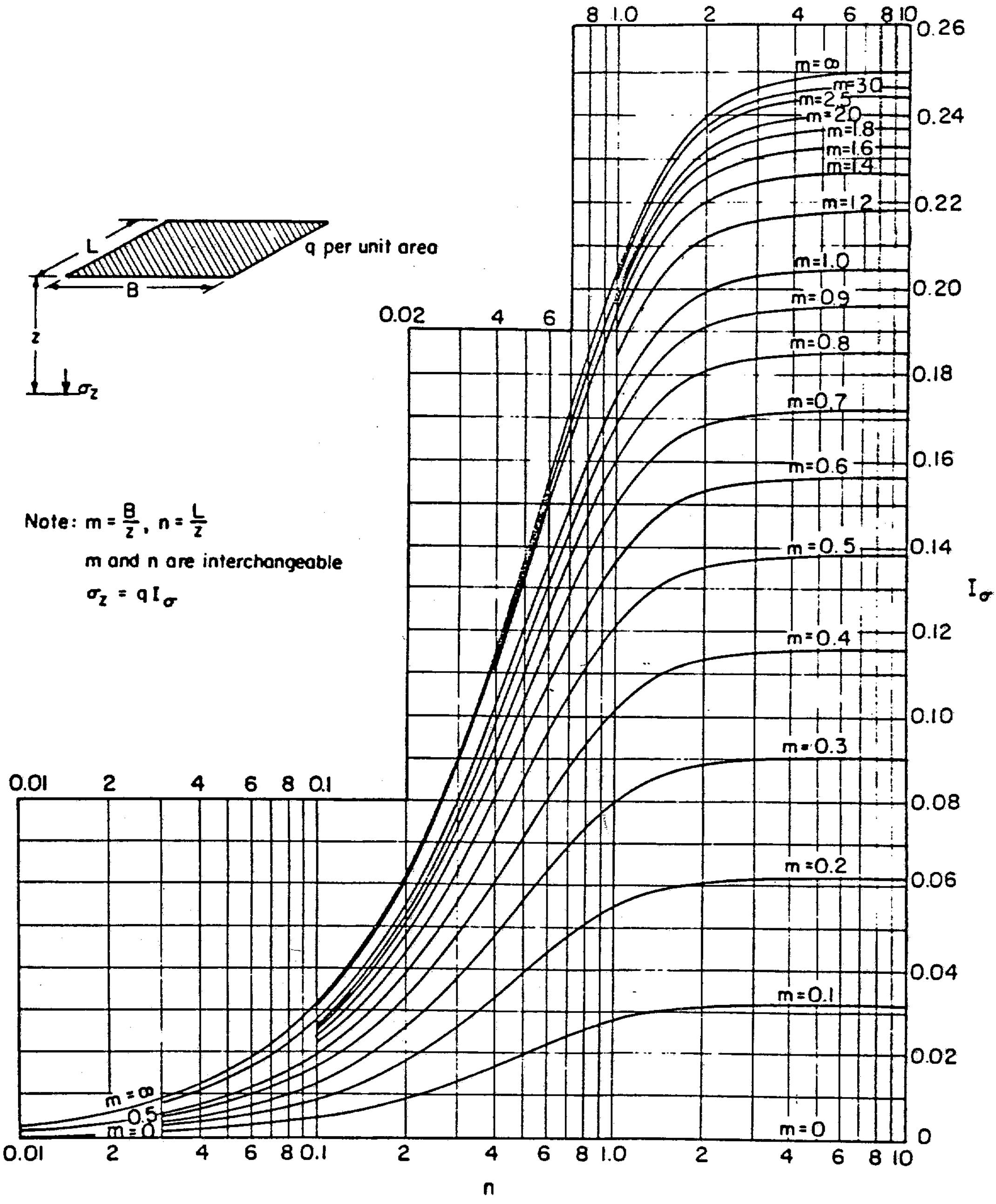
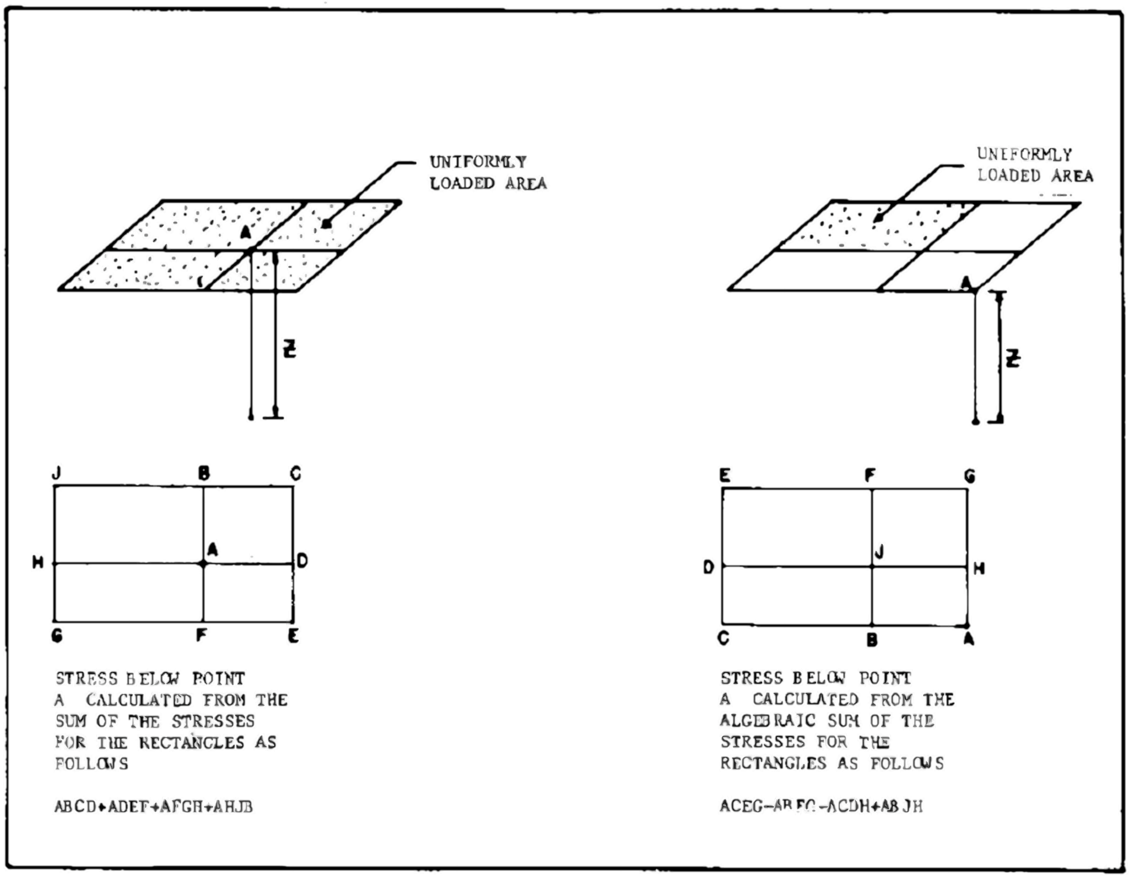


Figure 5-7 enables the direct computation of the stress under a corner of a square or rectangular foundations. For points beneath the mat, divide the mat into four rectangles with their common corner above the point to be investigated. Obtain influence values I for the individual rectangles from Figure

[108] Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, Stresses and Deflections in Foundations and Pavements, Fall, 1965.

5-7, and sum the values to obtain the total I. Points on the edge of the mat can be analysed by dividing the mat into two rectangles and adding the two resultant corner stresses. For points outside the area covered by the mat, use superposition of rectangles and add or subtract appropriate I values to obtain the resultant I. See example in Figure 5-8.

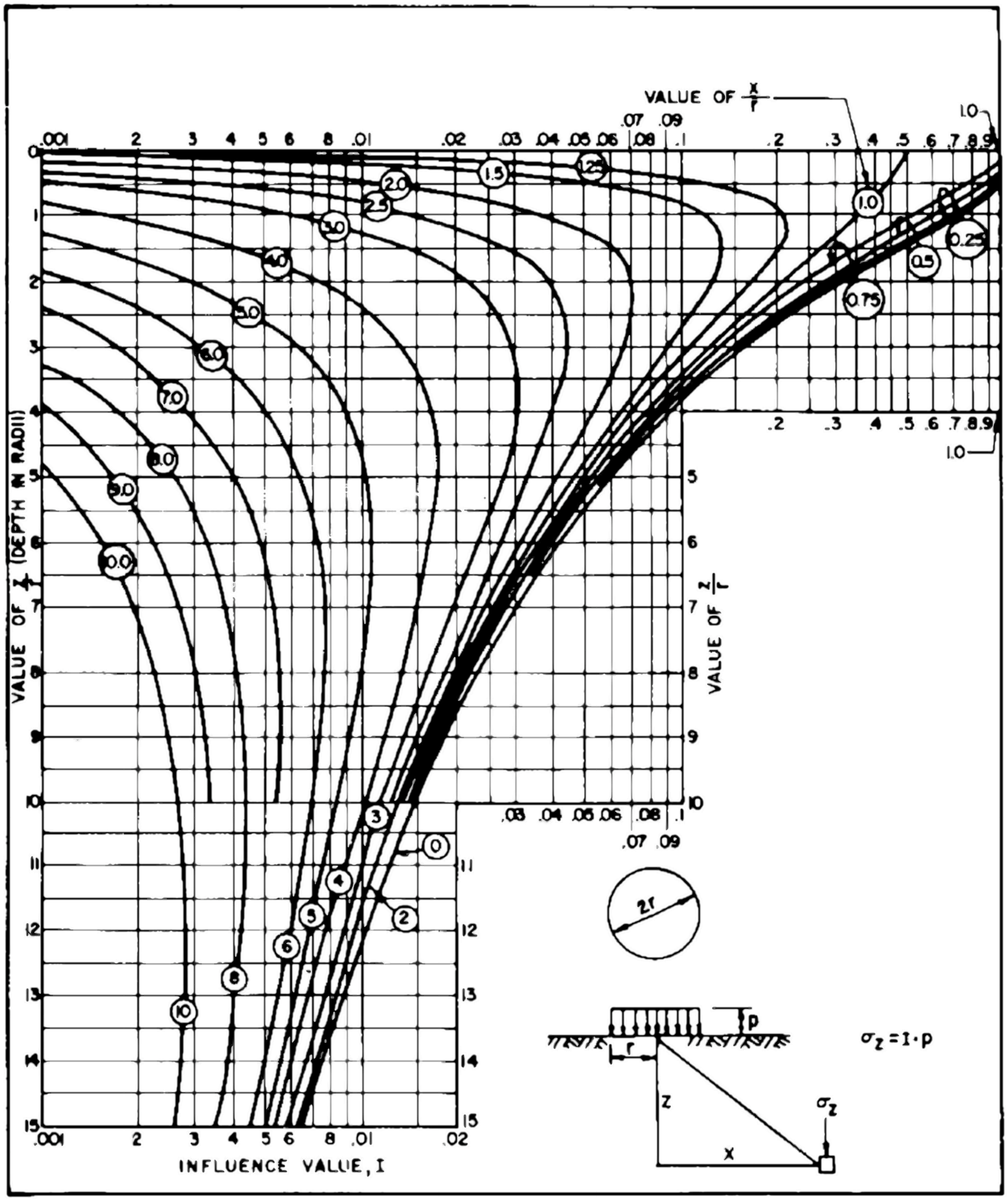
Figure 5-8 Use of Charts for Corner Stress to Compute Stress at other points Under a Foundation



5.2.4.3.2.2. Uniformly Loaded Circular Area

Use Figure 5-9 to compute stresses under circular footings.

Figure 5-9 Influence Value for Vertical Stress under a Uniformly Loaded Circular Area (Boussinesq Case)[109]

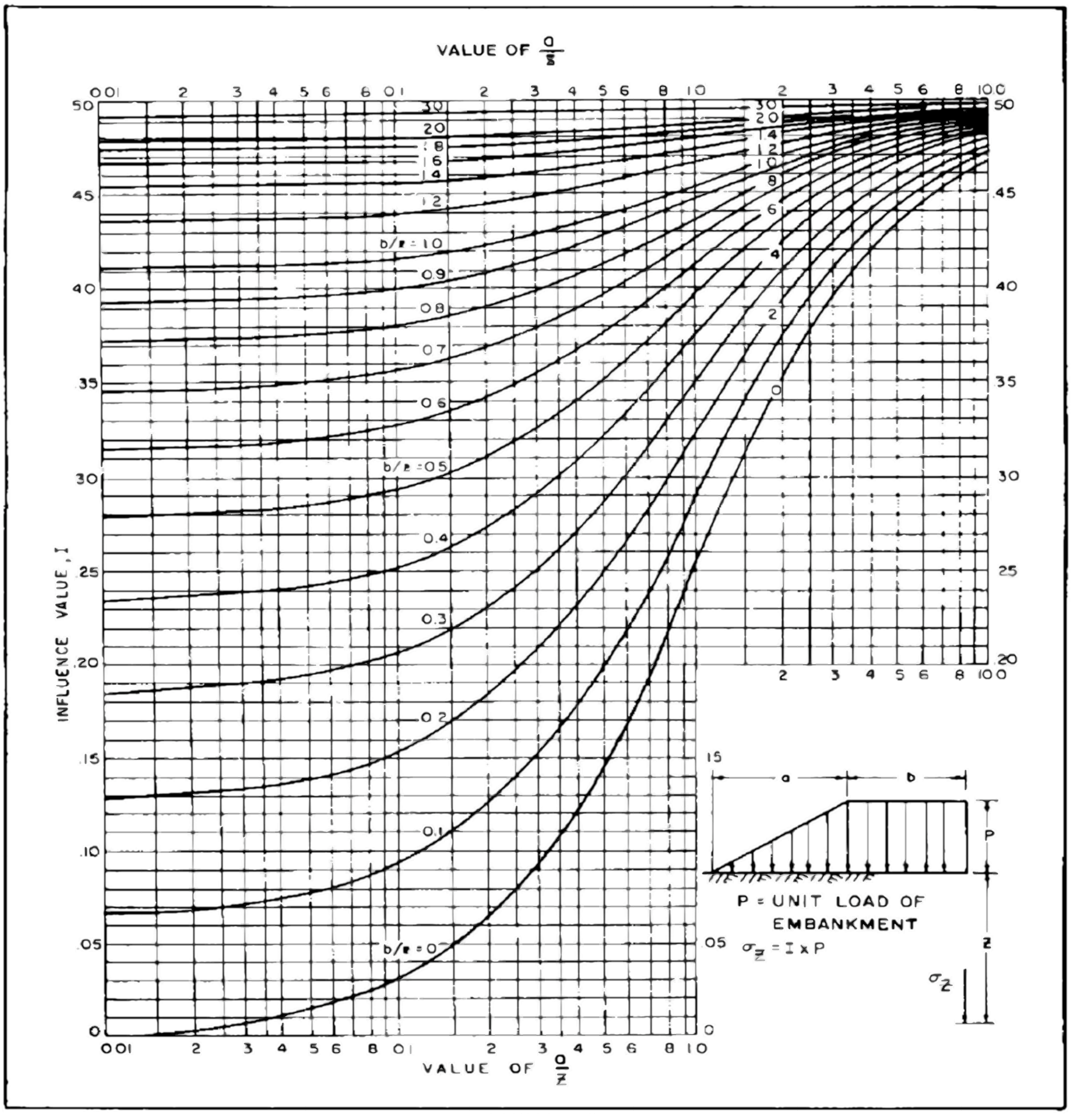


5.2.4.3.2.3. Embankment of Infinite Length

Use Figure 5-10 for embankments of simple cross section. For fills of more complicated cross section, add or subtract portions of this basic embankment load. For a symmetrical triangular fill, set dimension b equal to zero and add the influence values for two right triangles.

[109] Foster, C.R., and Ahlvin, P.G., Stresses and Deflections Induced by Uniform Circular Load, Highway Research Board Proceedings, Highway Research Board, Washington, D.C., 1954.

Figure 5-10 Influence Value for Vertical Stress Under Embankment Load of Infinite Length (Boussinesq Case)[110]

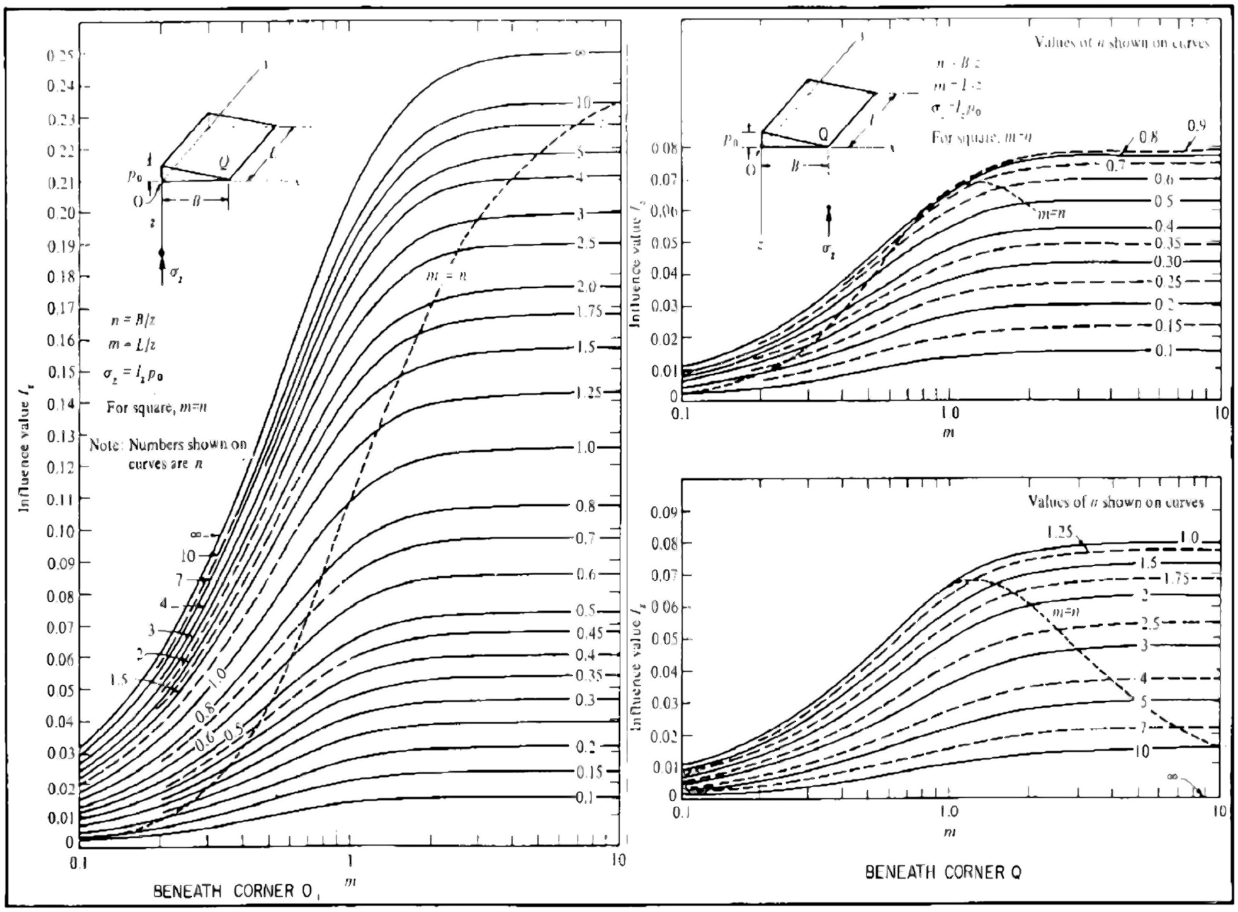


5.2.4.3.2.4. Sloping Fill of Finite Dimension

Use Figure 5-11 for stress beneath the corners of a finite sloping fill load.

[110] Osterberg, J.O., Influence Values for Vertical Stresses in a Semi-Infinite Mass Due to an Embankment Loading, Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

Figure 5-11 Influence Value for Vertical Stress beneath Triangular Load (Boussinesq Case)

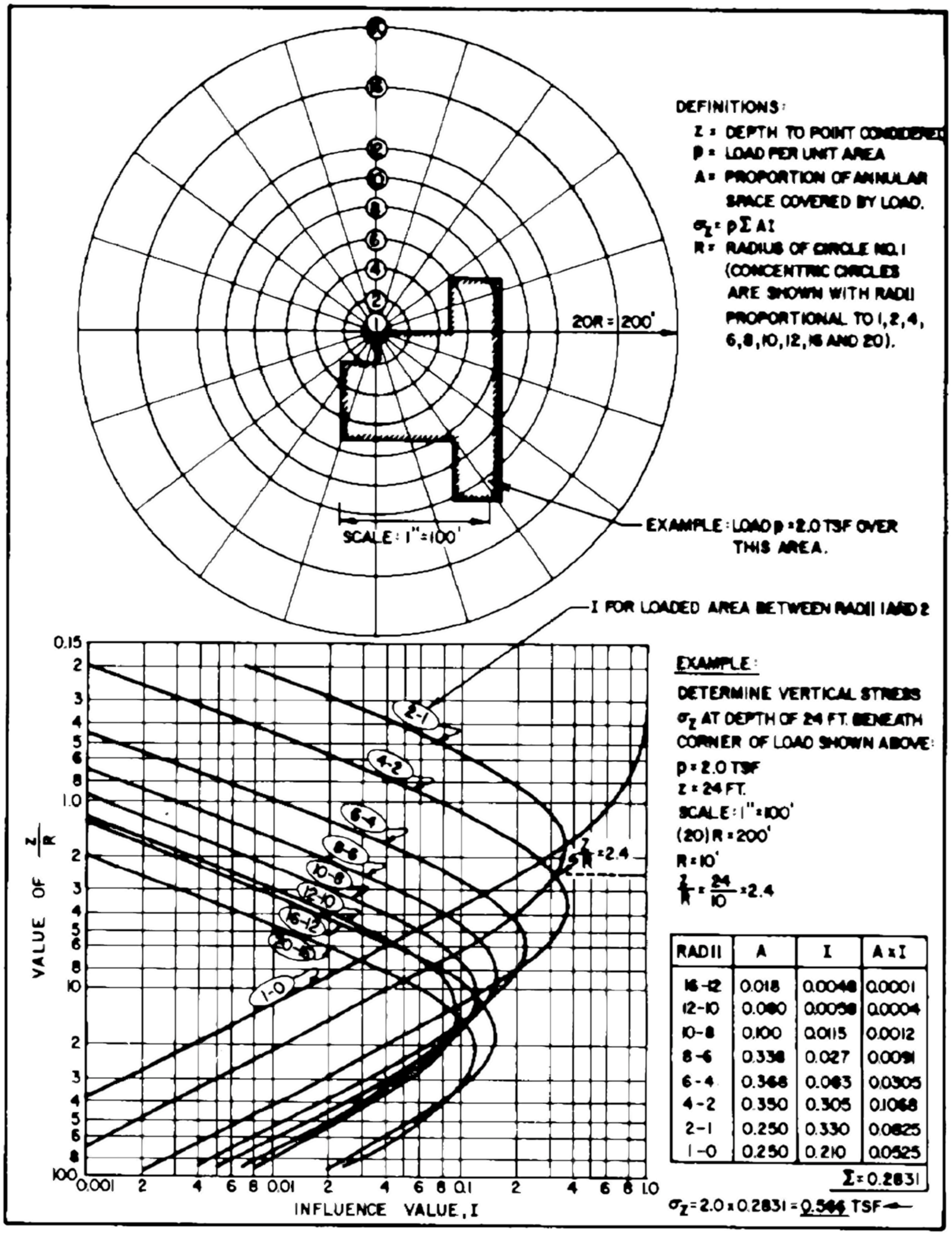


5.2.4.3.2.5. Vertical Stresses beneath Irregular Loads

Use Figure 5-12 for complex loads where other influence diagrams do not suffice. Proceed as follows:

- o Draw a circle of convenient scale and the concentric circles shown within it. The scale for the circle may be selected so that when the foundation plan is drawn using a standard scale (say 1"=100'), it will lie within the outer circle.
- o Plot the loaded area to scale on this target with the point to be investigated at the centre.
- o Estimate the proportion A of the annular area between adjacent radii that is covered by the load.
- o See the bottom chart of Figure 5-12 for influence values for stresses at various depths produced by the loads within each annular space. The product I x A multiplied by the load intensity equals vertical stress.
- o To determine a profile of vertical stresses for various depths beneath a point, the target need not be redrawn. Obtain influence values for different ordinates Z/R from the influence chart.

Figure 5-12 Influence Chart for Vertical Stress beneath Irregular Load[111]



5.2.4.3.2.6. Horizontal Stresses

Elastic analysis is utilized to determine horizontal stresses on unyielding walls from surcharge loads, and pressures on rigid buried structures.[112]

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[111] Jimenez Salas, J.A., Soil Pressure Computations: A Modification of Newmark's Method, Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948.

5.2.4.3.2.7. Shear Stresses

Elastic solutions generally are not applicable when shear stresses are critical, as in stability problems. To determine if a stability analysis is required, determine the maximum shear stress from elastic formulas and compare this stress with the shear strength of the soil. For embankment loads such as are shown in Figure 5-10, maximum shear stress in the foundation is exactly or approximately equal to p/π depending upon the shape of the load and point in question. If the maximum shear stress equals shear strength, plastic conditions prevail at some point in the foundation soil and if the load is increased, a larger portion of the foundation soil passes into plastic equilibrium. In this case, failure is possible and overall stability must be evaluated.

5.2.5. Layered or Anisotropic Foundations

Actual foundation conditions differ from the homogeneous isotropic, semi-infinite mass assumed in the Boussinesq expressions. The modulus of elasticity usually varies from layer to layer, and soil deposits frequently are more rigid in the horizontal direction than in the vertical.

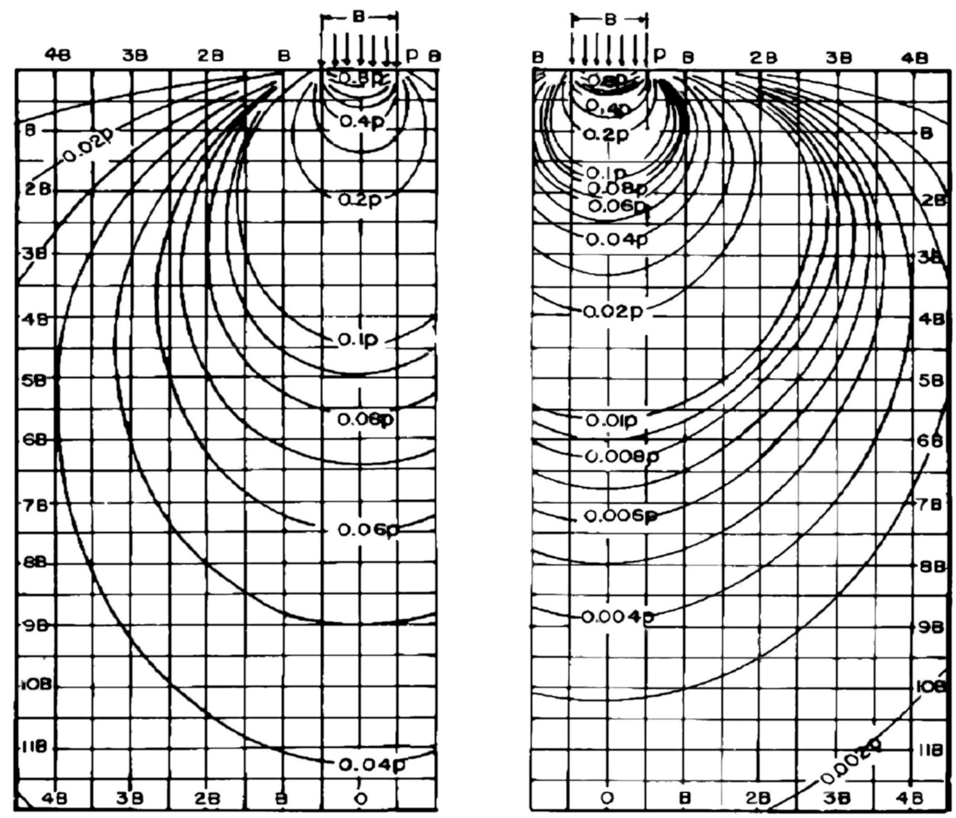
5.2.5.1. Westergaard

Soils that are stratified with strong layers may reinforce soft layers so that the resulting stress intensity at deeper depths is less than that formulated for isotropic soil after the Boussinesq approach. The Westergaard solution assumes that stratified soil performs like an elastic medium reinforced by rigid, thin sheets, and is applicable to soil profiles consisting of alternate layers of soft and stiff materials, such as soft clays with frequent horizontal layers and sand having greater stiffness in the horizontal direction. The increases in vertical stress beneath a corner of a rectangular uniformly loaded area may be evaluated from Equation 5-6 where the influence factor I_σ are found below.

Figure 5-13, Figure 5-14, and Figure 5-15 can be used for calculating vertical stresses in Westergaard material for three loading conditions. Computations for these figures are made in a manner identical to that for the corresponding Boussinesq charts. For an illustration, see Figure 5-6.

[112] For more information, see Poulos, H.G. and Davis, E.H., Elastic Solutions for Soil and Rock Mechanics, John Wiley & Sons, Inc., New York, 1974.

Figure 5-13 Vertical Stress Contours for Square and Strip Footings (Westergaard Case)



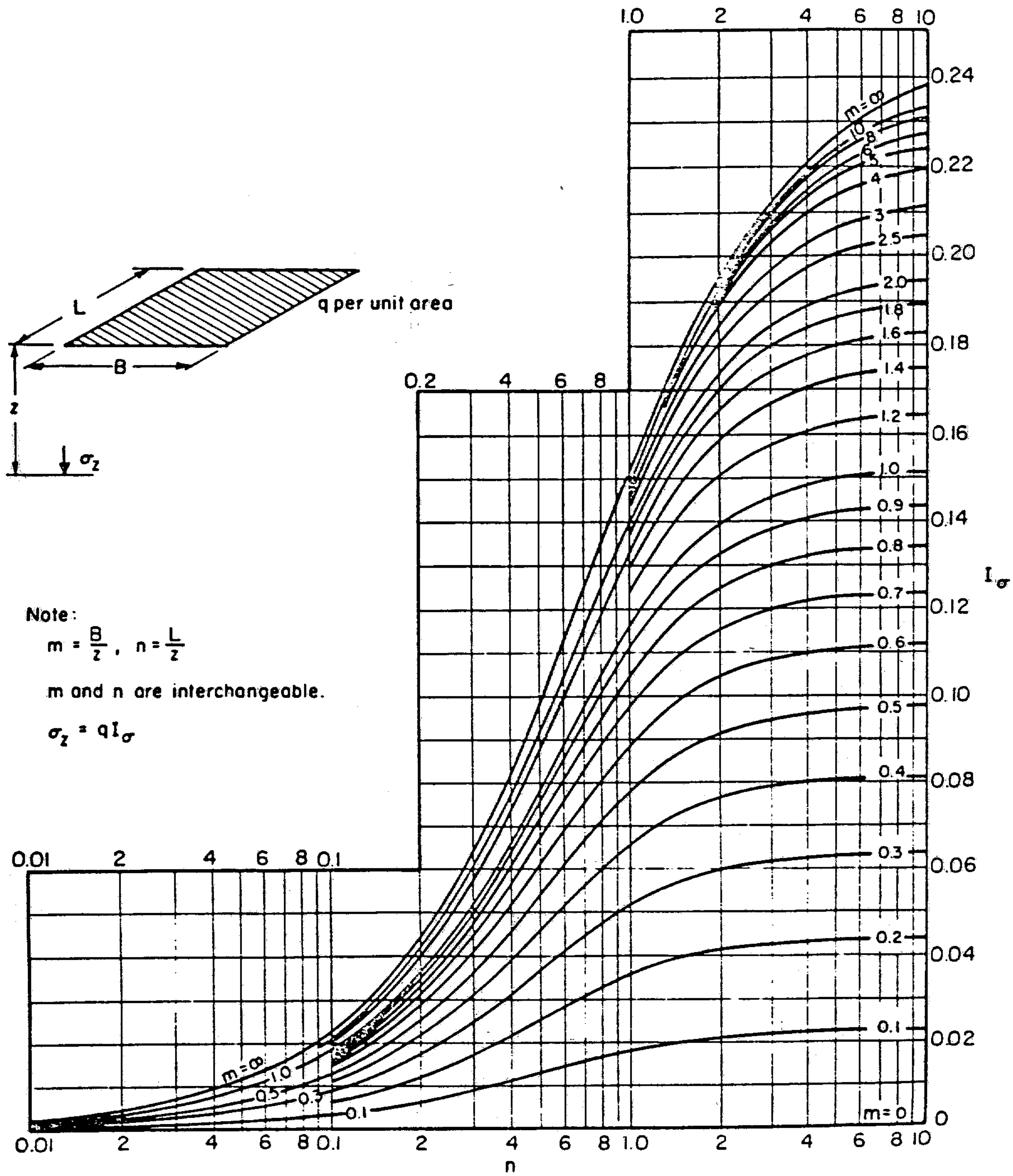
a. INFINITELY LONG FOOTING

b. SQUARE FOOTING

EXAMPLE:

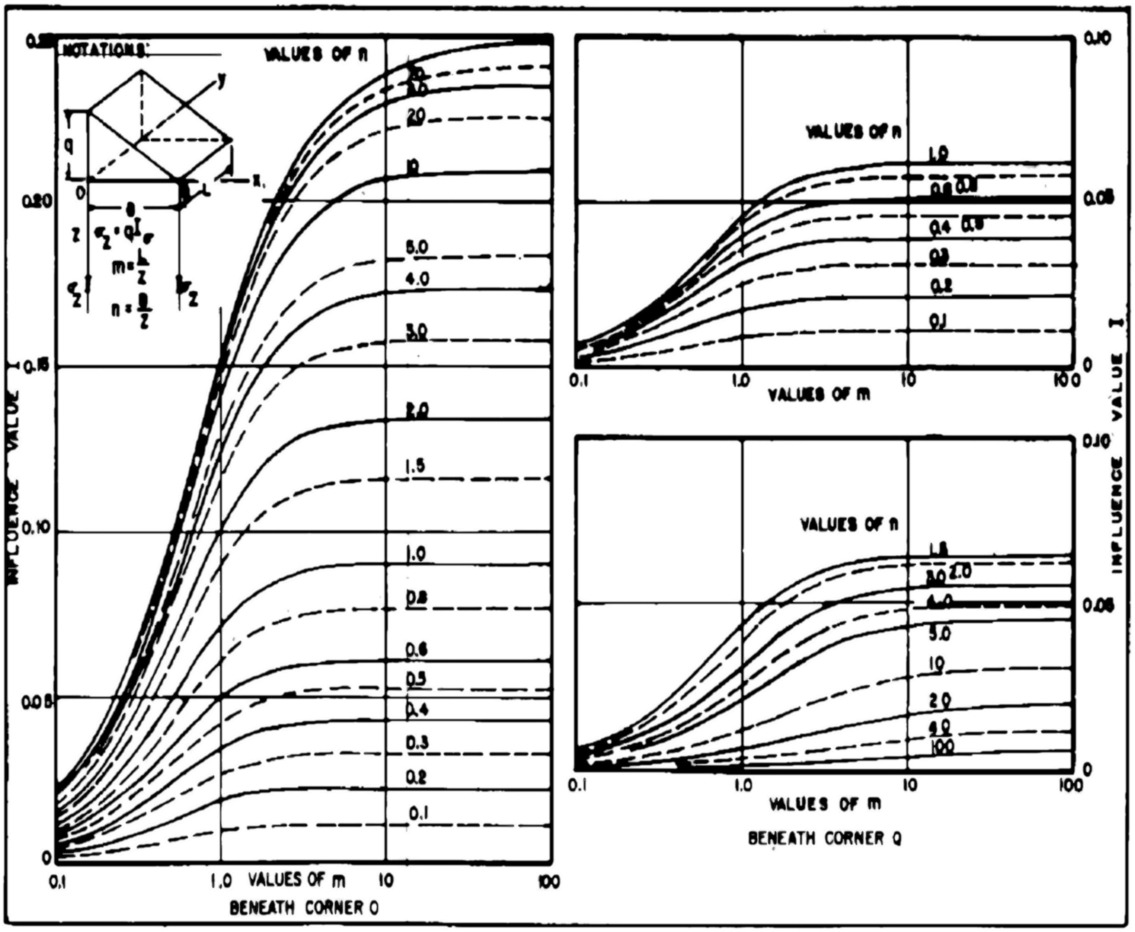
FIND THE PRESSURE INCREASE DUE TO A STRIP FOOTING OF WIDTH B, AT A POINT LOCATED 6B BELOW ITS BASE AND 3B FROM THE CENTER OF THE FOOTING. SURFACE LOAD ON THE FOOTING IS P PER-UNIT AREA. FROM THE LEFT PANEL, PRESSURE INCREASE =0.05 P

Figure 5-14 Influence Value for Vertical Stress Beneath a Corner of a Uniformly Loaded Rectangular Area (Westergaard Case)[113]



[113] Duncan, J.M., and Buchignani, A.L., An Engineering Manual for Settlement Studies, Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, June, 1976.

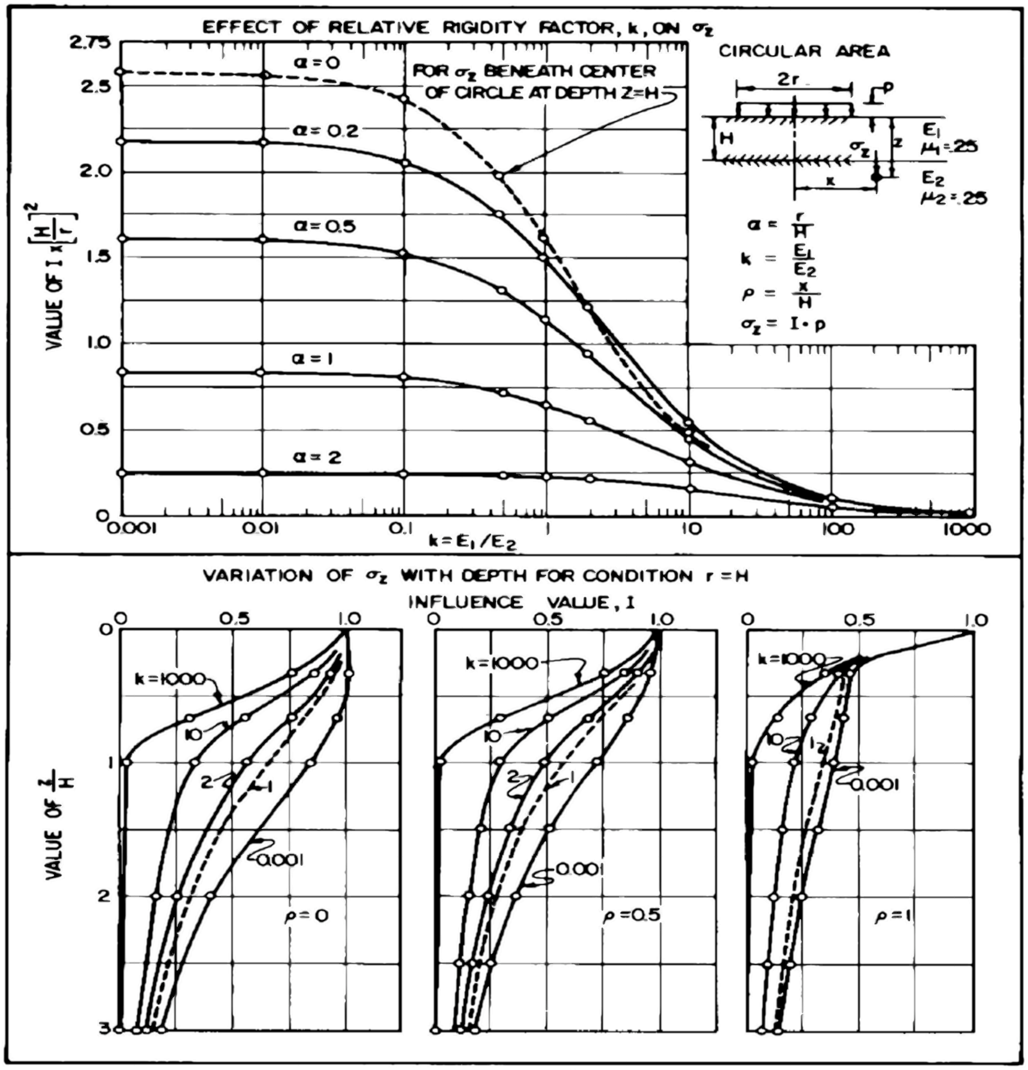
Figure 5-15 Influence Value for Vertical Stress Beneath a Triangular Load (Westergaard Case)



5.2.5.2. Layered Foundations

When the foundation soil consists of a number of layers of substantial thickness, having distinctly different elastic properties, the vertical and other stresses are markedly different from those obtained by using the Boussinesq equation. (See Figure 5-16 for influence values of vertical stresses in a two-layer foundation with various ratios of modulus of elasticity. See Figure 5-17 for an example.)

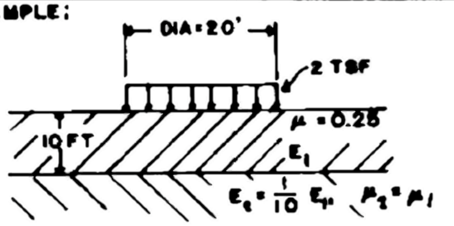
Figure 5-16 Influence Values for Vertical Stresses beneath a Uniformly Loaded Circular Area (Two-Layer Foundation)[114]



[114] Mehta, M.R., and Veletsos, A.S., Stresses and Displacement in Layered Systems, Structural Research Series No. 178, University of Illinois, Urbana, IL.

Figure 5-17 Stress Profile in a Two-Layer Soil Mass

EXAMPLE:



1. DETERMINE PROFILE OF STRESS INCREASE DUE TO APPLIED LOAD BELOW THE EDGES.

$a = \frac{10}{10} = 1$, $k = 10$

$\rho = \frac{10}{10} = 1$ — USE RIGHT HAND GRAPH OF LOWER PANEL OF FIGURE 14

DEPTH FT.	$\frac{z}{H}$	I	$\sigma_z = I \cdot p$ TSF
5	0.5	0.35	0.70
10	1.0	0.21	0.42
15	1.5	0.15	0.30
20	2.0	0.12	0.24
25	2.5	0.10	0.20
30	3.0	0.07	0.14

- Rigid Surface Layer over Weaker Underlying Layer. If the surface layer is the more rigid, it acts as a distributing mat and the vertical stresses in the underlying soil layer are less than Boussinesq values.
- Weaker Surface Layer over Stronger Underlying Layers. If the surface layer is less rigid than the underlying layer, then the vertical stresses in both layers exceed the Boussinesq values.[115]

∞ Critical Depth. If there is no distinct change in the character of subsurface strata within the critical depth, elastic solutions for layered foundations need not be considered. Critical depth below the foundation within which soil compression contributes significantly to surface settlements. For fine-grained compressible soils, the critical depth extends to that point where applied stress decreases to 10% of effective overburden pressure. In coarse-grained material critical depth extends to that point where applied stress decreases to 20% of effective overburden pressure.

5.2.6. Approximate 2:1 Distribution

An approximate stress distribution assumes that the total applied load on the surface of the soil is distributed over an area of the same shape as the loaded area on the surface, but with dimensions that increase by an amount equal to the depth below the surface, Figure 5-18. At a depth z in feet below the ground surface, the total load Q applied at the ground surface by a structure is assumed to be uniformly distributed over an area (B + z) wide by (L + z) long. The increase in vertical pressure $\Delta\sigma_z$ at depth z for an applied load Q is given by

[115] For influence diagrams for vertical stresses beneath rectangular loaded areas, see Burmister, D.M., Stress and Displacement Characteristics of a Two-Layer Rigid Base Soil System: Influence Diagrams and Practical Applications, Proceedings, Highway Research Board, Washington, D.C., 1956. Use these influence diagrams to determine vertical stress distribution for settlement analysis involving a soft surface layer underlain by stiff material.

Equation 5-7: $\Delta\sigma_z = \dfrac{Q}{(B+z)(L+z)}$ **(Square and Rectangular Footings)**

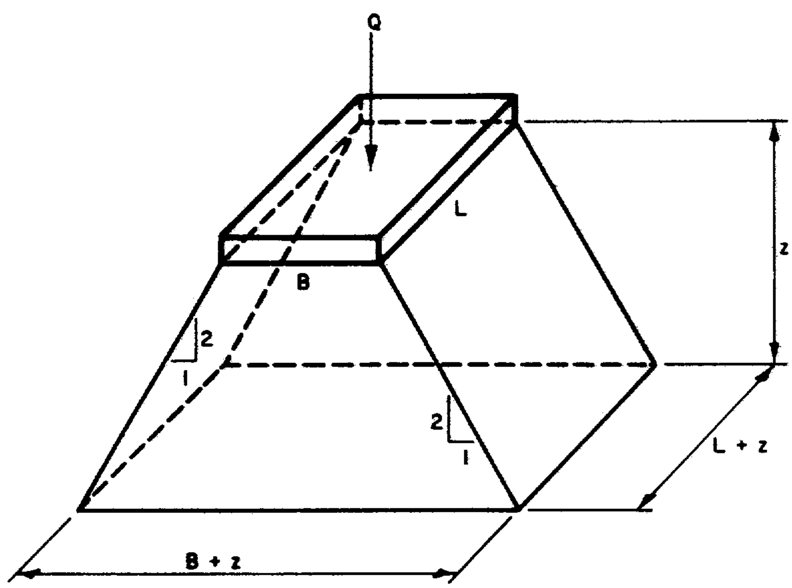
Equation 5-8: $\Delta\sigma_z = \dfrac{4Q}{\pi(D+z)^2}$ **(Circular Footings)**

Where

- B = width of the footing (rectangular or square footing)
- L = length of the footing (rectangular or square footing)
- D = diameter of the footing (circular footing)

$\Delta\sigma_z$ may be the pressure σ_{st} caused by construction of the structure.

Figure 5-18 Approximate stress distribution by the 2:1 method



Vertical stresses calculated by Equation 5-7 agree reasonably well with the Boussinesq method discussed below for depths between B and 4B below the foundation.

5.2.7. Contact Pressure and Deformation Pattern

The shape of the deformation pattern varies depending on flexibility of the foundation and type of soil. Figure 5-19 illustrates the relative distribution of soil contact pressures and displacements on cohesionless and cohesive soil. Linear contact pressure distributions from uniformly applied pressure q are often assumed for settlement analysis, Figure 5-19c and Figure 5-19d. An applied load Q may cause an unequal linear soil contact pressure distribution, Figure 5-19e.

5.2.7.1. Cohesionless soil

Cohesionless soil is often composed of granular or coarse-grained materials with visually detectable particle sizes and with little cohesion or adhesion between particles. These soils have little or no strength when unconfined and little or no cohesion when submerged. Apparent adhesion between particles in cohesionless soil may occur from capillary tension in pore water. Settlement usually occurs rapidly with little long-term consolidation and secondary compression or creep. Time rate effects may become significant in proportion to the silt content such that the silt content may dominate consolidation characteristics.

d) Uniformly loaded rigid foundations (footings of limited size or footings on cohesionless soil) may cause less soil contact pressure near the edge than near the centre, Figure 5-19a, because this soil is pushed aside at the edges due to the reduced confining pressure. This leads to lower strength and lower modulus of elasticity in soil near the edge compared with soil near the centre. The parabolic soil contact pressure distribution may be replaced with a saddle-shaped distribution, Figure 5-19b, for rigid footings or mats if the soil pressure does not approach the allowable bearing capacity.

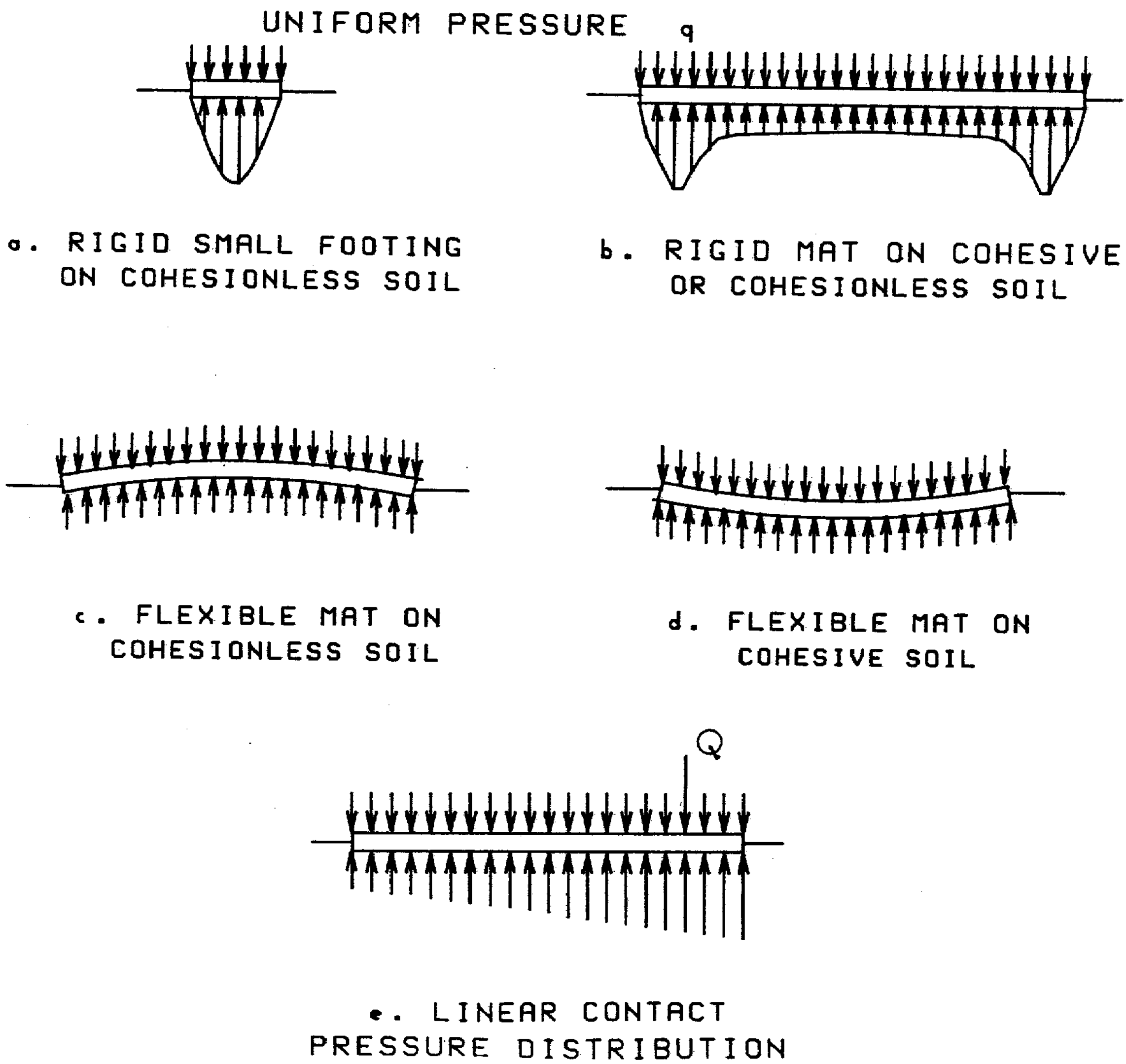
e) The distortion of a uniformly loaded flexible footing, mat, or embankment on cohesionless soil will be concave downward, Figure 5-19c, because the soil near the centre is stressed under higher confining pressure such that the modulus of elasticity of the soil is higher than near the edge.

f) The theory of elasticity is not applicable to cohesionless soil when the stress or loading increment varies significantly throughout the soil such that an equivalent elastic modulus cannot be assigned. Semi-empirical and numerical techniques have been useful to determine equivalent elastic parameters at points in the soil mass based on stress levels that occur in the soil.

5.2.7.2. Cohesive soil

Cohesive soil often contains fine-grained materials consisting of silts, clays, and organic material. These soils have significant strength when unconfined and air-dried. Most cohesive soil is relatively impermeable and when loaded deforms similar to gelatine or rubber; i.e., the undrained state. Cohesive soils may include granular materials with bonding agents between particles such as soluble salts or clay aggregates. Wetting of soluble agents bonding granular particles may cause settlement in loose or high void ratio soil. Refer to 7.5 for evaluation of settlement in collapsible soil.

Figure 5-19 Relative distribution of soil contact pressures and displacements of rigid and flexible mats or footings on cohesionless and cohesive soils



a) A uniform pressure applied to a rigid foundation on cohesive soil, Figure 5-19b, can cause the soil contact pressure to be maximum at the edge and decrease toward the centre because additional contact pressure is generated to provide stress that shears the soil around the perimeter.

b) A uniform pressure applied to a flexible foundation on cohesive soil, Figure 5-19d, causes greater settlement near the centre than near the edge because the cumulative stresses are greater near the centre because of the pressure bulb stress distribution indicated in Figure 5-5. Earth pressure measurements from load cells beneath a stiffening beam supporting a large, but flexible, ribbed mat also indicated large perimeter earth pressures resembling a saddle-shaped pressure distribution similar to Figure 5-19b[116].

c) Elastic theory has been found useful for evaluation of immediate settlement when cohesive soil is subjected to moderate stress increments. The modulus of elasticity is a function of the soil shear strength and often in creases with increasing depth in proportion with the increase in soil shear strength.

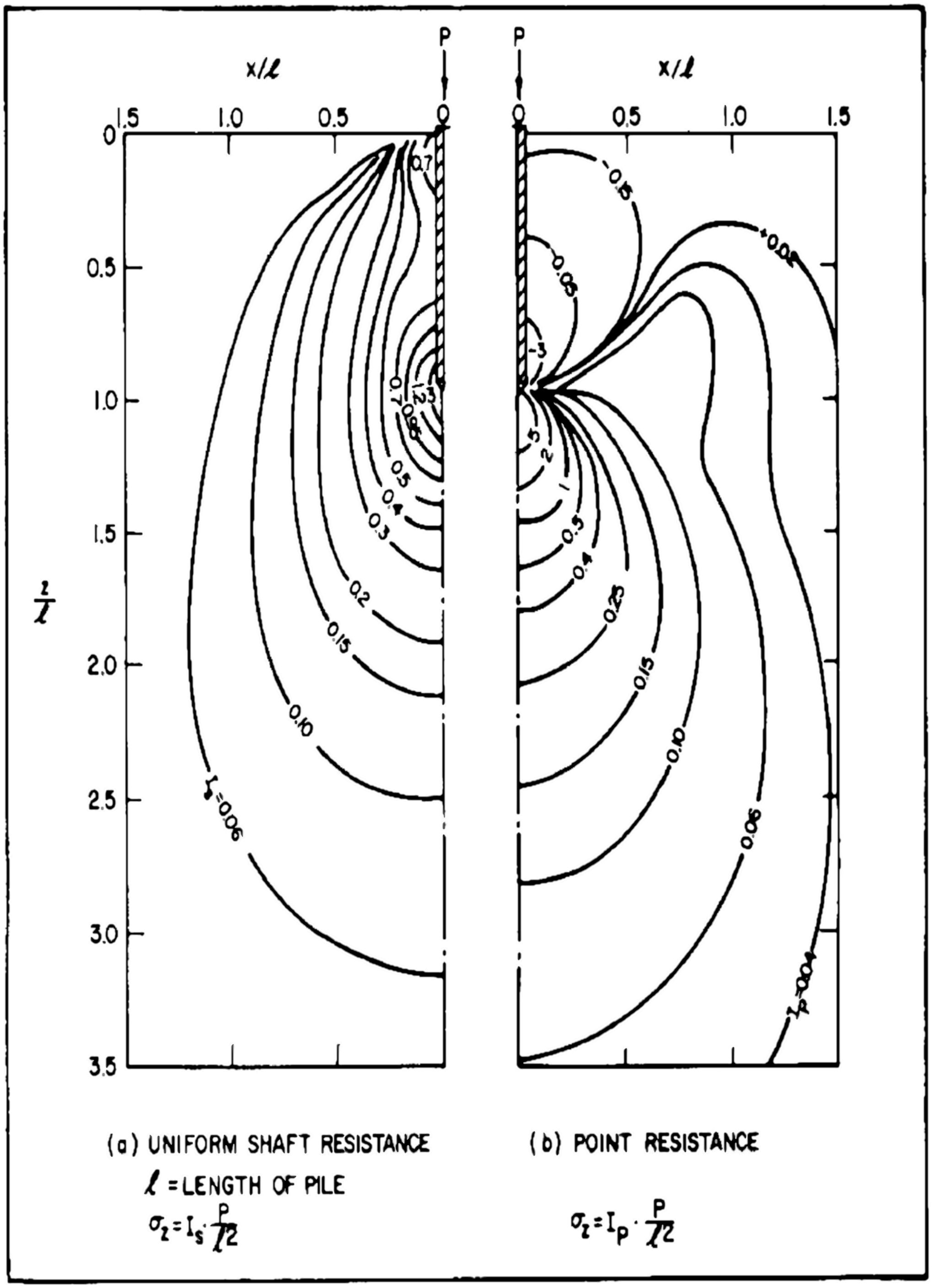
d) Cohesive soil subject to stresses exceeding the maximum past pressure of the soil may settle substantially from primary consolidation and secondary compression and creep.

5.2.8. Stresses Induced by Pile Loads

Estimates of the vertical stresses induced in a soil mass by an axially loaded pile are given in Figure 17 for both friction and end-bearing piles.

[116] Johnson, L. D. 1989. "Performance of a Large Ribbed Mat on Cohesive Soil," *Foundation Engineering: Current Principles and Practices*, Vol 1, F. H. Kulhawy, ed. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Figure 5-20 Influence Values for Vertical Stresses around a Pile in an Elastic Solid[117]



$\sigma_z = I_s \cdot \frac{P}{\ell^2}$

$\sigma_z = I_p \cdot \frac{P}{\ell^2}$

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117 Grillo, O., Influence Scale and Influence Chart for the Computation of Stresses Due, Respectively, to Surface Point Load and Pile Load, Proceedings of Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 6, pp 70-73, 1948.

5.2.9. Limitations of Theoretical Solutions

Boundary conditions may differ substantially from idealized conditions to invalidate solutions by elasticity theory.

5.2.9.1. Initial Stress

Elastic solutions such as the Boussinesq solution assume a weightless material not subject to initial stress. Initial stress always exists *in situ* because of overburden weight of overlying soil, past stress history, and environmental effects such as desiccation. These initial stresses through Poisson's ratio, nonlinear elastic modulus and soil anisotropy significantly influence *in situ* stress and strain that occur through additional applied loads.

5.2.9.2. Error in Stress Distribution

Actual stresses beneath the centre of shallow footings may exceed Boussinesq values by 15 to 30% in clays and 20 to 30% in sands[118].

5.2.9.3. Critical Depth

The critical depth z_c is the depth at which the increase in stresses $\Delta\sigma_z$ from foundation loads decrease to about 10 (cohesionless soil) to 20 (cohesive soil) percent of the effective vertical overburden pressure σ'_o (item 5). Errors in settlement contributed by nonlinear, heterogeneous soil below the critical depth are not significant.

5.3. Deep Underground Openings

Pressures acting on underground openings after their completion depend on the character of the surrounding materials, inward movement permitted during construction, and restraint provided by the tunnel lining.

5.3.1. Openings in Rock

Stress analysis differs for two rock groups: sound, nonswelling rock that can sustain considerable tensile stresses, and fractured blocky, seamy, squeezing, or swelling rock. For detailed explanations of these rock groups, see § 2.

∞ Sound Rock. Determine stresses surrounding tunnels or openings in intact, isotropic rock, such as crystalling igneous types, or homogeneous sandstone and limestone, by elastic analyses.[119]

  For these materials, stresses in rock surrounding spheroidal cavities are lower than those for tunnels with the same cross section. Use elastic analyses to determine the best arrangement of openings and pillars, providing supports as required at locations of stress concentrations. For initial estimates of roof pressure, Table 5-1 may be used.

∞ Broken and Fractured Rock. Pressure on tunnels in chemically or mechanically altered rock must be analysed by approximate rules based on experience.[120]

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[118] Burmister, D. M. 1954. "Influence Diagram of Stresses and Displacements in a Two-Layer Soil System With a Rigid Base at a Depth H," Contract No. DA-49-129-ENG-171 with US Army Corps of Engineers, Columbia University, New York, NY. Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

[119] Obert, L., Duvall, W.I. and Merill, R.H., Design of Underground Openings in Competent Rock, Bulletin, U.S. Bureau of Mines.

∞ Squeezing and Swelling Rocks. Squeezing rocks contain a considerable amount of clay. The clay fraction may be from non-swelling kaolinite group or from highly swelling montmorillonite group. These rocks are preloaded clays and the squeezing is due to swelling. The squeeze is intimately related to an increase in water content and a decrease in shear strength.

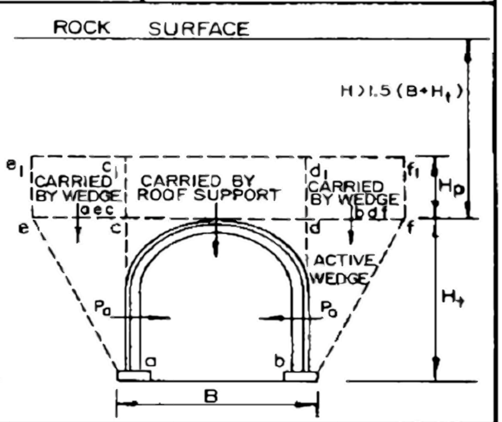
5.3.2. Loads on Underground Openings in Rock

∞ Vertical Rock Load. Table 5-1 gives the height or rock above the tunnel roof that must be supported by roof lining.

Table 5-1 Overburden Rock Load Carried by Roof Support[120]

Rock Conditions	Rock Load H_p in Feet	Remarks
1. Hard and intact	Zero	Sometimes spalling or popping occurs.
2. Hard stratified or schistose	0 to 0.5 B	Light pressures.
3. Massive, moderately jointed	0 to 0.25 B	Load may change erratically from point to point.
4. Moderately blocky and seamy	0.25 B to 0.35 $(B+H_t)$	No side pressure.
5. Very blocky and seamy	0.35 to 1.10 $(B+H_t)$	Little or no side pressure.
6. Completely crushed but chemically intact	1.10 $(B+H_t)$	Considerable side pressure. Softening effect of seepage towards bottom of tunnel.
7. Squeezing rock, moderate depth	(1.10 to 2.10) $(B+H_t)$	Heavy side pressure.
8. Squeezing rock, great depth	(2.10 to 4.50) $(B+H_t)$	
9. Swelling rock	Up to 250 ft. irrespective of value of $(B+H_t)$	Very heavy pressures.

Notes:
1. Above values apply to tunnels at depth greater than 1.5 $(B+H_t)$.
2. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for rock conditions 4 to 6 can be reduced by fifty percent.
3. Some very dense clays which have not yet acquired properties of shale rock may behave as squeezing or swelling rock.
4. Where sandstone or limestone contain horizontal layers of immature shale, roof pressures will correspond to rock condition "very blocky and seamy."



∞ Horizontal Pressures. Determine the horizontal pressure P_a on tunnel sides by applying the surcharge of this vertical rock load to an active failure wedge (see diagram in Table 5-1). Assume values or

[120] Proctor, R.V. and White, T.L., Rock Tunneling With Steel Supports, Commercial Shearing Inc., Youngstown, OH, 1968.

rock shear strength (see § 3 for a range of values) on the active wedge failure plane, which allow for the fractured or broken character of the rock. Evaluate the possibility of movement of an active failure plane that coincides with weak strata or bedding intersecting the tunnel wall at an angle.

- Support Pressures as Determined from Rock Quality. Alternate analyses exist for determining these support pressures.[121] The analysis incorporates rock quality designation (RQD) and various joint properties of the surrounding material, and is applicable for sound or fractured rock. Results may be used directly for evaluating type of roof or wall support required.

5.3.3. Openings in Soft Ground

- Ground Behaviour. The method of construction of tunnels depends upon the response of the ground during and after excavation. The stand up time depends upon the type of soil, the position of groundwater, and the size of opening. Depending upon the response during its movement period, the ground is classified as: (1) firm, (2) ravelling, (3) running, (4) flowing, (5) squeezing or (6) swelling.

  1) In firm ground, no roof support is needed during excavation and there is no perceptible movement.

  2) In ravelling ground, chunks or flakes of material begin to fall prior to installing the final group supports. Stand up time decreases with increasing size of excavation. With rising groundwater, ravelling ground may become running ground. Sand with clay binder is one example of this type of soil.

  3) In running ground, stand up time is zero. The roof support must be inserted prior to excavation. Removal of side supports results in inflow of material that comes to rest at its angle of repose. Dry cohesionless soils fall into this category.

  4) Flowing ground acts as a thick liquid and it invades the opening from all directions including the bottom. If support is not provided, flow continues until the tunnel is completely filled. Cohesionless soil below groundwater constitutes flowing ground.

  5) Squeezing ground advances gradually into the opening without any signs of rupture. For slow advancing soil, stand up time is adequate, yet the loss of ground results in settlement of the ground surface. Soft clay is a typical example of squeezing ground.

  6) Swelling ground advances into the opening and is caused by an increase in volume due to stress release and/or moisture increase. Pressures on support members may increase substantially even after the movement is restrained.

- Loss of Ground. As the underground excavation is made, the surrounding ground starts to move toward the opening. Displacements result from stress release, soil coming into the tunnel from ravelling, runs, flows, etc. The resulting loss of ground causes settlement of the ground surface. The loss of ground associated with stress reduction can be predicted reasonable well, but the ground loss due to ravelling, flows, runs, etc. requires a detailed knowledge of the subsurface conditions to avoid unacceptable amounts of settlement.[122]

- Loads. The support pressures in the underground openings are governed by the unit weight of the soil, groundwater table, soil properties, deformations during excavation, interaction between soil and the supports, shape of the opening, and the length of time that has elapsed since the installation of

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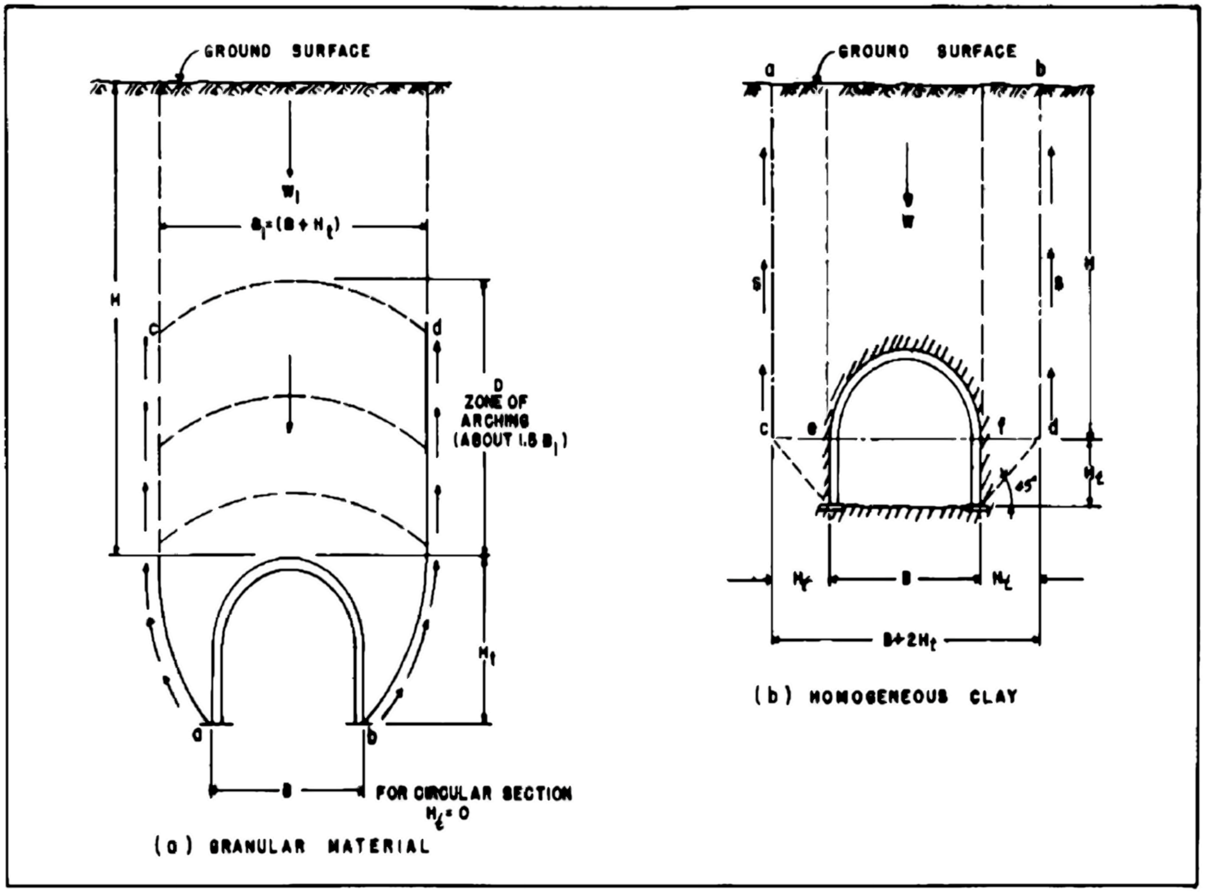
[121] Barton, N., Lien, R. and Lunde, J., Engineering Classification of Rock Masses for Tunnel Support, Rock Mechanics, Volume 6, No. 4, Journal International Society of Rock Mechanics, 1974.

[122] For acceptable levels of ground loss in various types of soils, see Proctor, R.V. and White, T.L., Earth Tunneling with Steel Supports, Commercial Shearing Inc., Youngstown, OH, 1977.

lining. Other factors such as the presence of another opening adjacent to it, excavation of a large deep basement near an existing opening, load from neighbouring structures, and change in groundwater conditions, will also affect the design pressures on the tunnel supports. A schematic representation of the load action is shown in Figure 5-21.

Figure 5-21 Load Action on Underground Openings in the Earth



Estimate of load for temporary supports in earth tunnels may be obtained from Table 5-2.[123]

Table 5-2 Loads for Temporary Supports in Earth Tunnels at Depths of More Than 3(B+H$_t$)/2

Type of Ground	Ground Condition	Design Load H_p[124]	Remarks
Running ground above water table	Loose	$\frac{B+H_t}{2}$	
	Medium	$\frac{2(B+H_t)}{5}$	

[123] Further information can be found in Peck, R.B., Tunneling in Soft Ground, Geotechnical Considerations, Seminar on Underground Construction, Vail, CO, 1976.

[124] For circular tunnels, $H_t = 0$, B = diameter

Type of Ground	Ground Condition	Design Load H_p[124]	Remarks
	Dense	$\frac{3(B+H_t)}{10}$	
Running ground in compressed-air tunnel		Disregard air pressure; H_p, equal to that for running ground, above water table with equal density.	
Flowing ground in free-air tunnel		H or 2 ($B + H_t$) which ever is smaller	
Raveling ground	Above water table	$\frac{T-t}{T}H_p$	
	Below water table (free air)	$\frac{T-t}{T}H_p$	
	Below water table (compressed air)	$2\frac{T-t}{T}H_p - \frac{p_c}{\gamma}$	
Squeezing Ground	Homogeneous	$H = \frac{p_c}{\gamma} - \frac{Hq_u}{2\gamma(B+2H_t)}$	After complete blowout, $p_c = 0$
	Soft Soil, stiff sides	$H = \frac{p_c}{\gamma} - \frac{Hq_u}{2\gamma B}$	
	Stiff soil, soft sides	$H = \frac{p_c}{\gamma} - \frac{Hq_u}{2\gamma(B+6H_t)}$	
Swelling Ground	Intact	Very small	Permanent roof support should be completed within a few days after mining
	Fissured, above water table	H_p equal to that for raveling ground with same stand up to time H	
	Fissured, below water table, free-air tunnel		

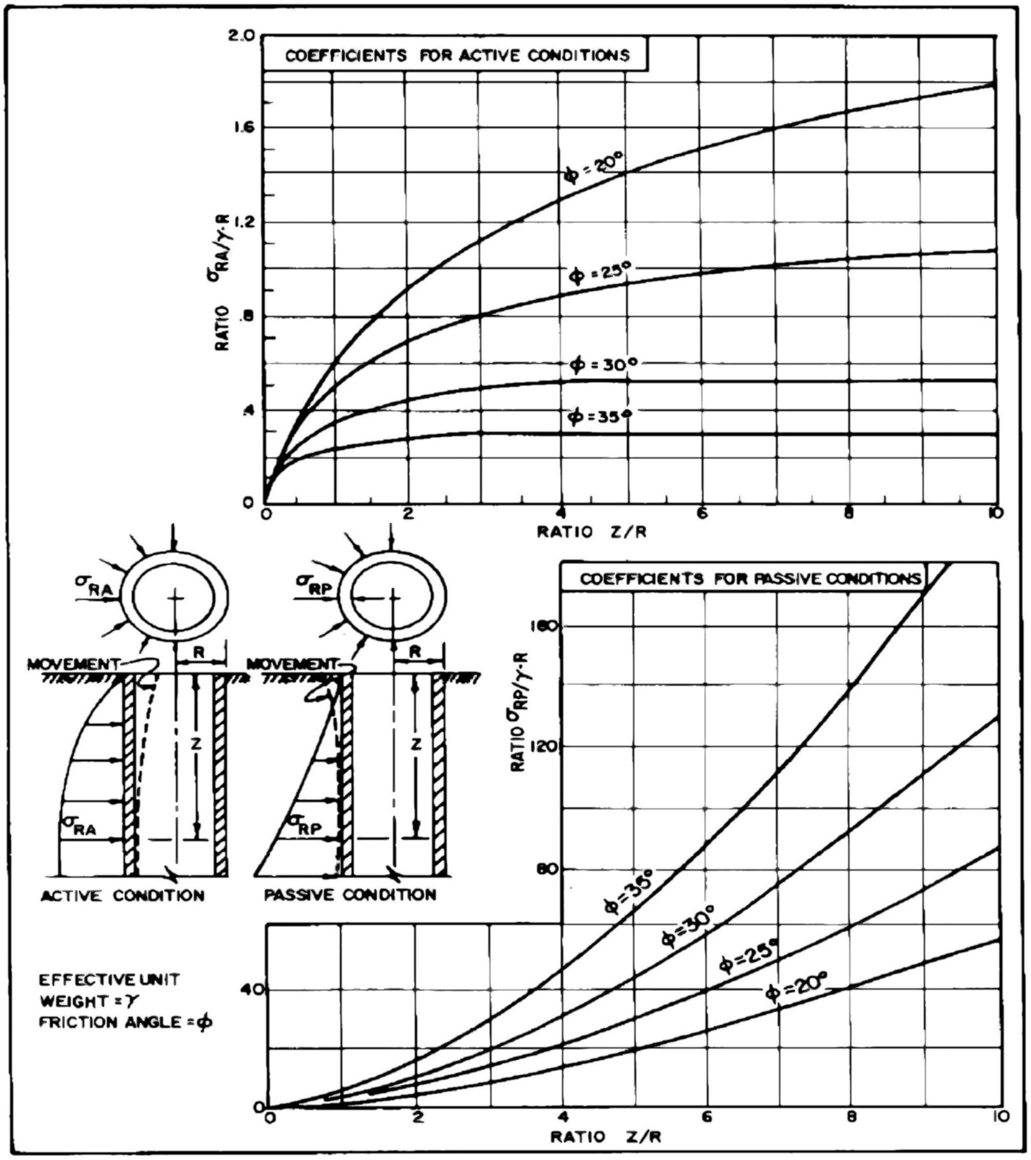
- p_c = air pressure in pounds per square foot
- q_u, = unconfined compressive strength of ground above roof in pounds per square foot
- γ = unit weight of soil in pounds per cubic foot
- t = stand up time, minutes
- T = elapsed time between excavating and completion of permanent structure, minutes
- H = vertical distance between ground surface and tunnel roof in feet
- H_p = design load in feet of earth
- H_t = height of tunnel
- B = width of tunnel

5.3.4. Pressure on Vertical Shafts

5.3.4.1. Shaft in Sand

- In the excavation of a vertical cylindrical shaft granular soil, pressures surrounding the shaft approach active values. If outward directed forces from a buried silo move the silo walls into the surrounding soil, pressures approach passive values as an upper limit.
  - Pressure Coefficients. See Figure 5-22 for active and passive pressure coefficients for a cylindrical shaft of unlimited depth in granular soils.
  - Modification of Active Pressures. For relatively shallow shafts (depth less than twice the diameter), rigid bracing at the top may prevent development of active conditions. In this case, horizontal pressures may be as large as at-rest pressures on a long wall with plane strain in the surrounding soil.
  - If groundwater is encountered, use submerged unit weight of sand and add hydrostatic pressure.

Figure 5-22 Coefficients for Active and Passive Pressure on Underground Cylindrical Shafts or Silos



5.3.4.2. Shaft in Clay

- Pressure on Walls of Shafts in Soft Clay. For a cylindrical shaft, no support is needed from the ground surface to a depth of $Z_o = 2c/\gamma$. To determine the approximate value of ultimate horizontal earth pressure on a shaft lining at any depth z, use

Equation 5-9: $ph = \gamma z - c$

Where

- γ = effective unit weight of clay
- z = depth
- c = cohesion

This pressure is likely to occur after several months.

- Pressure on Walls of Shafts in Stiff Clay. On shafts located in stiff, intact, or fissured swelling clays, initially the pressure on the shaft lining is very small. Over a period, the pressure may increase to several times the overburden pressure (i.e., ultimately to the swelling pressure if shaft lining is sufficiently rigid). Local experience in that soil or field measurements can provide useful information.

5.4. Shear Strength in Soils

5.4.1. Importance of Soil Strength

The most important property of soils is strength. Slopes of all kinds, including hills, riverbanks, and man-made cuts and fills, stay in place only because of the strength of the material of which they are composed. Knowledge of soil strength is important for the design of structure foundations, embankments, retaining walls, pavements, and cuts.

Basic concepts indicate a soil can derive its strength from two sources: friction between particles, and cohesion between particles.

- Cohesionless soils, such as gravel, sand, and silt, derive strength from friction between particles. The frictional resistance between soil particles is dependent on the overburden pressure above the particles and the angle of internal friction between the particles.
- Cohesive soils, composed mainly of clay, derive strength from the attraction, or bond, between particles.
- Mixtures of cohesionless and cohesive soils derive strength from both friction between particles and cohesion.

The entire strength of the soil from these considerations is expressed by the Mohr-Coulomb equation:

Equation 5-10: $$\tau_f = \sigma'_n \tan\phi + c$$

Where

- τ_f = Shear strength of the soil at the failure plane
- $\sigma'_n = \overline{\sigma}_v$ = Effective stress of soil
- c = Soil cohesion
- ϕ = Internal angle of friction[125]

This is illustrated in Figure 5-23.

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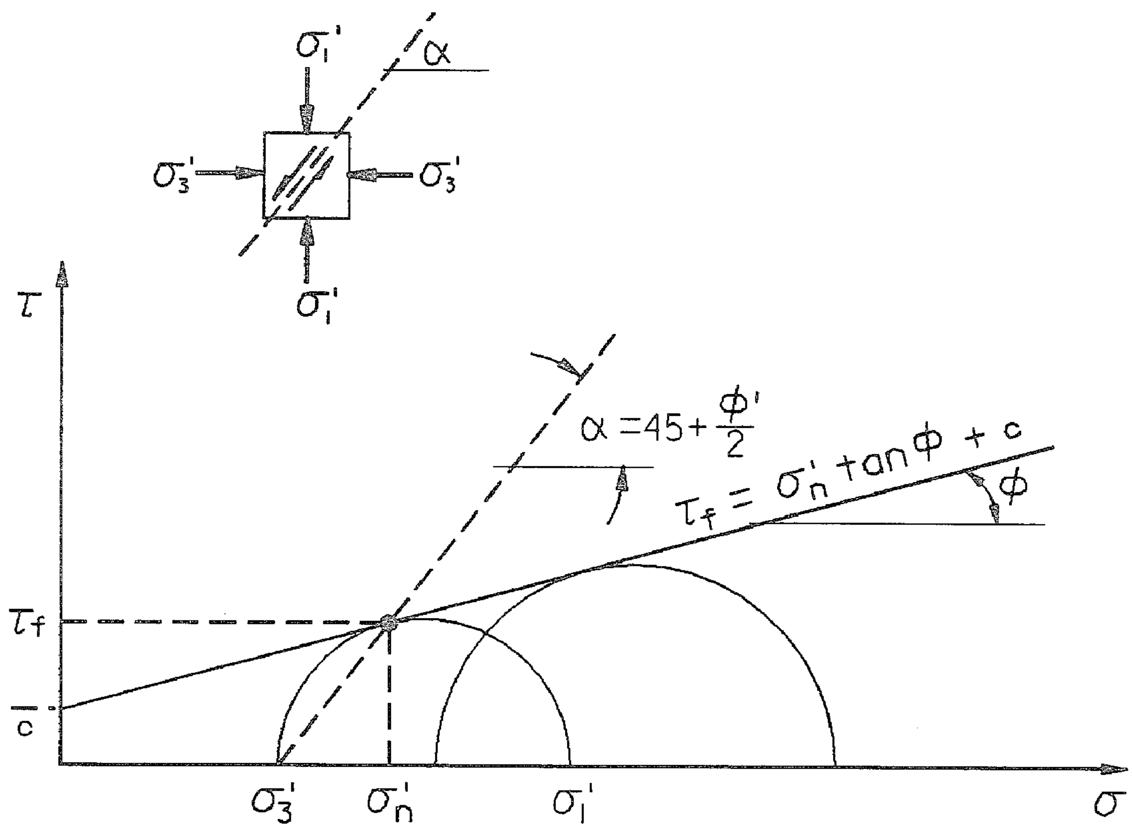
[125] In retaining wall analysis of cohesive soils, ϕ and c are taken from drained tests for long-term analysis ($\phi = \phi'$, $c = c'$) and undrained tests ($\phi = 0$, $c = q_u/2$) for short-term analysis.

Figure 5-23 Mohr-Coulomb Failure Criterion



For example, a pile of dry sand will have a maximum angle of repose of 30º. This is approximately equal to the friction angle between soil particles (30º). A purely cohesionless soil would have an angle of internal friction and a cohesion c = 0, in which case all of the strength of the soil would be derived form the product of internal friction and effective stress. The strength of granular soil increases immediately as the load increases.

The concept of cohesive strength is more difficult to explain, as the cohesion is dependent on nebulous quantities such as the ionic bond between soil mineral grains. However, the practical aspect is easily understood in the relation to granular soils. The strength of a pure cohesive soil increases very slowly after load is applied since consolidation is required for strength gain. Therefore, placement of highways on cohesive soils must be controlled to prevent exceeding the soil cohesive strength. With a purely cohesive soils, $\phi = 0$ and the soils strength is purely dependent upon the cohesion of the soil.

In reality, most cohesive clay deposits contain some non-cohesive silt or sand. Hence, under an increased load some increase in soil strength can be expected. Conversely many "cohesionless" soils also display cohesion, although in most cases this is "apparent" cohesion and should not be used for engineering calculations.

5.4.2. Selection of Type of Stress Analysis

The selection of the type of stress analysis is also connected with the type of strength test. A table of these can be found in Table 3-3.

∞ Total Stress Analysis. It is appropriate for the immediate (during and end of construction) safety of foundations and structures (embankments) consisting of or resting on clays where permeability is low. It is also applicable to embankment stability where rapid drawdown can occur. Use of unconfined compression tests or UU test is appropriate. Sample disturbance has significant effect on shear strength in these types of tests.

- Effective Stress Analysis. Evaluation of long-term stability of slopes, embankments, and earth supporting structures in cohesive soil requires the use of effective stress strength parameters, and therefore CU tests with pore water pressure measurements or CD tests are appropriate. Tests must be run at a slow enough strain rate so that pore pressures are equalized during the CU test or are dissipated throughout the CD test. Essentially all analyses of granular soils are made using effective stress.
- Stress Path Method. The stress path method is based on modelling the geological and historical stress conditions, as they are known to influence soil behaviour. To apply the method, stress history is determined and future stresses are computed based on actual construction plans. The stresses are modelled in a set of triaxial or similar strength tests. More information on this is found at 7.3.5.2.2.2.

5.4.3. Comparison of Laboratory and Field Strengths

Laboratory soil samples are obtained from the ground by sampling from boreholes, sealing, and transporting these samples to the laboratory. The degree of disturbance affecting the samples will vary according to the type of soil, sampling method and the skill of the driller. At best, some disturbances will occur from the removal of *in situ* stresses during sampling and laboratory preparation for testing. In general, disturbance tends to reduce the shear strength obtained from unconfined or unconsolidated tests and increase the strength obtained from consolidated tests. There is, therefore, considerable attraction for measuring shear strength in the field, *in situ*.

The vane shear test is the most commonly used field test for obtaining shear strength in soft to medium clays. As the test is performed rapidly, the strength measured as indicative of the undrained shear strength. In reviewing different types of field and lab testing in clays to determine the undrained shear strength, the designer should expect the vane shear test to provide the most accurate value with U and UU tests yielding lower results and CU tests yielding higher results.

5.4.4. Selection of Design Shear Strength

Frequently, on a large project the designer will receive a huge quantity of undrained shear strength test results from both the field and lab. This mountain of data must be concisely summarized to permit rational interpretation of results. The tests should be analysed on a hole-by-hole basis. All tests from one hole should be reviewed and the existing undrained shear strengths selected. The results for each type of test should be plotted versus depth to determine the pattern of strength variation for each test type with depth and to assess the reliability of the data, i.e., a CU test result that is lower than the U test result at the same depth should be suspect. The general pattern of *in situ* shear strength results should be to increase with depth in the same clay deposit. Clays that have been overconsolidated may only exhibit this increase at greater depths as the amount of preconsolidation increases shear strength.

5.5. Drained (S) Direct Shear Test

5.5.1. Apparatus

The apparatus should consist of the following:

a. Shear box of bronze or stainless steel, open at the top and divided horizontally into two frames that can be fitted together accurately with alignment pins and elevating screws. A schematic diagram of a direct shear box is shown in Figure 5-24. The lower frame of the shear box shall contain a reservoir for water, with the bottom grooved or provided with a grooved area to permit drainage. The upper frame of the shear box should contain an accurately machined piston, the bottom of which is also grooved to permit drainage. The upper frame shall be provided with horizontal locking screws to lock it to the piston within the upper frame of the box. The various metal parts of the shear box shall be of a like, noncorrosive material. A typical shear box, assembled and unassembled, is shown in Figure 5-25; a narrow-edged shear box as shown is preferable to a wide-edged one. Generally, shear boxes for direct shear tests shall have minimum inside dimensions of 3" x 3". The maximum thickness of a 3" x 3" specimen shall be 1/2" after consolidation. If the soil to be tested contains particles larger than the No. 4 sieve, the test should be performed in a larger shear box or else the shear strength should be determined by means of the triaxial test.

Figure 5-24 Schematic diagram of direct shear box

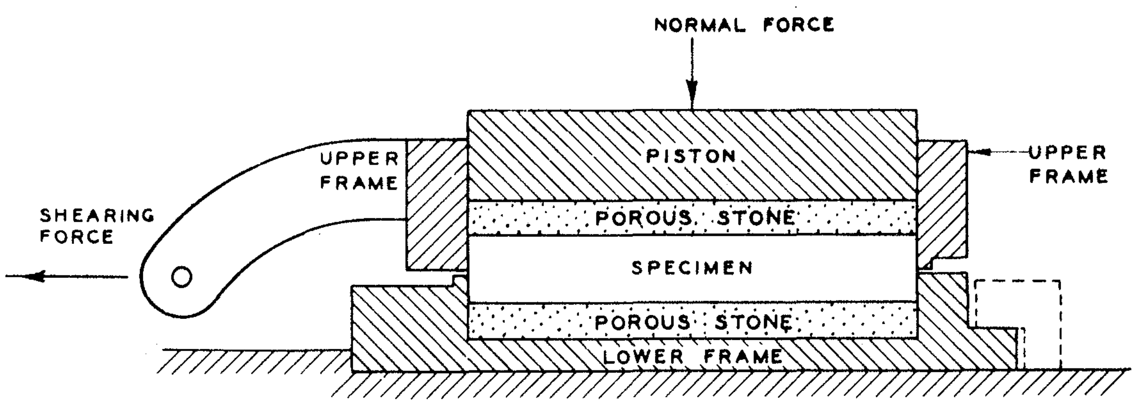
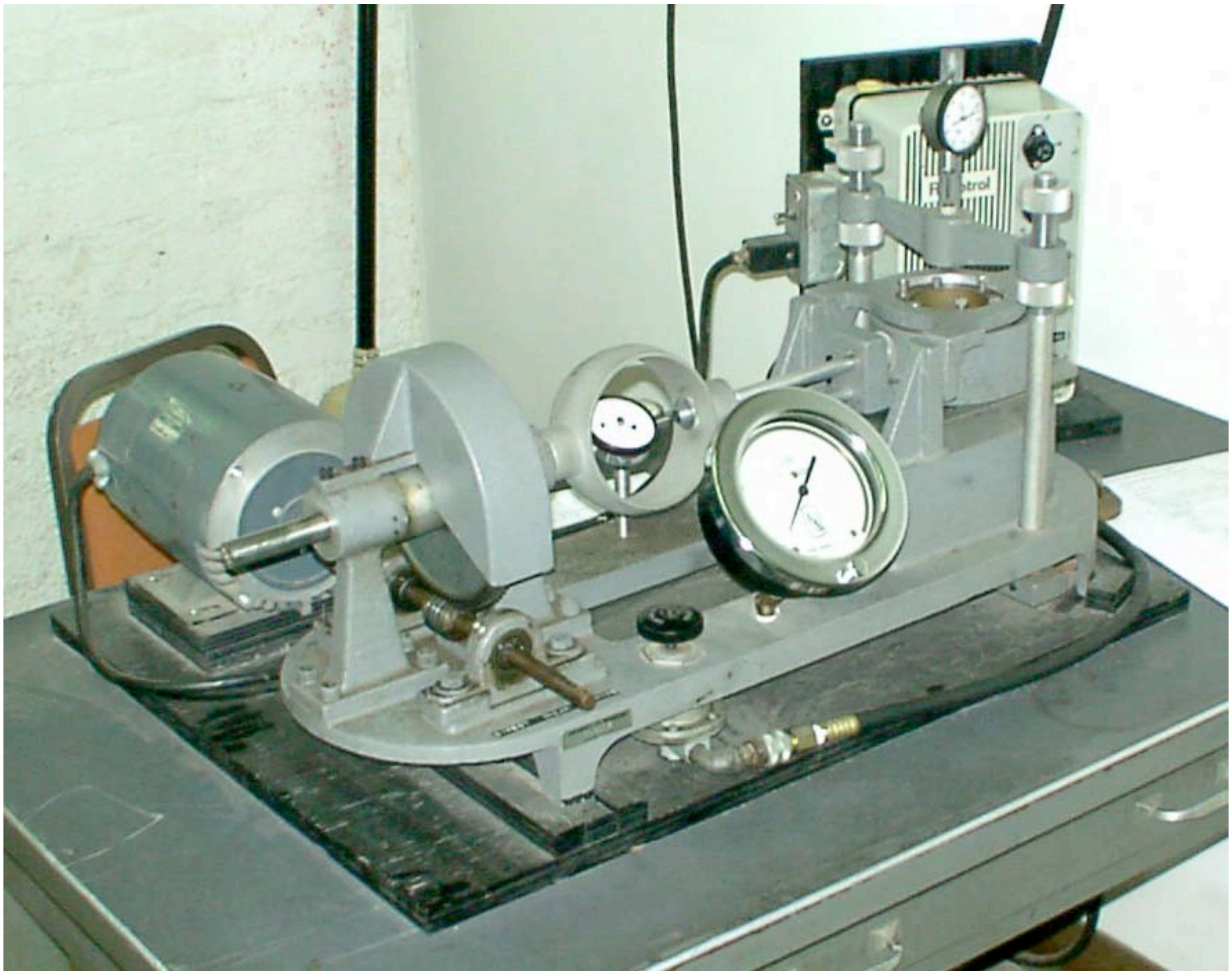
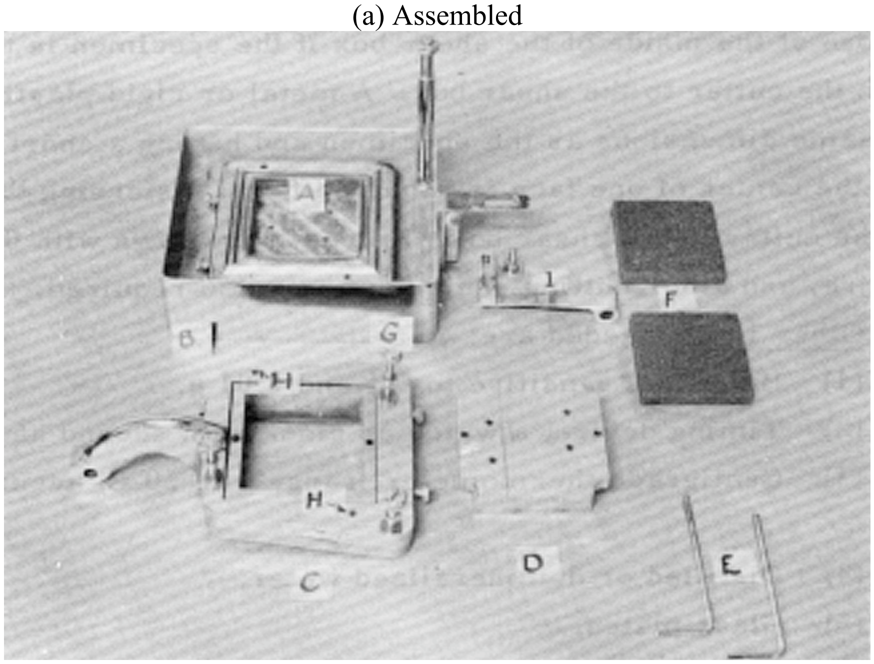


Figure 5-25 Typical Direct Shear Box



(a) Assembled



(b) Unassembled

A, lower frame; B, water reservoir; C, upper frame; D, piston; E, alignment pins; F, porous stones; G, elevating screws; H, holes for alignment pins; I, bracket for vertical dial indicator.

b. Porous stones shall be smooth, coarse grade Alundum or Carborundum B finish-ground except for the surface in contact with the specimen, which shall be rough-finished by sandblasting or by using hand tools. Porous metal plates of similar porosity and texture may also be used. It is very important that the permeability of porous stones not be reduced by the collection of soil particles in the pores of the stones; hence, frequent checking and cleaning (by flushing and boiling, or by ultrasonic cleaner) are required to ensure the necessary permeability (see 7.7.2).

c. Loading devices for applying the normal load and horizontal shear force to the specimen. Any type of loading device may be used that meets the following requirements:

   (1) For applying normal load. The equipment for applying the normal load shall be capable of transmitting the load to the specimen quickly, without impact, and maintaining the load constant for the duration of the test. The equipment should be calibrated to ensure that the loads indicated are those actually applied to the soil specimen.

   (2) For applying shear force. The horizontal shear force may be applied by either controlled-stress or controlled-strain methods, though the controlled-strain method is preferred in that the ultimate, as well as the maximum, stress can be determined. Controlled-stress equipment should be capable of applying the horizontal force in increments to the specimen in the same manner as that described above for the normal load. Controlled-strain equipment should be capable of shearing the specimen at a uniform rate of strain and should permit adjustment of the rate of strain over a relatively wide range. The controlled straining of the specimen is usually done with a motor and gearbox arrangement, and a load-indicating device such as a proving ring or frame determines the shear force.

d. Dial indicators for measuring (1) vertical deformation of the specimen, having a range of 0.25” and an accuracy of 0.0001”; and (2) horizontal displacement of the specimen, having a range of 0.5” and an accuracy of 0.001”.

e. Equipment for preparing specimen including a specimen cutter with sharp cutting edges. The cutter shall have inside dimensions the same as those of the inside of the shear box if the specimen is to be transferred from the cutter to the shear box. A metal or rigid plastic plate having the same dimensions as the specimen and having a short handle attached at the centre of one face is required for transferring the specimen from the cutter to the shear box. Knives, wire saws with 0.010” diameter wire, and other cutting equipment are also required.

f. Other items needed are:

   (1) Balances, sensitive to 0.1 and 0.01 g.

   (2) Timing device, a watch or clock with second hand.

   (3) Centigrade thermometer, range 0 to 50º C, accurate to 0.1º C.

   (4) Distilled or demineralised water.

   (5) Glass plates.

   (6) Apparatus necessary to determine water content and specific gravity.

5.5.2. Preparation of Specimen

A sample sufficient to provide a minimum of three identical specimens is required. Specimens shall be prepared in a humid room to prevent evaporation of moisture. Progressive trimming in front of the specimen cutter generally prepares the specimen. However, satisfactory specimens of hard soils often may be obtained by cutting and trimming without using the specimen cutter. Specimens of very soft, sensitive soils may be obtained more consistently by pushing the cutter into the sample without preliminary trimming. Extreme cart shall be taken in preparing undisturbed specimens of sensitive soils to prevent disturbance of their natural structure. Preferably, specimens of compacted soil should be trimmed from samples compacted in a compaction mould, using a pressing or kneading action of a tamper having an area less than one-sixth the area of the sample. A less desirable procedure is to compact the soil to the desired density and water content directly in the shear box, in a single layer using a similar kneading action. The procedure usually used in preparing specimens by progressive trimming follows:

a. Cut a sample of soil approximately 1 1/4" high and 4 1/2" in diameter from the sample to be tested.

b. Place the sample of soil on a glass plate and centre the specimen cutter on top of the sample. Push the cutter vertically into the sample not more than 1/4" and carefully trim the soil from the edge of the cutter. Repeat the operation until the specimen protrudes above the top of the cutter. Care should be taken to insure that no voids are formed between the cutter and specimen.

c. Remove the portion of the specimen protruding above the cutter, using a wire saw for soft specimens, and a straightedge, knives, or other convenient tools for harder specimens. Trim the specimen flush with the top of the cutter. If a pebble or other protrusion is encountered on the surface, remove it and fill the void with soil.

d. Place a previously weighed glass plate on the surface of the specimen. Many soils will adhere to glass; consequently, it is advisable to use waxed paper or similar material between the specimen and glass plate. Invert the specimen, trim the bottom as described in step b, and on this surface place another weighed glass plate.

e. From the soil trimmings obtain 200 g of material for water content and specific gravity determinations.

f. Repeat the procedures outlined above to produce two additional specimens.

5.5.3. Procedure

5.5.3.1. Preliminary

The procedure for setting up the test specimen shall consist of the following:

(1) Record all identifying information for the specimen, such as project, boring number, and other pertinent data, on the data sheet (Figure 11-9 is a suggested form); note any difficulties encountered in preparation of the specimen. Measure the inside area and height of the shear box and record as the initial dimensions of the specimen on the data sheet. Weigh and record the weight of specimen plus tare (specimen cutter and glass plates).

(2) Assemble the shear box with the upper frame held in alignment with the lower frame by means of the alignment pins or screws. Place a previously saturated porous stone, rough side up, on the base plate of the shear box in the bottom of the lower frame.

(3) Insert the specimen cutter, sharpened edge first, into the upper frame of the shear box until it is wedged firmly and is parallel with the top of the upper frame (the inside edge of the upper frame should be bevelled slightly to accept the cutter). Lay a piece of waxed paper, slightly smaller than the specimen, on the surface of the specimen, and with a smooth, continuous press of the transferring plate, force the specimen from the cutter and into firm contact with the porous stone. While pressing the specimen from the cutter, care must be exercised to prevent tilting or otherwise disturbing the specimen. Withdraw the transferring plate, remove the specimen cutter, and peel the waxed paper from the specimen. Place a previously saturated porous stone, rough side down, on top of the specimen and lower the piston onto this porous stone.

(4) Place the shear box in position on the loading apparatus. At this stage of the test, the upper and lower frames are in contact. Assemble the loading equipment, and mount the two dial indicators to be used for measuring vertical and horizontal deformation. The dial indicator measuring vertical deformation should be set so that it can measure movement in either direction.

5.5.3.2. Consolidation

The procedure for consolidating the specimen shall consist of the following:

(1) Apply the desired normal load gently to the specimen. The range of normal loads for the three specimens will depend on the loadings expected in the field. The maximum normal load should be at least equal to the maximum normal load expected in the field in order that the shear strength data need not be extrapolated for use in design analysis. Generally, normal loads less than about 6 ksf may be applied in a single increment, whereas greater normal loads should be applied in several increments to prevent the soil from squeezing out of the box, For very soft soils it is usually necessary to apply even the relatively lighter normal loads in increments.

(2) As soon as possible after applying the normal load, fill the water reservoir with distilled or demineralised water to a point above the top of the specimen. Maintain this water level during the consolidation and subsequent shear phases so that the specimen is at all times effectively submerged.

Allow the specimen to drain and consolidate under the desired normal load or increments thereof prior to shearing. During the consolidation process, record on the data sheet (Figure 11-24) the vertical dial readings after various elapsed times. Readings at 0.1, 0.2, 0.5, 1, 2, 4, 8, 15, and 30 minutes, and 1, 2, 4, 8, and 24 hours for each increment of normal load are usually satisfactory. During the course of the test, plot the dial readings versus elapsed time on a semilogarithmic plot (Figure 7-50). Allow each load increment to remain on the specimen until it is determined that primary consolidation is complete.

5.5.3.3. Shear Test

The procedure for shearing the specimen after consolidation shall consist of the following:

(1) Raise the upper frame of the shear box about 1/16" by turning the elevating screws. The amount of clearance between the upper and lower frames should be sufficient to prevent the two frames from coming in contact during the shear test, yet not permit the soil to extrude between the frames. Lock the upper frame to the loading piston by means of the horizontal locking screws. In raising the upper frame, the applied load on the specimen is increased by an amount equal to the weight of the upper frame. Adjust vertical load by reducing applied load by this amount. Retract the elevating screws.

(2) Remove the alignment pins.

(3) Shear the specimen at a relatively slow rate so that a fully drained condition (no excess pore pressures) exists at failure. The following equation shall be used as a guide in determining the minimum time required from start of test to shear failure:

Equation 5-11: $t_f = 50t_{50}$

Where

- t_f = total elapsed time to failure in minutes

- t_{50} = time in minutes required for the specimen to achieve 50% consolidation[126] under the normal load or increments thereof (see Figure 7-50).

It is to be noted, however, that time-consolidation curves indicated by soils that exhibit a tendency to swell under a given increment of normal load are not meaningful and, therefore, cannot be used in determining minimum times required to failure. In such instances, the following procedures may be used to obtain valid time-consolidation curves:

a. An increment of normal load is applied to the specimen and the specimen is inundated with water and allowed to come to equilibrium; the time-consolidation curve for any increment of normal stress applied thereafter is valid.

b. Alternatively, the specimen may be inundated with water following the completion of primary consolidation under the final increment of normal load. However, the specimen must be allowed to come to equilibrium after inundation (prior to shear.) Prior to inundation, the specimen should be maintained in a humid atmosphere by covering the shear box and filling the water reservoir with moist paper towels, cotton, or other cellular material.

(4) Considerable experience and judgment are generally required in determining the proper rate of shear load application. The following discussions will provide guidance in this respect:

a. Controlled-stress test. The rate of load application may be determined approximately by the following procedure. Estimate the maximum shear stress and select an initial load increment of about 10% of the estimated failure load. Apply each increment to the specimen and permit at least 95% consolidation before applying the next increment. The time-consolidation curve obtained during the consolidation sequence of the test may be used for determining if 95% consolidation has been achieved. When 50 to 70% of the estimated failure load has been applied to the specimen (depending on the shape of the stress-deformation curve), reduce the size of the increments to one-half the initial size. As failure is approached, use a series of increments equal to one-fourth of the initial load increment to accurately define the failure load.

b. Controlled-strain test. The rate of strain may be determined approximately by dividing the estimated horizontal deformation at maximum shear stress by the computed time to failure, t_f. The test shall be continued until the shear stress becomes essentially constant, as shown in Figure 5-26b, or until a horizontal deformation of 0.5" has been reached.

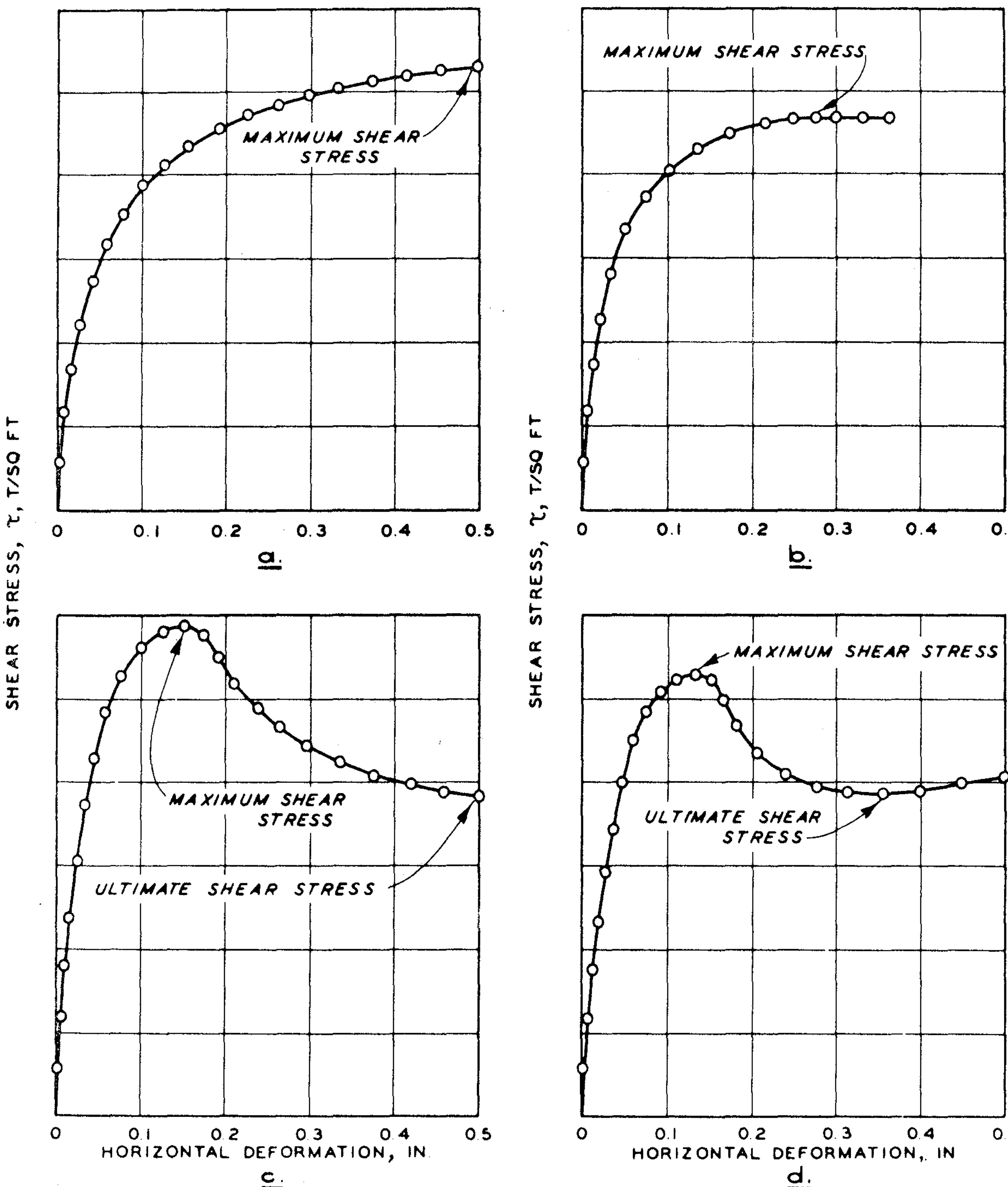
(5) The horizontal and vertical deformations and the applied shear force shall be observed at convenient intervals. Figure 11-10 is a suggested form for recording the observations.

(6) Remove the specimen from the shear box, blot any excess moisture, and trim away a minimum of 1/16" from all sides of the specimen to form a rectangular block.

(7) Determine the water content of the specimen. The dry weight of the specimen should be computed using the water content based on specimen trimmings taken at the start of the test.

[126] If the time for 50% consolidation is difficult to determine, values for higher percentages of consolidation may be used to compute t_f. The following relations may be used: $t_f = 35t_{60} = 25t_{70} = 18t_{80} = 12t_{90}$

Figure 5-26 Examples of stress-deformation curves



5.5.4. Computations

The computations shall consist of the following:

a. From the recorded data compute and record on the data sheet (see Figure 11-9) the initial and final water contents. Compute also the void ratio before test, after consolidation, and after test, and the initial and final degrees of saturation, and dry density before test, using the following equations:[127]

Equation 5-12: $$e_o = \frac{V_o - V_s}{V_s}$$

Equation 5-13: $$e_s = \frac{V_c - V_s}{V_s}$$

Equation 5-14: $$e_f = \frac{V_f - V_s}{V_s}$$

Equation 5-15: $$S_o = 100\frac{\frac{w_o}{100}\frac{W_s}{\gamma_w}}{V_o - V_s} = \left[\frac{w_o W_s}{V_o - V_s}\right]$$

Equation 5-16: $$S_f = 100\frac{\frac{w_f}{100}\frac{W_s}{\gamma_w}}{V_f - V_s} = \left[\frac{w_f W_s}{V_f - V_s}\right]$$

Equation 5-17: $$\gamma_d = \left[62.4\frac{W_s}{H_o A}\right]$$

Where

- e_o = Void ratio before test
- e_c = Void ratio after consolidation
- e_f = Void ratio after test
- S_o = Initial degree of saturation, percent
- S_f = Final degree of saturation, percent
- γ_d= Dry density before test, lb/ft^3
- W_s = weight of dry soil, g
- A = area of specimen cm^2
- G_s = specific gravity of solids
- γ_w = unit weight of water, g/cm^3
- w_o = water content of specimen before test, percent
- w_f = water content of specimen after test, percent
- H_o = initial height of specimen, cm
- H_c = height of specimen after consolidation, cm

[127] Equations in brackets are based on units of measurement shown on the following page in explanation of symbols.

- H_f = height of specimen at end of test, cm
- V_s = volume of solids, $cm^3 = W_s/G_s$
- V_o = volume of specimen before test, cm^3 = A x H_o
- V_c = volume of specimen after consolidation, cm^3 = A x H_c
- V_f = volume of specimen after test, cm^3 = A x H_f

Units of measurement are those commonly used in computations for the direct shear tests.

b. Complete the data sheet, Figure 11-10.

c. The shear stress, τ, in kips/square foot may be calculated from the following equation:

Equation 5-18: $$\tau = 0.93\frac{F}{A}$$

Where

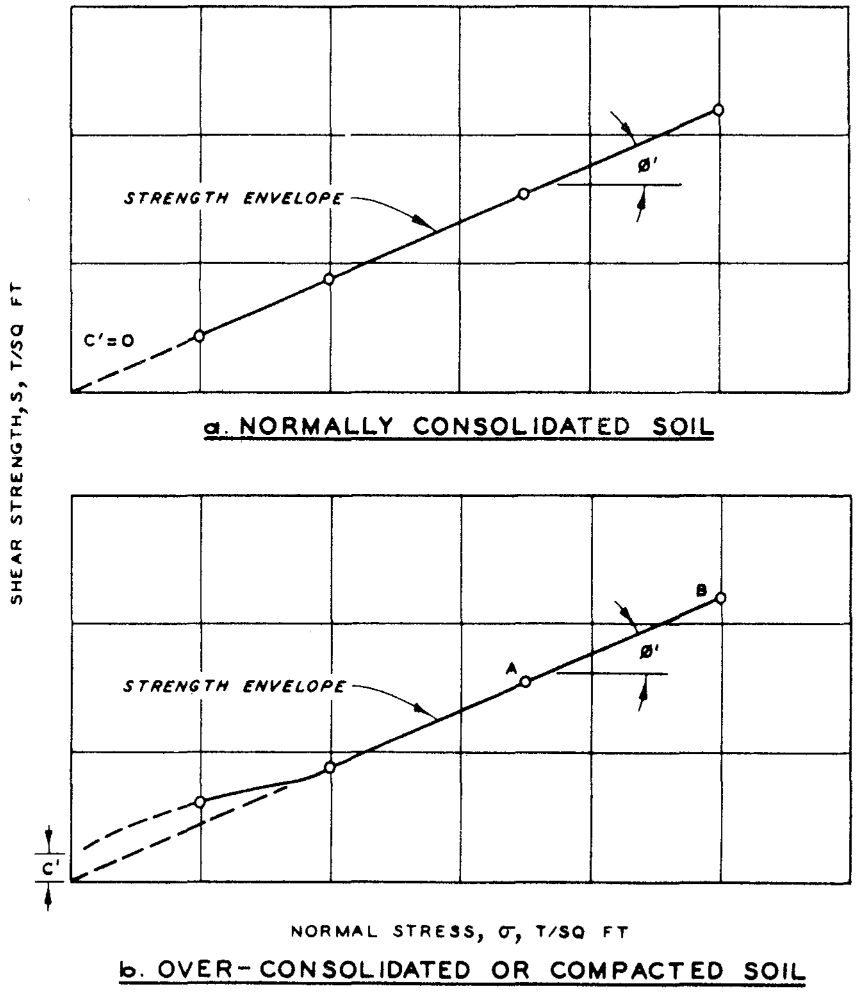
- F = applied shear force, lb.
- A = horizontal cross-sectional area of the specimen, cm_ (assumed to be constant and equal to the initial area for routine testing)

5.5.5. Presentation of Results

The results of the direct shear test shall be shown on the report form, Figure 11-11. The shear stress and vertical deformation during shear shall be plotted versus the horizontal deformation. As shown in Figure 5-26, the maximum shear stress, τ_{max}, is either the actual maximum or peak shear stress or, if the shear stress increases continuously during the test, the shear stress at 0.5" horizontal deformation. When the shear stress decreases after reaching a maximum value, the minimum shear stress attained before 0.5" horizontal deformation is considered to be the ultimate shear stress, τ_{ult}, as shown in Figure 5-26c and Figure 5-26d. The time to failure, t_f, is the elapsed time between the start of shear and the maximum shear stress. The maximum shear stress shall be plotted against the normal stress, as shown in Figure 5-27, and the strength envelope drawn to determine the drained or effective angle of internal friction, ϕ. In normally consolidated soils (see Figure 5-27a), the strength envelope is based on normal stresses greater than any past or existing overburden pressure.

A brief description of undisturbed specimens should be given on the report form under "Remarks." The description should include color, approximate consistency, and any unusual features (such as stratification, fissures, roots, shells, sand pockets, etc.) For compacted specimens, give method of compaction used and the relation to maximum density and optimum water content.

Figure 5-27 Examples of Strength Envelopes



5.5.6. Simplified Procedure for Drained (S) Direct Shear Tests

In problems involving the long-term stability of fine-grained soils, the cohesion intercept, c', due to overconsolidation is generally ignored. The drained direct shear test may be performed by shearing duplicate specimens under the maximum normal stress expected in the field. The strength envelope for determining the effective angle of internal friction, ϕ', shall be drawn as a straight line from the origin to the average value of the maximum shear stresses under this normal stress. Regardless of the magnitude of the expected normal stress, the normal stress used in this simplified procedure for testing compacted specimens must be at least 6 ksf to avoid the effects of prestressing caused by the compaction. Although the testing of duplicate specimens under the same normal stress is satisfactory for relatively pervious or normally consolidated soils, it may not be conservative for preconsolidated clays. For the latter soils, the duplicate specimens should be sheared under different normal stresses, both of which are known to exceed the preconsolidation pressure, as illustrated by points A and B in Figure 5-27b.

5.5.7. Possible Errors

Following are possible errors that would cause inaccurate determinations of strength and stress-deformation characteristics:

a. Specimen disturbed while trimming. The trimming of specimens must be done in the humid room with every care taken to minimize disturbance of the natural soil structure or change in the natural water content. As a rule, the effect of trimming disturbance is inversely proportional to the size of the specimen.

b. Specimen disturbed while fitting into shear box. The specimen must exactly fit the inside of the shear box to insure complete lateral confinement, yet a pretrimmed specimen must be inserted without flexing or compressing. The specimen cutter must have the identical inside dimensions as those of the shear box.

c. Galvanic action in shear box. To prevent any change in the strength or stress-deformation characteristics due to galvanic currents in tests of long duration, all metal parts of the shear box should be of the same noncorrosive material.

d. Permeability of porous stones too low. Unless the porous stones are frequently cleaned, they may become clogged by soil particles and full drainage of the specimen inhibited.

e. Slippage between porous stone and specimen. When testing undisturbed firm or stiff clays, particularly under low normal loads, it may not be possible to transfer the required shear force to the specimen by means of the standard porous stone. In such a case, slippage of the porous stone will result and a portion of the shear force will be applied to the specimen by the rear edge of the upper frame. The slippage may be marked by tilting -of the upper frame and the development of an inclined shear plane through the upper rear corner rather than one through the mid-height of the specimen. The use of dentated porous stones or of wire cloth or abrasive grit between the stone and the specimen may be necessary to effect the transfer of shear stress.

f. Rate of strain too fast. The time to failure in the drained (S) direct shear test must be long enough to achieve essentially complete dissipation of excess pore pressure at failure. The criterion given should be considered as no more than an approximate guide to the minimum time to failure; twice this time may be necessary for some soils. In general, it is safer to shear too slowly.

5.6. Drained (S) Repeated Direct Shear Test

5.6.1. Apparatus

The apparatus may be similar to that described in 5.5.1. A controlled strain device in which the direction of shear is capable of being reversed should be used to apply the horizontal shear force to the specimen.

5.6.2. Specimen Preparation

a. Specimens should be prepared using procedures similar to those described in 5.5.2; special saws may be required for trimming stiff-to-hard materials. Special care must be exercised to ensure that the specimen is not subjected to air-drying during or after trimming operations.

b. The specimen should consist of two pieces of intact material trimmed to fill the inside of the shear box or confining ring. The two pieces should be of approximately equal height and have a total height not in excess of 1” (preferably, the total height should be 0.5”, but this is often not practical for stiff, fissured materials). The top and bottom surfaces of each piece should be plane and parallel. A close fit of each piece to the inside of the shear box is necessary. Stiff-to-hard materials may be cut to shape with a band saw or, in the case of very hard materials, with a diamond wheel.

c. From the soil trimmings obtain about 200 g of material for water content and specific gravity determinations.

5.6.3. Procedure

5.6.3.1. Preliminary

The procedure for setting up the test specimen shall consist of the following:

(1) Record all identifying information for the specimen, such as project title, boring number, sample number, and other pertinent data, on the data sheet (Figure 11-9 is a suggested form); note any difficulties encountered in preparation of the specimen. Determine the dimensions of the specimen and record as the initial dimensions of the specimen on the data sheet. Weigh and record the weight of specimen (plus tare, if used).

(2) The lower half of the specimen should be firmly seated against a saturated porous plate in the lower half of the shear box. A 0.010" to 0.020" projection of the lower half of the specimen above the lip of the box is desirable; certainly, the top of this half of the specimen should not be initially below the lid. Then the upper half of the specimen should be placed in the shear box, the upper porous plate (saturated) added, and the remainder of the shear apparatus assembled. A specimen of softer material may be precut inside the shear box. In this case, an intact specimen is firmly seated between saturated porous plates in the apparatus; then, a plane should be cut with a small-diameter (0.008" to 0.044" diameter) steel wire through the specimen at the separation between the upper and lower halves of the box. After cutting, the two halves of the specimen should be separated, and the cut surfaces inspected for planeness. Any irregularities should be removed with a straightedge.

(3) Place the shear box in position on the loading apparatus. At this time, the upper and lower frames are in contact. Assemble the loading equipment and mount the two dial indicators or other deformation measuring apparatus to be used for measuring vertical and horizontal deformation. Both indicators should be set so that they can measure deformation in either direction.

5.6.3.2. Consolidation

The procedure for consolidating the specimen shall consist of the following:

(1) Apply the normal stress to the specimen without impact. A single value of normal stress may be used. A standard value of about 12 ksf is recommended as being high enough to prevent the swelling of most clay shale material yet low enough to minimize the problem of soil extruding from between the two halves of the box during shear. In addition, tests under higher normal stresses may be used to determine whether the strength envelope is a straight line.

(2) As soon as possible after applying the normal stress, fill the water reservoir with distilled or demineralised water to a point above the top of the specimen. Maintain this water level during the consolidation (or swell) and subsequent shear phases so that the specimen is at all times effectively submerged.

(3) The specimen should be allowed to consolidate or swell to an essentially equilibrium condition under the normal stress; a minimum period of 16 hours should be allowed before shear.

5.6.3.3. Repeated Shear Test

The procedure for shearing the specimen after consolidation shall consist of the following:

(1) A gap should be formed between the two halves of the box to ensure that normal and shear stresses are borne only by soil. This gap should be kept between 0.015" to 0.025" to minimize extrusion of remolded soil from the shearing surface. Periodically during the test, the gap should be checked by inserting thickness gages, and adjusted as needed.

(2) Remove the alignment pins.

(3) Shearing displacement should be initiated at a controlled rate not in excess of 0.5"/day (about 0.00035"/min). Shear movement under constant normal stress should be continued with a reversal of direction after about 0.25" displacement to each side of the starting position until a minimum shearing resistance is attained. A semilogarithmic plot of shear stress (arithmetic scale) versus cumulative shear displacement (logarithmic scale) should be maintained during the test to show when a minimum value has been reached; only the shear stress measured at the midpoint of each shearing stroke (that is, when the two halves of the shear box are aligned vertically) should be plotted.

(4) Observations of vertical and horizontal deformations and the applied shear force sufficient to define the stress deformation curve for each shearing stroke shall be made.

(5) If, after the standard test is complete, the effects of increased normal stress and decreased displacement rate are to be studied, this information should also be obtained according to uniform procedures.

   a. First, the normal stress should be doubled while the two halves of the shear box are vertically aligned, and the specimen should be permitted to come to equilibrium (minimum of 16 hours). Shear displacement should be initiated at a rate of about 0.5"/day and continued until a minimum shearing resistance is reached.

   b. Second, the effect of decreased rate of displacement should be determined as follows. After the minimum shearing resistance is reached under the high normal stress (approximately 24 ksf) and at a displacement rate of about 0.5"/day, the rate of displacement should be reduced to a tenth of the standard rate (that is, to about 0.05"/day) without any change in the normal stress. The rate of displacement should be reduced soon after the upper half of the shear box has passed through the initial, vertically aligned position. Shearing with repeated reversals of direction as described above should be continued until a minimum shearing resistance is reached.

(6) Remove the specimen from the shear box, blot any excess moisture, and determine the water content of the specimen.[128] The dry weight of the specimen should be computed using the water content based on specimen trimmings taken during specimen preparation.

5.6.4. Computations

See 5.5.4.

5.6.5. Presentation of Results

Report forms have not been standardized; the method of presentation of results will be determined by the project (design) engineer requesting the tests.

5.6.6. Possible Errors

The following are possible errors that may cause inaccurate determinations of strength and stress-deformation characteristics:

a. Air -Drying of Specimen during Preparation. The trimming of specimens should be done in a humid room with every precaution taken to prevent change in natural water content. Air-drying may cause the specimen to slake readily when inundated with water, and thus change the strength or stress-deformation characteristics.

[128] If considerable remolded material exists in the shear zone, it should be removed and its water content should be determined in addition to that of the remainder of the specimen.

b. Top and Bottom Surfaces of Each Half of a Specimen Not Plane and Parallel. Irregular surfaces may introduce a geometric component to the measured shearing resistance.

c. Too Large a Gap. Maintaining too large a gap between the upper and lower frames of the shear box may result in excessive extrusion of the specimen.

d. Absence of Gap. A gap must be maintained throughout the test to prevent the normal load from being borne by the lower frame of the shear box.

e. Inaccurate Measurement of Shear Stress. Because of the very small shear resistances offered by some clay shale materials, measurement of shear stress must be very precise.

f. Permeability of Porous Stones Too Low. Unless the porous stones are frequently cleaned, they may become clogged by soil particles and ingress or egress of water to or from the specimen may be inhibited.

g. Galvanic Action in Shear Box. To prevent any change in the strength or stress-deformation characteristics due to galvanic currents in tests of long duration, all metal parts of the shear box should be constructed of the same noncorrosive material.

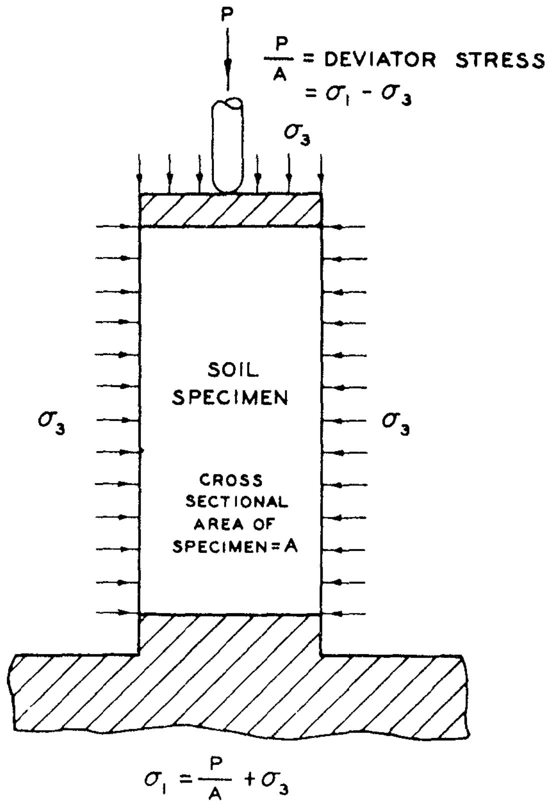
h. Stopping Test Too Soon. The test must be carried to a cumulative shear deformation sufficient to establish that the minimum shear resistance offered by a specimen under a given normal stress has indeed been determined. A semilogarithmic plot of shear stress (arithmetic scale) versus cumulative shear displacement (logarithmic scale) is essential in making this determination.

5.7. Triaxial Compression Tests

5.7.1. Principles of the Triaxial Compression Test

The triaxial compression test is used to measure the shear strength of a soil under controlled drainage conditions. In the basic triaxial test, a cylindrical specimen of soil encased in a rubber membrane is placed in a triaxial compression chamber, subjected to a confining fluid pressure, and then loaded axially to failure. Connections at the ends of the specimen permit controlled drainage of pore water from the specimen. The procedures presented herein apply only to the basic test conducted with limited drainage conditions, and do not include special types or variants of this test. In general, a minimum of three specimens, each under a different confining pressure, are tested to establish the relation between shear strength and normal stress. The test is called "triaxial" because the three principal stresses are known and controlled. Prior to shear, the three principal stresses are equal to the chamber fluid pressure. During shear, the major principal stress, σ_1 is equal to the applied axial stress (P/A) plus the chamber pressure, σ_3 (see Figure 5-28). The applied axial stress, $\sigma_1 - \sigma_3$, is termed the "deviator stress." The intermediate principal stress, σ_2, and the minor principal stress, σ_3 are identical in the test, and are equal to the confining or chamber pressure hereafter referred to as σ_3.

Figure 5-28 Diagram showing stresses during triaxial compression test



A soil mass may be considered as a compressible skeleton of solid particles. In saturated soils, the void spaces are completely filled with water; in partially saturated soils the void spaces are filled with both water and air. Only the skeleton of solid particles carries shear stresses, whereas both the solid particles and the pore water carry the normal stress on any plane. In a triaxial test, the shear strength is determined in terms of the total stress (intergranular stress plus pore water pressure), unless (a) complete drainage is provided during the test so that the pore pressure is equal to zero at failure, or (b) measurements of pore pressure are made during the test. When the pore pressure at failure is known, the shear strength can be computed in terms of the stress carried by the soil particles (termed effective or intergranular stress). In recent years, significant advances have been made in the techniques of measuring pore pressures in the triaxial test and in the interpretation of the data obtained; however, difficulties still exist in this respect. Pore pressure measurements during shear are seldom required in routine investigations, as the basic triaxial tests are sufficient to furnish shear strengths for the limiting conditions of drainage. Procedures for measuring pore pressures in the triaxial test during shear are discussed elsewhere[129] and are beyond the scope of this procedure.

5.7.2. Types of Tests

The three types of basic triaxial compression tests are unconsolidated-undrained, consolidated-undrained, and consolidated-drained, subsequently referred to as the Q (or UU), R (or CU), and S (or CD) tests, respectively. As these names imply, they are derived from the drainage conditions allowed to prevail

[129] A. W. Bishop and D. J. Kenkel, *The Measurement of Soil Properties in the Triaxial Test*, 2nd ed. (London, Edward Arnold Ltd., 1962).

during the test. The type of test is selected to closely simulate, or to bracket, the conditions anticipated in the field.

5.7.3. Apparatus

5.7.3.1. Loading Device

Various devices may be used to apply axial load to the specimen. These devices can be classified as either apparatus in which axial loads are measured outside the triaxial chamber or apparatus in which axial loads are measured inside the triaxial chamber by using a proving ring or frame, an electrical transducer, or a pressure capsule. Any equipment used should be calibrated to permit determination of loads actually applied to the soil specimen.

Loading devices can be further grouped under controlled-strain or controlled-stress types. In controlled-strain tests, the specimen is strained axially at a predetermined rate; in controlled-stress tests, predetermined increments of load are applied to the specimen at fixed intervals of time.

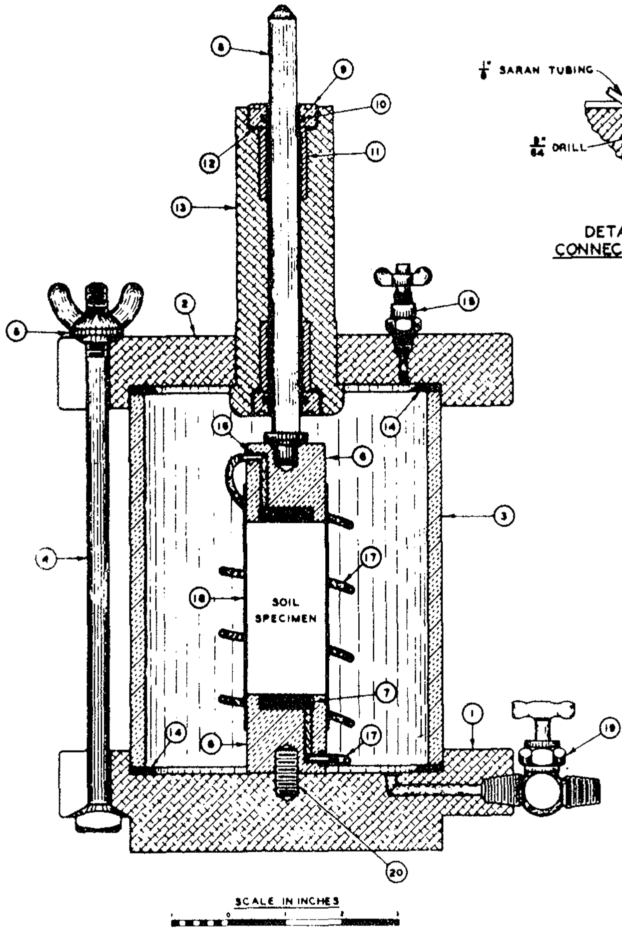
Controlled-strain loading devices, such as commercial testing machines, are preferred for short-duration tests using piston-type test apparatus. If available, an automatic stress-strain recorder may be used to measure and record applied axial loads and strains.

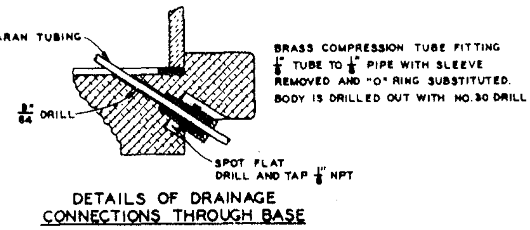
5.7.3.2. Triaxial Compression Chamber

The triaxial compression chamber consists primarily of a head plate and a base plate separated by a transparent plastic cylinder.[130] A drawing of a typical triaxial compression chamber for 1.4” diameter specimens is shown in Figure 5-29. Chamber dimensions and type will vary depending on the size of specimen tested and on pressure and load requirements. The base plate has one inlet through which the pressure liquid is supplied to the chamber and two inlets leading to the specimen base and cap to permit saturation and drainage of the specimen when required. The head plate has a vent valve so that air can be forced out of the chamber as it is filled with the pressure fluid. The cylinder is held tightly against rubber gaskets by bolts or tie rods connecting the head plate and base plate.

[130] Adequate safety precautions should be taken, or the transparent plastic cylinder should be replaced by a metal cylinder, if chamber pressures in excess of 100 psi are used.

Figure 5-29 Details of typical triaxial compression chamber



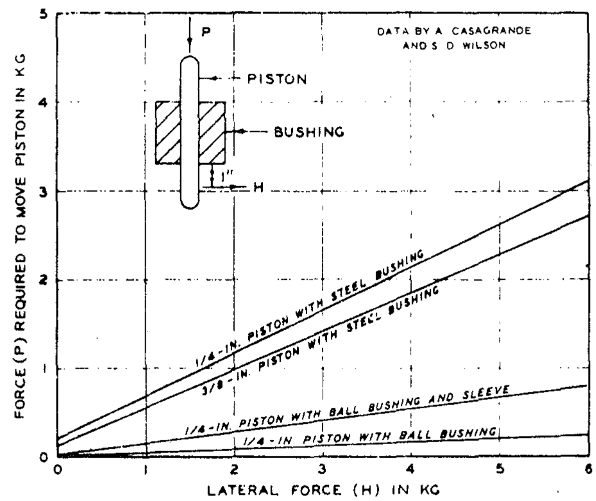


PART NO.	NAME	MATERIAL	QUANTITY
1	BASEPLATE	ALUMINUM	1
2	HEADPLATE	ALUMINUM	1
3	CYLINDER	TRANSPARENT PLASTIC	1
4	TIE ROD AND WING NUT	STAINLESS STEEL	3
5	TIE ROD WASHER	STAINLESS STEEL	3
6	SPECIMEN CAP AND BASE	PLASTIC	2
7	POROUS DISC	CERAMIC OR METAL	2
8	PISTON (GROUND AND POLISHED, CASE HARDENED) 1/2" DIAM	STEEL	1
9	"O" RING HOUSING	BRASS	2
10	11/16" O D "O" RING	RUBBER	2
11	LINEAR BALL BUSHING	STAINLESS STEEL	2
12	GASKET	FIBER	2
13	PISTON HOUSING	ALUMINUM	1
14	GASKET	RUBBER	2
15	1/8" N P T NEEDLE VALVE	BRASS	1
16	1/4" O D "O" RING	RUBBER	2
17	1/8" O D TUBING	PLASTIC	2
18	SPECIMEN MEMBRANE 1.4" I D , 5" LONG	RUBBER	
19	1/4" x 1/4" N P T NEEDLE VALVE OR A CIRCLE SEAL VALVE OF THE 9500 SERIES	BRASS	1
20	3/8" THREADED STUD	ALUMINUM	1

In piston-type test apparatus in which the axial, loads are measured outside the triaxial compression chamber, piston friction can have a significant effect on the indicated applied load, and measures should be taken to reduce friction to tolerable limits. Pistons generally should consist of ground and polished casehardened steel rods with diameters between 1/4" and 1/2" for testing 1.4" diameter specimens, heavier pistons are required for larger specimens. The following measures have been found to reduce piston friction to tolerable amounts.

(1) The use of linear ball bushings as shown in Figure 5-29.The unique design of these bushings permits unlimited axial movement of the piston with a minimum of friction. Leakage around the piston is reduced by means of O-rings, Quad-rings, flexible diaphragms, or other devices. A seal incorporating O-rings is shown in Figure 5-29. The beneficial effects of using linear ball bushings in comparison with steel bushings are demonstrated by the data shown in Figure 5-30. The amount of lateral force transmitted to the piston, if the specimen cap tends to tilt during a test, cannot be determined; however, the data shown in Figure 5-30 indicate that the resulting piston friction would be negligible even for relatively large lateral forces.

Figure 5-30 Effect of lateral force on piston friction in triaxial compression apparatus

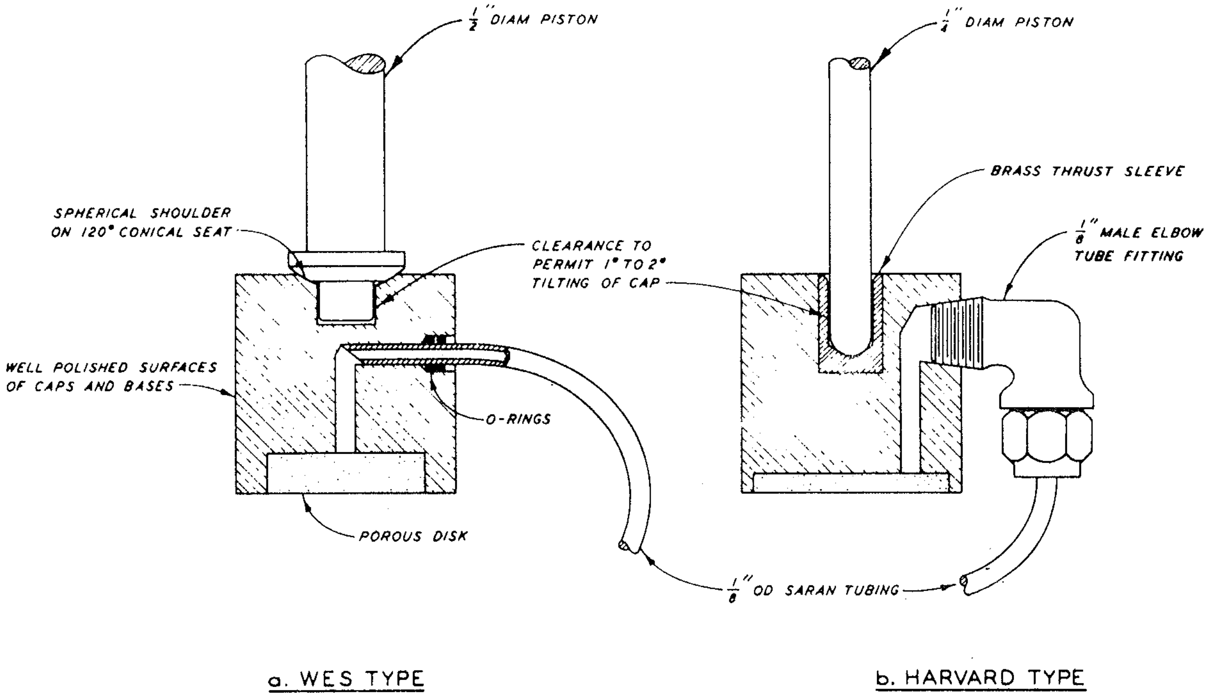


(2) Rotation of the piston within the bushings during the application of the deviator stress. (Commercial devices are available to rotate the piston during the test.) This method is very effective in reducing friction; however, a more complex design of the specimen cap is necessary, and unless the piston is rotated continuously, appreciable friction would still develop during long-time tests. When linear ball bushings are used, the piston should never be rotated except under special conditions designated by the manufacturer. Although these measures will reduce piston friction to negligible amounts during the course of the test, it is always preferable to measure the actual piston friction before the start of the test. This can readily be done by starting the axial load application with the bottom of the piston raised slightly above the top of the specimen cap. Thus any starting friction or residual friction, as indicated by the load necessary to move the piston down into contact with the cap, can be subtracted from the measured load.

5.7.3.3. Specimen Caps and Base

Specimen caps and bases should be constructed of a lightweight noncorrosive material and should be of the same diameter as the test specimen in order to avoid entrapment of air at the contact faces. Solid caps and bases should be used for the Q test to prevent drainage of the specimens. Caps and bases with porous metal or porous stone inserts and drainage connections, as shown in Figure 5-31, should be used for the R and S tests. The porous inserts should be more pervious than the soil being tested to permit effective drainage. For routine testing, stones of medium porosity are satisfactory. The specimen cap should be designed to permit slight tilting with the piston in contact position, as shown in Figure 5-31.

Figure 5-31 Details of Typical 1.4" Diameter Specimen Caps Showing Drainage and Piston Seats



5.7.3.4. Rubber Membranes

Rubber membranes used to encase the specimen should provide reliable protection against leakage, yet offer minimum restraint to the specimen. Commercially available rubber membranes having thicknesses ranging of 0.0025" (for soft clays) to 0.010" (for sands or for clays containing sharp particles) are generally satisfactory for 1.4" diameter specimens. Rubber membranes about 0.010" or greater in thickness are suitable for larger specimens. Membranes should be carefully inspected prior to use, and if any flaws or pinholes are evident, the membranes should be discarded. The use of two thin membranes separated by a thin film of silicone grease will afford protection against leakage through a undetected pinhole and will minimize the possibility of air leakage from the chamber fluid into the specimen during tests of relatively long duration. Since no rubber membrane is completely impervious, the use of special membranes or chamber fluids may sometimes be necessary, such as during periods of undrained shear that exceed a few hours. The membrane is sealed against the cap and base by rubber O-rings or rubber bands. Leakage around the ends of the membrane, where it is sealed against the cap and the base, as well as through fittings, valves, etc., can develop unless close attention is given to details in the manufacture and use of the apparatus.[131]

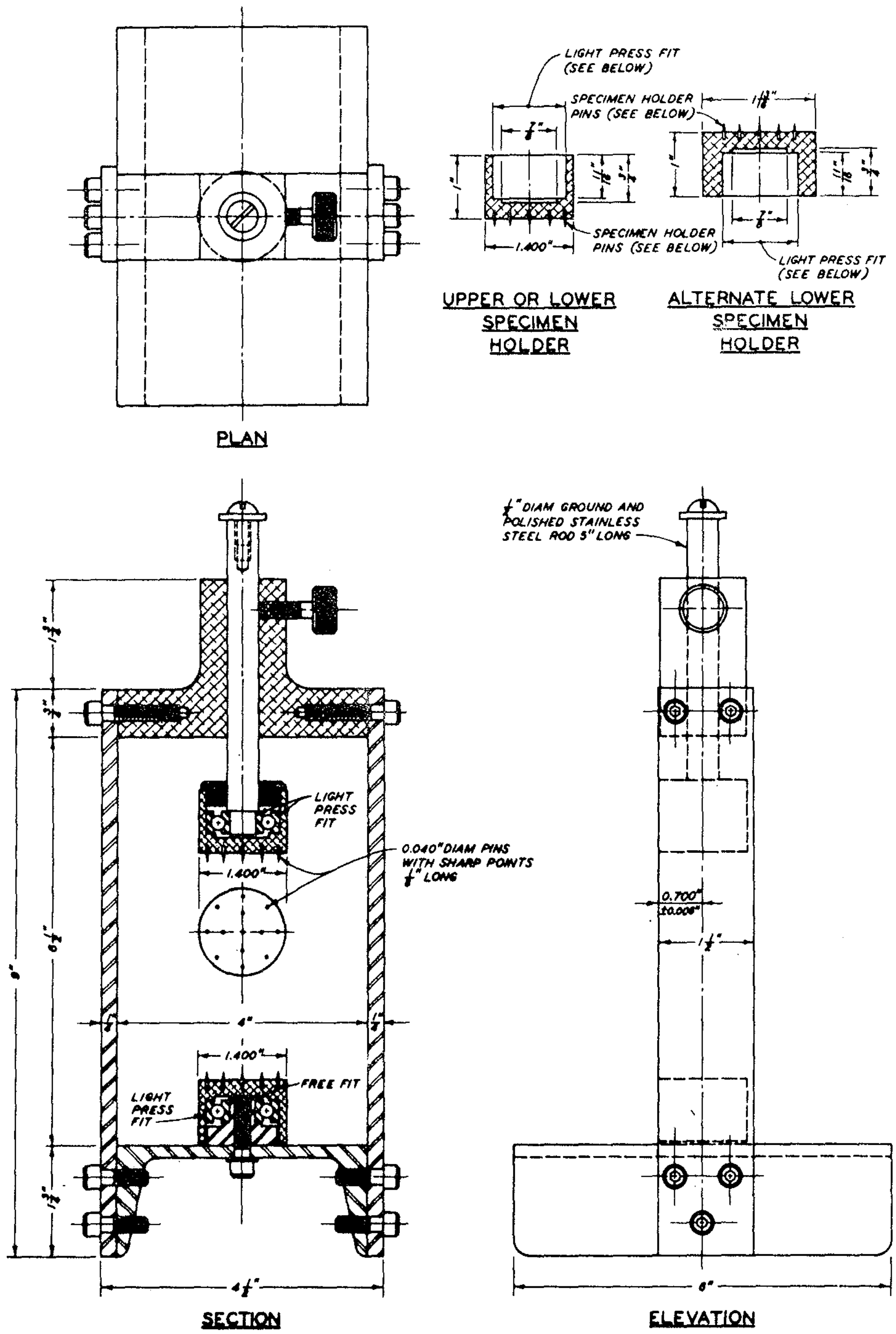
5.7.3.5. Equipment for Preparing Specimen

(1) Cohesive Soils. A specimen-trimming frame is recommended for preparing specimens of most cohesive soils. The specimen is held in a vertical position between two circular plates containing pins that press into the ends of the specimen to prevent movement during trimming. The edges of the trimming frame act as vertical guides for the cutting equipment and control the final diameter of the specimen. Details of a typical trimming frame for 1.4" diameter specimens are shown in Figure 5-32.

[131] S. J. Poulos, *Report on Control of Leakage in the Triaxial Test*, Soil Mechanics Series No. 71, Harvard University (Cambridge, Mass., March 1964).

Wire saws and knives of various sizes and types are used with the trimmer (see Figure 5-34). Split or solid cylinders with a beveled cutting edge can also be used to trim specimens. The use of a motorized soil lathe may be advantageous in reducing the time required for preparing specimens of certain types of soils. A miter box or cradle (see Figure 5-35) is required to trim the specimen to a fixed length and to insure that the ends of the specimen are parallel with each other and perpendicular to the axis of the specimen.

Figure 5-32 Details of a trimming frame for preparing 1.4” diameter specimens



(2) Cohesionless soils. A forming jacket consisting of a split mould that encloses a rubber membrane is required for cohesionless soils. The inside diameter of the mould minus the double thickness of the membrane is equal to the diameter of the specimen required. A funnel or special spoon (see Figure 6-19) for placing the material inside the jacket and a tamping hammer or vibratory equipment for compacting the material is necessary.

(3) Soils containing gravel. Large-size forming jackets, the dimensions of which will depend on specimen size requirements subsequently described, are necessary for preparing specimens of material containing gravel. Special compacting equipment is also necessary for such soils, depending on the type of soil and the procedures used.

5.7.3.6. Equipment for Using Back Pressure to Saturate Specimens

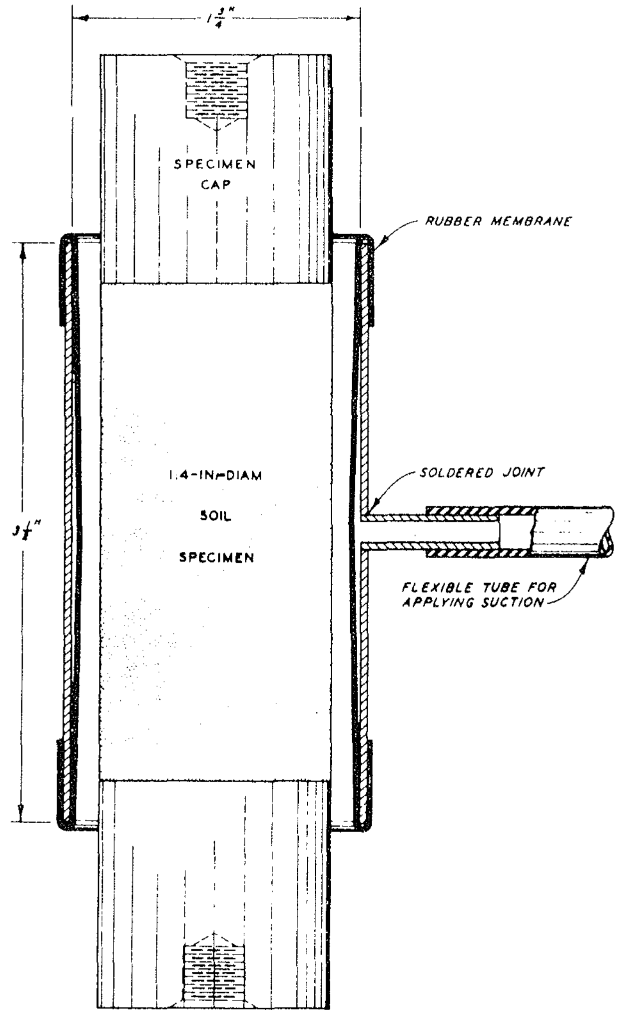
Special equipment required for saturating specimens by using backpressures is described in paragraph 6a.

5.7.3.7. Miscellaneous Equipment

Other items of equipment needed for triaxial compression tests are as follows:

(1) Membrane stretcher. A cylindrical tube, larger in diameter than the soil specimen, which has a tube connected to its side for application of a vacuum. Details of a membrane stretcher for 1.4” diameter specimens are shown in Figure 5-33.

Figure 5-33 Details of a membrane stretcher for 1.4” diameter specimens



(2) Pressure reservoir, generally a metal tank. The reservoir is filled with the fluid (usually deaired water) for applying the chamber pressure and is provided with a pressure regulator and a Bourdon gage. The regulator should be capable of controlling pressures to within ±1/2 %, though more precise methods of controlling and maintaining chamber pressures are required for tests of long duration.

(3) Measuring equipment, such as dial indicators and calipers. Precise instruments should be used for measuring the dimensions of a specimen with the desired accuracy.

(4) Deaired water, either distilled or demineralized.

(5) Vacuum and air pressure supply.

(6) Bourdon gages of various sizes and capacities.

(7) A timing device, either a watch or clock with second hand.

(8) Balances, sensitive to 0.01 g and to 0.1 g.

(9) Apparatus, necessary to determine water content and specific gravity.

5.7.4. Preparation of Specimens

Specimens shall have an initial height of not less than 2.1 times the initial diameter, though the minimum initial height of a specimen must be 2.25 times the diameter if the soil contains particles retained on the No. 4 sieve. The maximum particle size permitted in any specimen shall be no greater than one-sixth of the specimen diameter. Triaxial specimens 1.4, 2.8, 4, 6, 12, and 15" in diameter are most commonly used.

5.7.4.1. Cohesive Soils Containing Negligible Amounts of Gravel

Specimens 1.4" in diameter are generally satisfactory for testing cohesive soils containing a negligible amount of gravel, while specimens of larger diameter may be advisable for undisturbed soils having marked stratification, fissures, or other discontinuities. Depending on the type of sample, specimens shall be prepared by either of the following procedures:

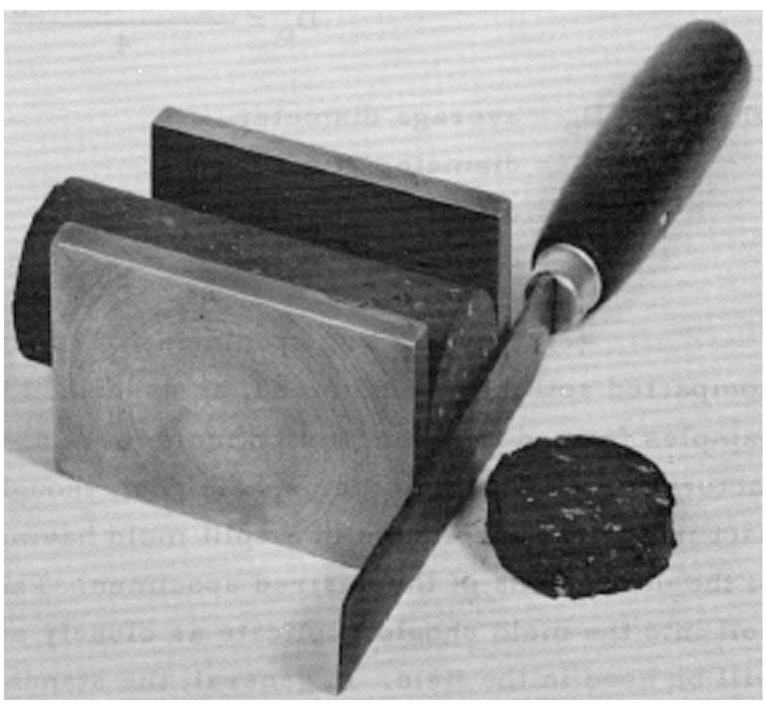
(1) Trimming specimens of cohesive soil. A sample that is uniform in character and sufficient in amount to provide a minimum of three specimens is required. For undisturbed soils, samples about 5" in diameter are preferred for triaxial tests using 4.4" diameter specimens. Specimens shall be prepared in a humid room and tested as soon as possible thereafter to prevent evaporation of moisture. Extreme care shall be taken in preparing the specimens to preclude the least possible disturbance to the structure of the soil. The specimens shall be prepared as follows:

   a. Cut a section of suitable length from the sample. As a rule, the specimens should be cut with the long axes parallel to the long axis of the sample; any influence of stratification is commonly disregarded. However, comparative tests can be made, if necessary, to determine the effects of stratification. When a 5" diameter undisturbed sample is to be used for 1.4" diameter specimens, cut the sample axially into quadrants using a wire saw or other convenient cutting tool. Use three of the quadrants for specimens; seal the fourth quadrant in wax and preserve it for a possible check test.

   b. Carefully trim each specimen to the required diameter, using a trimming frame or similar equipment (see Figure 5-34). Use one side of the trimming frame for preliminary cutting, and the other side for final trimming. A specimen after trimming is also shown in Figure 5-34. Ordinarily, pressing the wire saw or trimming knife against the edges of the frame and cutting from top to bottom trims the specimen. In trimming stiff or varved clays, move the wire saw from the top and bottom toward the middle of the specimen to prevent breaking off pieces at the ends. Remove any small shells or pebbles encountered during the trimming operations. Carefully fill voids on the surface of the specimen with remolded soil obtained from the trimmings. Cut specimen to the required length (usually 3 to 3 1/2" for 1.4" diameter specimens and 6 to 7" for 2.8" diameter specimens) using a mitre box, as shown in Figure 5-35.

Figure 5-34 Prepared triaxial specimen, trimming frame, and cutting tools

Figure 5-35 Squaring ends of specimen with a mitre box



c. From the soil trimmings, obtain 200 g of material for specific gravity and water content determinations.

d. Weigh the specimen to an accuracy of ± 0.01 g for 1.4" diameter specimens and ± 0.1 g for 2.8" diameter specimens.

e. Measure the height and diameter of the specimen to an accuracy of ± 0.01". Specimen dimensions based on measurements of the trimming frame guides and mitre box length are not sufficiently accurate. The average height H_o of the specimen should be determined from measurements, while the average diameter should be determined from measurements at the top, centre and bottom of the specimen as follows:

Equation 5-19: $$D_o = \frac{D_t + 2D_c + D_b}{4}$$

Where

- D_o = average diameter
- D_t = diameter at top
- D_c = diameter at centre
- D_b = diameter at bottom

(2) Compacting specimens of cohesive soil. Specimens of compacted soil may be trimmed from samples formed in a compaction mould (a 4" diameter sample is satisfactory for 1.4" diameter specimens), though it is preferable to compact individual specimens in a split mould having inside dimensions equal to the dimensions of the desired specimen. The method of compacting the soil into the mould should duplicate as closely as possible the method that will be used in the field. In general, the standard impact type of compaction will not produce the same soil structure and stress-deformation characteristics as the kneading action of the field compaction equipment. Therefore, the soil should preferably be compacted into the mould (whether a specimen-size or a standard compaction mould) in at least six layers, using a pressing or kneading action of a tamper having an area in contact with the soil of less than one-sixth the area of the mould, and thoroughly scarifying the surface of each layer before placing the next. The sample shall be prepared according to 9.7.2.2, thoroughly mixed with sufficient water to produce the desired water content, and then stored in an airtight container for at least 16 hours. The desired density may be produced by either (1) kneading or tamping each layer until the accumulative weight of soil placed in the mould is compacted to a known volume or (2) adjusting the number of layers, the number of tamps per layer, and the force per tamp. For the latter method of control, special constant force tampers (such as the Harvard miniature compactor for 1.4" diameter specimens[132] or similar compactors for 2.8" diameter and larger specimens[133]) are necessary. After each specimen compacted to finished dimensions has been removed from the mould, proceed in accordance with steps 1(c) through 1(e).

5.7.4.2. Cohesionless Soils Containing Negligible Amounts of Gravel

Soils that possess little or no cohesion are difficult if not impossible to trim into a specimen. If undisturbed samples of such materials are available in sampling tubes, satisfactory specimens can usually be obtained by freezing the sample to permit cutting out suitable specimens. Samples should be drained

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[132] A. Casagrande, J. M. Corso, and S. D. Wilson, *Report to Waterways Experiment Station on the 1949-1950 Program of Investigation of Effect of Long-Time Loading on the Strength of Clays and Shales at Constant Water Content*, Harvard University (Cambridge, Mass., July 1950).

[133] A. Casagrande and R. C. Hirschfeld, *Second Progress Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays*, Soil Mechanics Series No. 65, Harvard University (Cambridge, MA, April 1962)

before freezing. The frozen specimens are placed in the triaxial chamber, allowed to thaw after application of the chamber pressure, and then tested as desired. Some slight disturbance probably occurs because of the freezing, but the natural stratification and structure of the material are retained. In most cases, however, it is permissible to test cohesionless soils in the remolded state by forming the specimen at the desired density or at a series of densities that will permit interpolation to the desired density. Specimens prepared in this manner should generally be 2.8" in diameter or larger, depending on the maximum particle size. The procedure for forming the test specimen shall consist of the following steps:

(1) Oven-dry and weigh an amount of material sufficient to provide somewhat more than the desired volume of specimen.

(2) Place the forming jacket, with the membrane inside, over the specimen base of the triaxial compression device.

(3) Evacuate the air between the membrane and the inside face of the forming jacket.

(4) After mixing the dried material to avoid segregation, place the specimen, by means of a funnel or the special spoon, inside the forming jacket in equal layers. For 2.8" diameter specimens, 10 layers of equal thickness are adequate. Starting with the bottom layer, compact each layer by blows with a tamping hammer, increasing the number of blows per layer linearly with the height the layer above the bottom 1ayer.[134] The total number of blows required for a specimen of a given material will depend on the density desired. Considerable experience is usually required to establish the proper procedure for compacting a material to a desired uniform density by this method. A specimen formed properly in the above-specified manner, when confined and axially loaded, will deform symmetrically with respect to its midheight, indicating that a uniform density has been obtained along the height of the specimen.

(5) As an alternate procedure, the entire specimen may be placed in a loose condition by means of a funnel or special spoon. The desired density may then be achieved by vibrating the specimen in the forming jacket to obtain a specimen of predetermined height and corresponding density. A specimen formed properly in this manner, when confined and axially loaded, will deform symmetrically with respect to its midheight.

(6) Subtract weight of unused material from original weight of the sample to obtain weight of material in the specimen.

(7) After the forming jacket is filled to the desired height, place the specimen cap on the top of the specimen, roll the ends of the, membrane over the specimen cap and base, and fasten the ends with rubber bands or O-rings. Apply a low vacuum to the specimen through the base and remove the forming jacket.

(8) Measure height and diameter as specified for other specimens in this procedure.

5.7.4.3. Soils Containing Gravel

The size of specimens containing appreciable amounts of gravel is governed by the requirements of 5.7.4. If the material to be tested is in an undisturbed state, the specimens shall be prepared according to the applicable requirements of 5.7.4.1 and 5.7.4.2, with the size of specimen based on an estimate of the largest particle size. In testing compacted soils, the largest particle size is usually known, and the entire sample should be tested, whenever possible, without removing any of the coarser particles. However, it may be necessary to remove the particles larger than a certain size to comply with the requirements for

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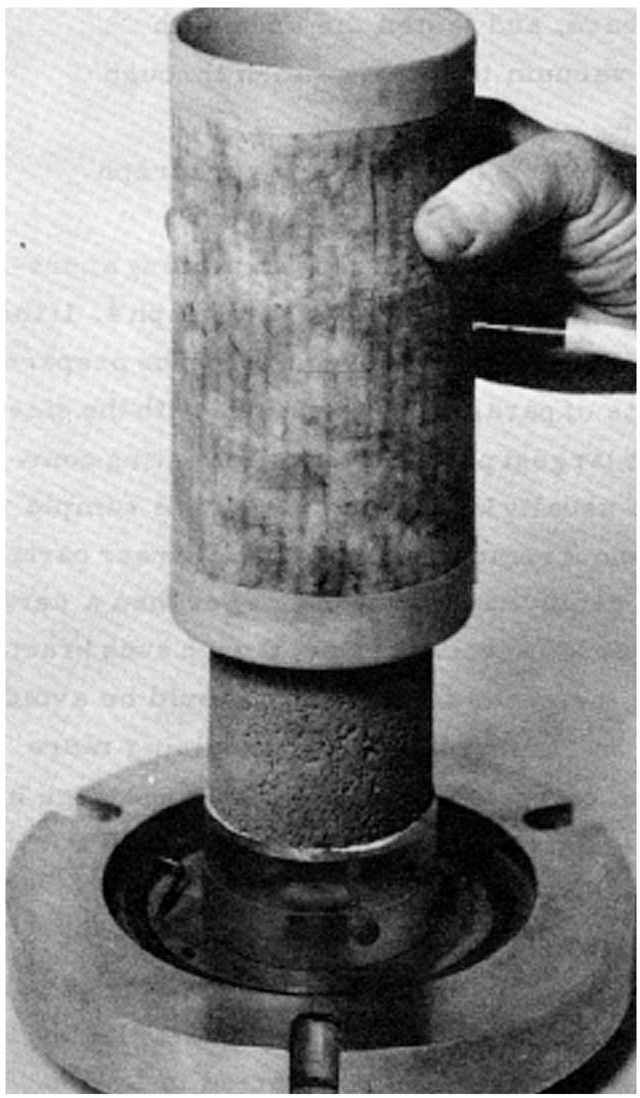
[134] Liang-Sheng Chen, "An investigation of stress strain and strength characteristics of cohesionless soils by triaxial compression tests," *Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering*, vol. V (Rotterdam, i948), pp. 35-43.

specimen size, though such practice will result in lower measured values of the shear strength and should be avoided if possible. Oversize particles should be removed and, if comprising more than 10% by weight of the sample, be replaced by an equal percentage by weight of material retained on the No. 4 sieve and passing the maximum allowable sieve size. The percentage of material finer than the No. 4 sieve thus remains constant (see 9.7). It will generally be necessary to prepare compacted samples of material containing gravel inside a forming jacket placed on the specimen base. If the material is cohesionless, it should be oven dried and compacted in layers inside the membrane and forming jacket using the procedure in 5.7.4.2 as a guide. When specimens of very high density are required, the samples should be compacted preferably by vibration to avoid rupturing the membrane. The use of two membranes will provide additional insurance against possible leakage during the test because of membrane rupture. If the sample contains a significant amount of fine-grained material, the soil usually must possess the proper water content before it can be compacted to the desired density. Then, a special split compaction mould is used for forming the specimen. The inside dimensions of the mould are equal to the dimensions of the triaxial specimen desired. No membrane is used inside the mould, as the membrane can be readily placed over the compacted specimen after it is removed from the split mould. The specimen should be compacted to the desired density.

Figure 5-36 Placing rubber membrane over a 2.8” diameter specimen using a membrane stretcher



5.7.5. Q Test

5.7.5.1. Procedure

The procedure for the Q test shall consist of the following steps:

(1) Record all identifying information for the sample project number or name, boring number, and other pertinent data, on a data sheet (see Figure 11-12).

(2) Place one of the prepared specimens on the base.

(3) Place a rubber membrane (see Figure 5-36) in the membrane stretcher, turn both ends of the membrane over the ends of the stretcher, and apply a vacuum to the stretcher. Carefully lower the stretcher and membrane over the specimen as shown in Figure 5-36. Place the specimen cap on the top of the specimen and release the vacuum on the membrane stretcher. Turn the ends of the membrane down around the base, up around the specimen cap, and fasten the ends with O-rings or rubber bands. With 1.4” diameter specimens of relatively insensitive soils, it is easier to roll the membrane over the specimen as shown in Figure 5-37.

Figure 5-37 Rolling rubber membrane over a 1.4” diameter specimen

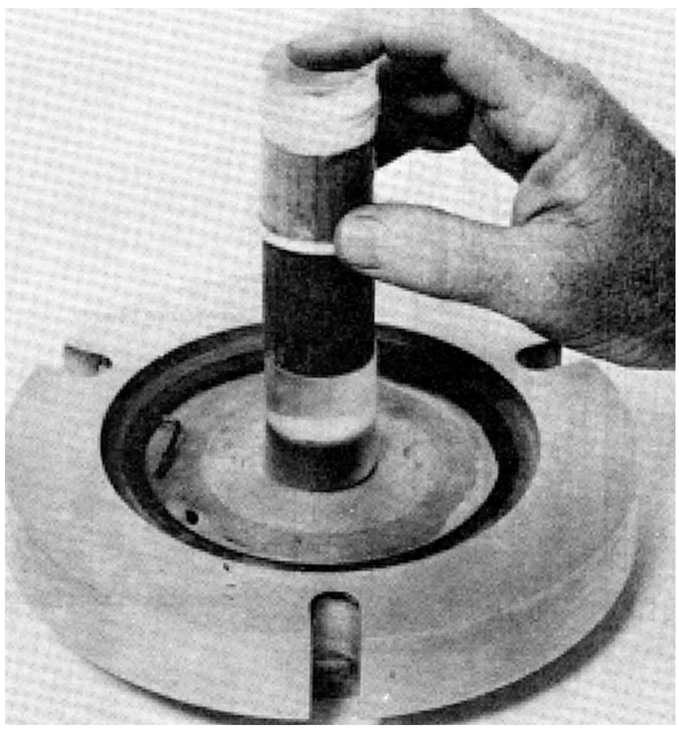
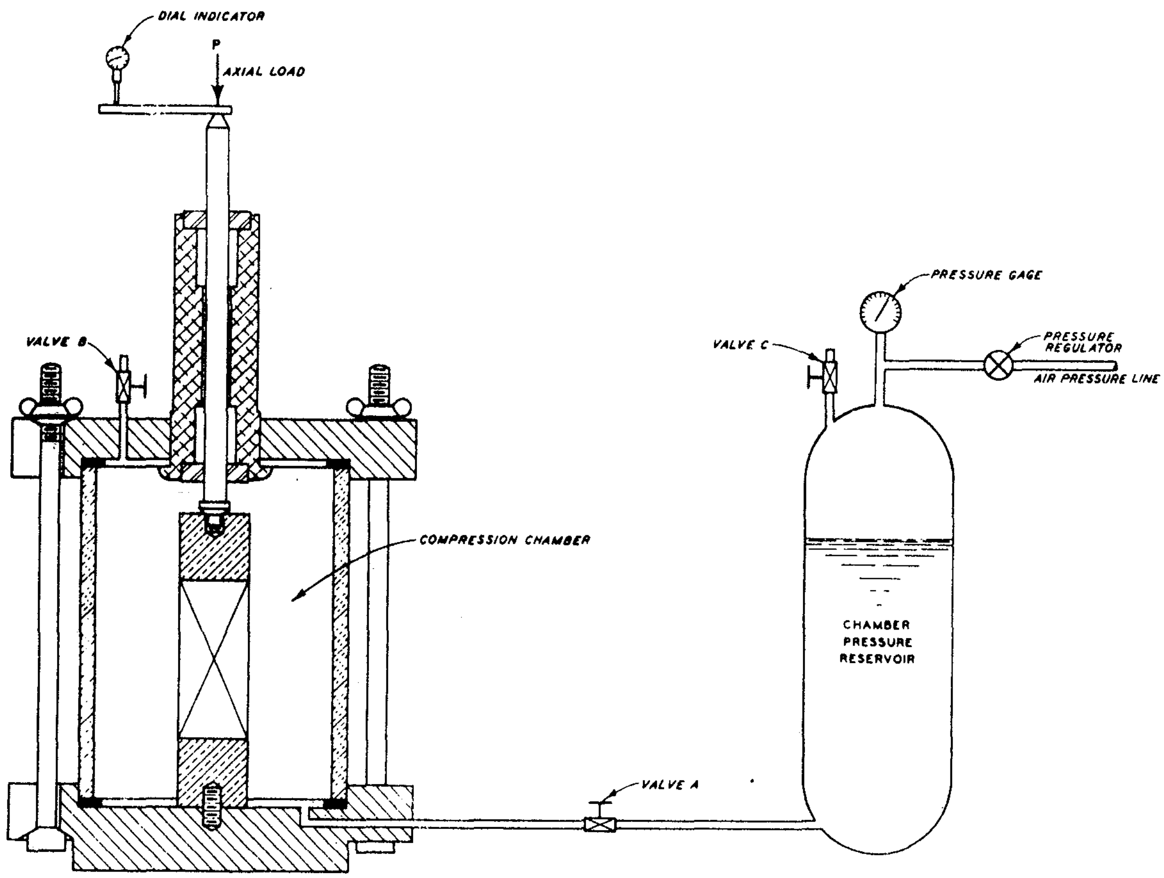


Figure 5-38 Schematic diagram of triaxial compression apparatus for Q test in the field



(4) Assemble the triaxial chamber and place it in position in the loading device. Connect the tube from the pressure reservoir to the base of the triaxial chamber. With valve C (see Figure 5-38) on the pressure reservoir closed and valves A and B open, increase the pressure inside the reservoir and allow the pressure fluid to fill the triaxial chamber. Allow a few drops of the pressure fluid to escape through the vent valve (valve B) to insure complete filling of the chamber with fluid. Close valve A and the vent valve.

(5) With valves A and C closed, adjust the pressure regulator to preset the desired chamber pressure. The range of chamber pressures for the three specimens will depend on the loadings expected in the field. The maximum confining pressure should be at least equal to the maximum normal load expected in the field in order that the shear strength data need not be extrapolated for use in design analysis. Record the chamber pressure on data sheets (Figure 11-12 and Figure 11-13). Now open valve A and apply the preset pressure to the chamber. Application of the chamber pressure will force the piston upward into contact with the ram of the loading device. This upward force is equal to the chamber pressure acting on the cross-sectional area of the piston minus the weight of the piston minus piston friction.

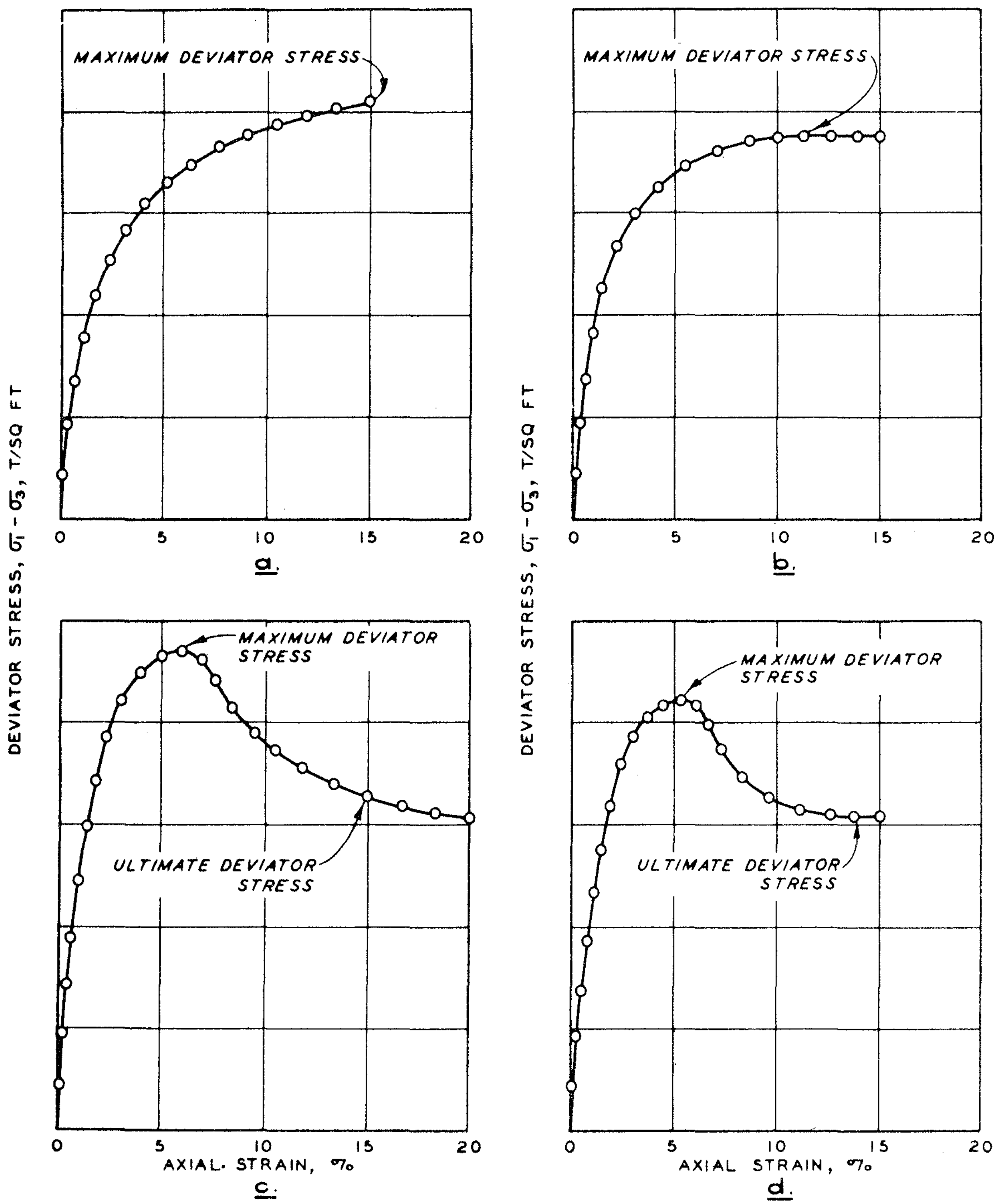
(6) Start the test with the piston approximately 0.1" above the specimen cap. This allows compensation for the effects of piston friction, exclusive of that which may later develop because of lateral force. Set the load indicator to zero when the piston meets the specimen cap. In this manner the upward thrust of the chamber pressure on the piston is also eliminated from further consideration. A slight movement of the load indicator indicates contact of the piston with the specimen cap. Set the strain

indicator and record on the data sheet (Figure 11-13) the initial dial reading at contact. Axially strain the specimen at a rate of about 1 percent/minute (for plastic materials) and about 0.3 percent/minute (for brittle materials that achieve maximum deviator stress at about 3 to 6% strain); at these rates the elapsed time to reach maximum deviator stress would be about 15 to 20 minutes.

(7) Observe and record the resulting load at every 0.3% strain for about the first 3% and, thereafter, at every 1%, or for large strains, at every 2% strain; sufficient readings should be taken to completely define the shape of the stress-strain curve so frequent readings may be necessary as failure is approached. Continue the test until an axial strain of 15% has been reached, as shown in Figure 5-39a, Figure 5-39b, and Figure 5-39d; however, when the deviator stress decreases after attaining a maximum value and is continuing to decrease at 15% strain (Figure 5-39c), the test shall be continued to 20% strain.

(8) For brittle soils (i.e., those in which maximum deviator stress is reached at 6% axial strain or less), tests should be performed at rates of strain sufficient to produce times to failure as set forth in paragraph 5.7.5.1(6) above; however, when the maximum deviator stress has been clearly defined, the rate may be increased such that the remainder of the test is completed in the same length of time as that taken to reach maximum deviator stress. However, for each group of tests about 20% of the samples should be tested at the rates set forth in paragraphs 5.7.5.1(6) and 5.7.5.1(7) above.

Figure 5-39 Examples of stress-strain curves



(9) Upon completion of axial loading, release the chamber pressure by shutting off the air supply with the regulator and opening valve C. Open valve B and draw the pressure fluid back into the pressure reservoir by applying a low vacuum at valve C. Dismantle the triaxial chamber. Make a sketch of the specimen, showing the mode of failure.

(10) Remove the membrane from the specimen. For 1.4" diameter specimens, carefully blot any excess moisture from the surface of the specimen and determine the water content of the whole specimen.

For 2.8” diameter or larger specimens, it is permissible to use a representative portion of the specimen for the water content determination. It is essential that the final water content is determined accurately, and a preferably different technician should verify weighings.

(11) Repeat the test on the two remaining specimens at different chamber pressures, though using the same rate of strain.

5.7.5.2. Computation

The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Figure 11-12) the initial water content, volume of solids, initial void ratio, initial degree of saturation, and initial dry density, using the formulas given in 3.4.2.

(2) Compute and record on the data sheet (Figure 11-13) the axial strain, the corrected area, and the deviator stress at each increment of strain, using the following formulas:

Equation 5-20: $$\varepsilon = \frac{\Delta H}{H_o}$$

Equation 5-21: $$A_{corr} = \frac{A_o}{1-\varepsilon}$$

Equation 5-22: $$\sigma = 0.93\frac{P}{A_{corr}}$$

Where

- ε = axial strain
- A_{corr} = corrected area of strain, cm_
- σ = deviator stress, ksf
- ΔH = change in height of specimen during test, cm
- H_o = initial height of specimen, cm. (Where a significant decrease in specimen volume occurs upon application of the chamber pressure, as in partially saturated soils, the height of the specimen after application of the chamber pressure should be used rather than the initial height.)
- A_0 = initial area of specimen, cm^2
- P = net applied axial load, lb (the actual load applied to specimen after correction for piston friction and for the upward thrust of the fluid pressure in the triaxial chamber)

(3) Record the time to failure on the data sheet (Figure 11-13).

(4) The rubber membrane increases the apparent strength of the specimen. Investigations[135] with specimens 1.5” in diameter and membranes 0.008” thick, for instance, indicate the increase in deviator stress to be 0.6 psi at 15% axial strain. The correction, σ_r, to be made to the measured deviator stress for the effect of the rubber membrane is computed as follows:

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[135] Bishop and Henkel, op. cit., p. 167-171.

Equation 5-23: $\sigma_r = \dfrac{\pi D_o M\varepsilon(1-\varepsilon)}{A_o}$

Where

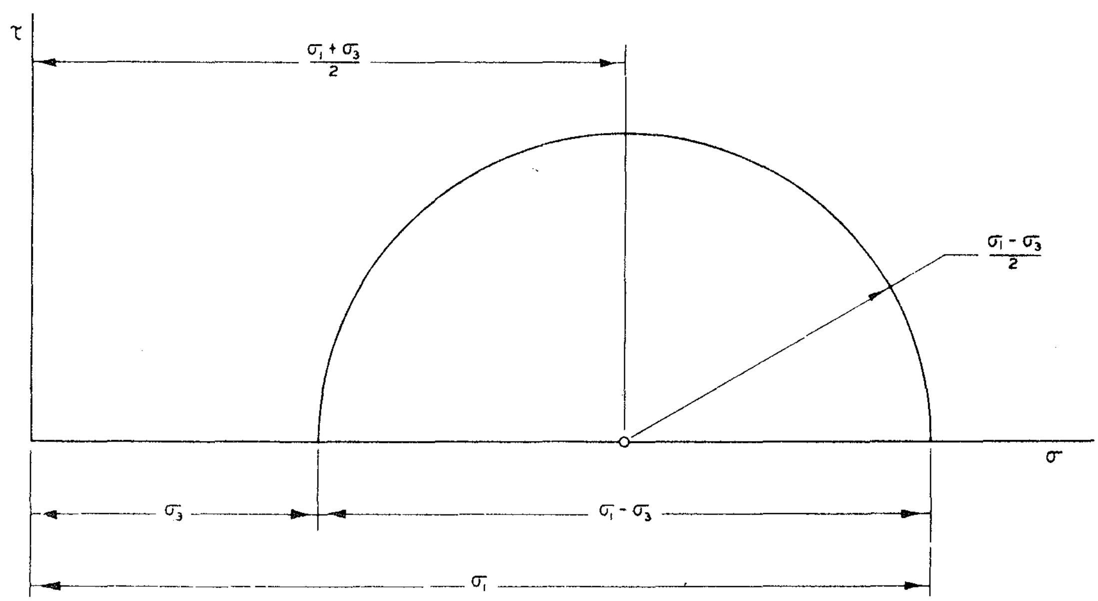
- D_o = initial diameter of specimen
- M = compression modulus of the rubber membrane
- ε = axial strain
- A_0 = initial cross-sectional area of the specimen

Without great error, the compression modulus may be assumed equal to that measured in extension. An apparatus for determining the extension modulus of rubber is described in another work.[135] In tests of very soft soils, the membrane effect may be significant, and in these tests, it is advisable to compute or estimate the correction and deduct it from the maximum deviator stress. For most soils tested using membranes of standard thickness, the correction is insignificant and can be ignored.

5.7.5.3. Presentation of Results

The results of the Q test shall be recorded on the report form shown as Figure 11-14. Enter pertinent information regarding the condition of the specimen or method of preparing the specimen under "Remarks." Plot the deviator stress versus the axial strain for each of the specimens as shown in Figure 5-39. The peak or maximum deviator stress represents "failure" of the specimen; when the deviator stress increases continuously during the test, the deviator stress at 15% axial strain shall be considered the maximum deviator stress. When the deviator stress decreases after reaching a maximum, the minimum deviator stress attained before 15% axial strain shall be considered the ultimate deviator stress, as shown in Figure 5-39c and Figure 5-39d.

Figure 5-40 Construction of Mohr's circle of stress

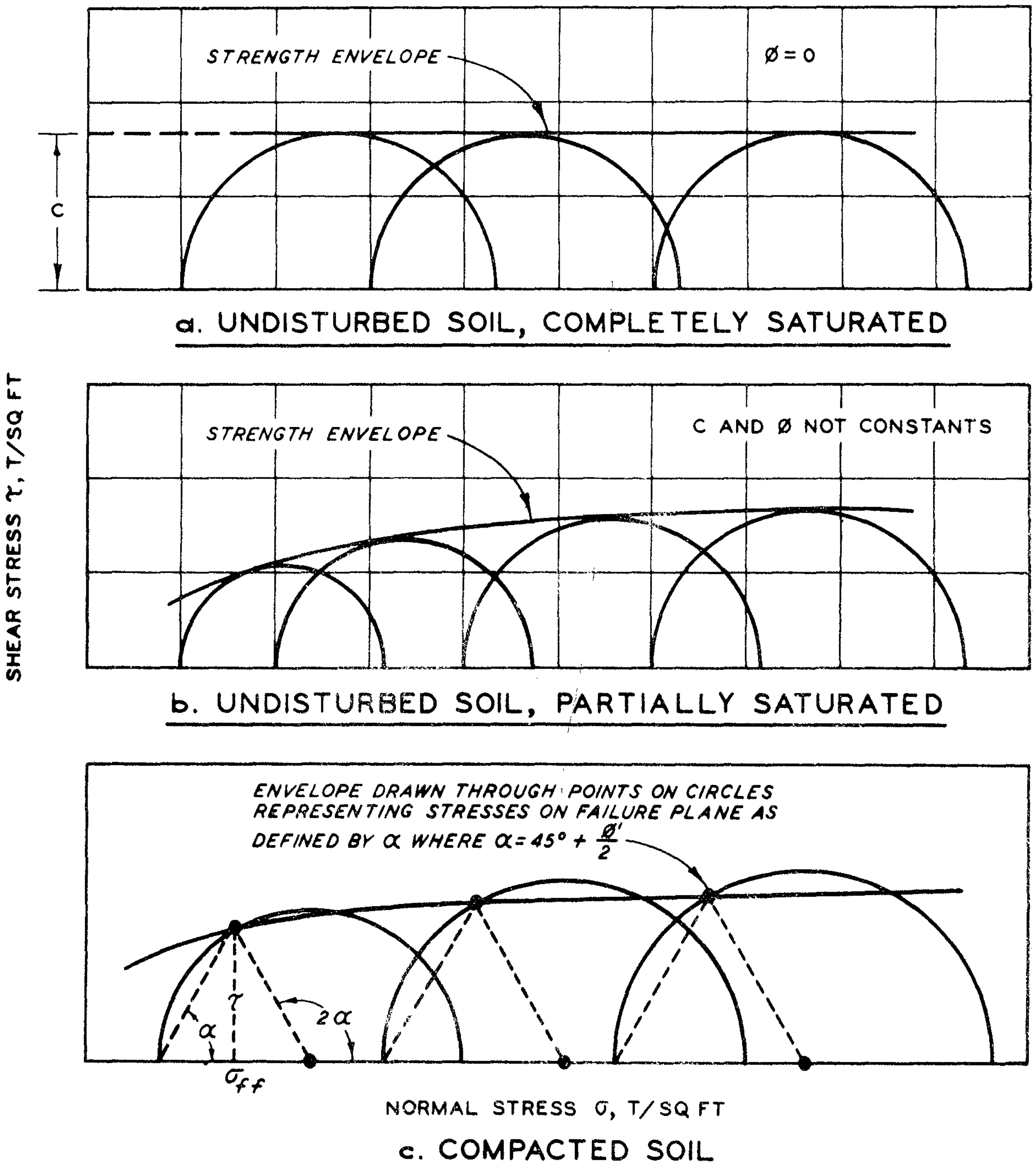


Construct Mohr stress circles on an arithmetic plot with shear stresses as ordinates and normal stresses as abscissas. As shown in Figure 5-40, the applied principal stresses, σ_1 and σ_3, are plotted on the circles are drawn with radii of one-half the maximum deviator stress $(\sigma_1 - \sigma_3)/2$ and with their centres at values equal to one-half the sums of the major and minor principal stresses $(\sigma_1 + \sigma_3)/2$. Plot a Mohr circle, or a sufficient segment thereof, for each specimen in the graph in the upper right corner of report form. A sketch of each specimen after failure should be shown above the Mohr circles (Figure 11-14). The following procedures should be followed in drawing strength envelopes:

(1) Undisturbed specimens. For undisturbed specimens, the strength envelope should be drawn tangent to the Mohr circles. Q tests of saturated soils usually indicate a strength envelope that is parallel to the abscissa as shown in Figure 5-41a, so the angle of internal friction is usually equal to zero. Strength envelopes indicated by Q tests on partially saturated soils are usually curved as shown in Figure 5-41b, particularly for the lower normal stresses. When the curvature is pronounced, the shear strength parameters ϕ and c are not constants.

(2) Compacted specimens. For compacted specimens, the strength envelope should be drawn through points on the Mohr circles representing stresses on the failure plane as shown in Figure 5-41c.

Figure 5-41 Examples of strength envelopes for Q tests



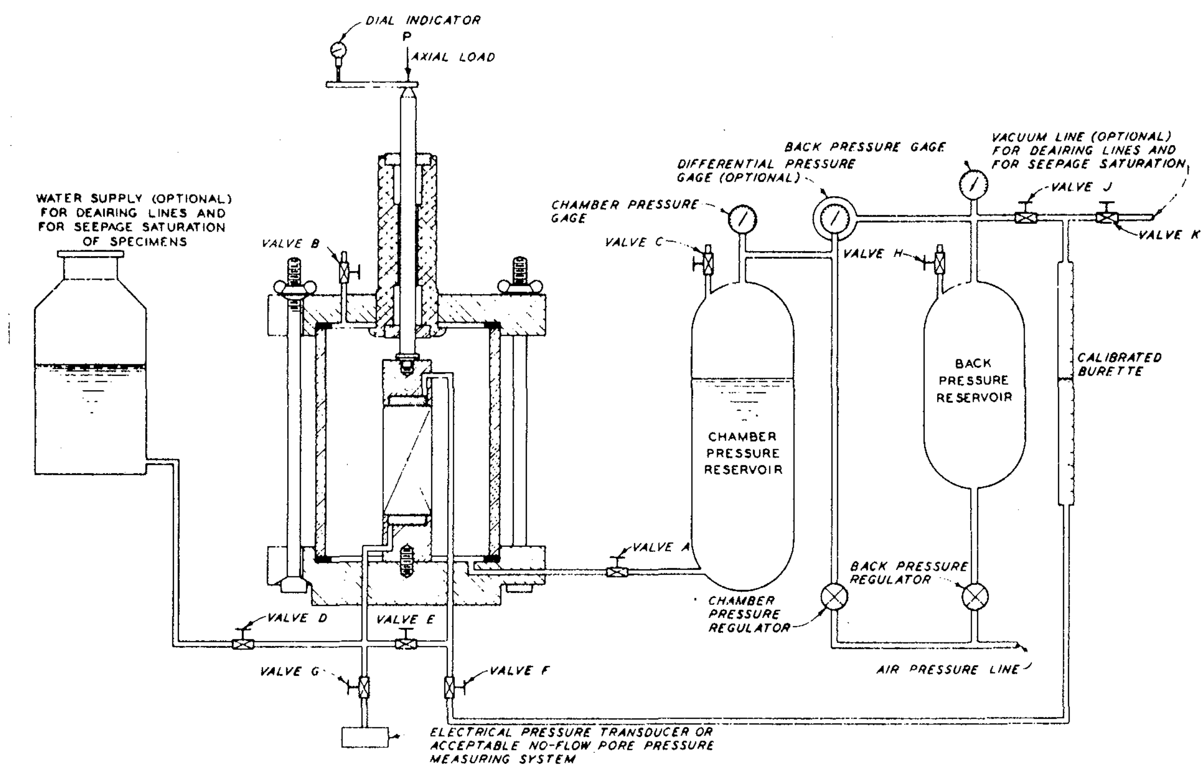
5.7.6. Q Test with Backpressure Saturation

In cases where a foundation soil exists that is partially saturated during exploration but which will become completely saturated without significant volume change either before or during construction, it is necessary to saturate Q test specimens, using back pressure, before they are sheared. Such field conditions may occur when, due to heavy rains or other reasons, the water table is raised above the level that existed during initial sampling: Construction of cofferdams, river diversions, and closure sections, or percolation of rain waters can also create conditions that increase foundation water contents after exploration but before embankment construction and subsequent consolidation of the foundation.

For the Q test with backpressure saturation, the apparatus should be set up similar to that shown in Figure 5-42. Filter strips should not be used and as little volume change as possible should be permitted during the test. After completing the steps outlined for specimen preparation and (1) through (4) of the Q test procedure (note that the procedures for attaching the membrane to the cap and base and for deairing the drainage lines are similar to those used in the R test), apply 3-psi chamber pressure to the specimen with all drainage valves closed. Allow a minimum of 30 minutes for stabilization of the specimen pore water pressure, measure ΔH, and begin backpressure procedures. After verification of saturation, and remeasurement of ΔH, close all drainage lines leading to the back pressure and pore water measurement apparatus.

Holding the maximum applied backpressure constant, increase the chamber pressure until the difference between the chamber pressure and the backpressure equals the desired effective confining pressure. Then proceed as outlined in paragraphs (6) through (11) of the standard Q test procedure.

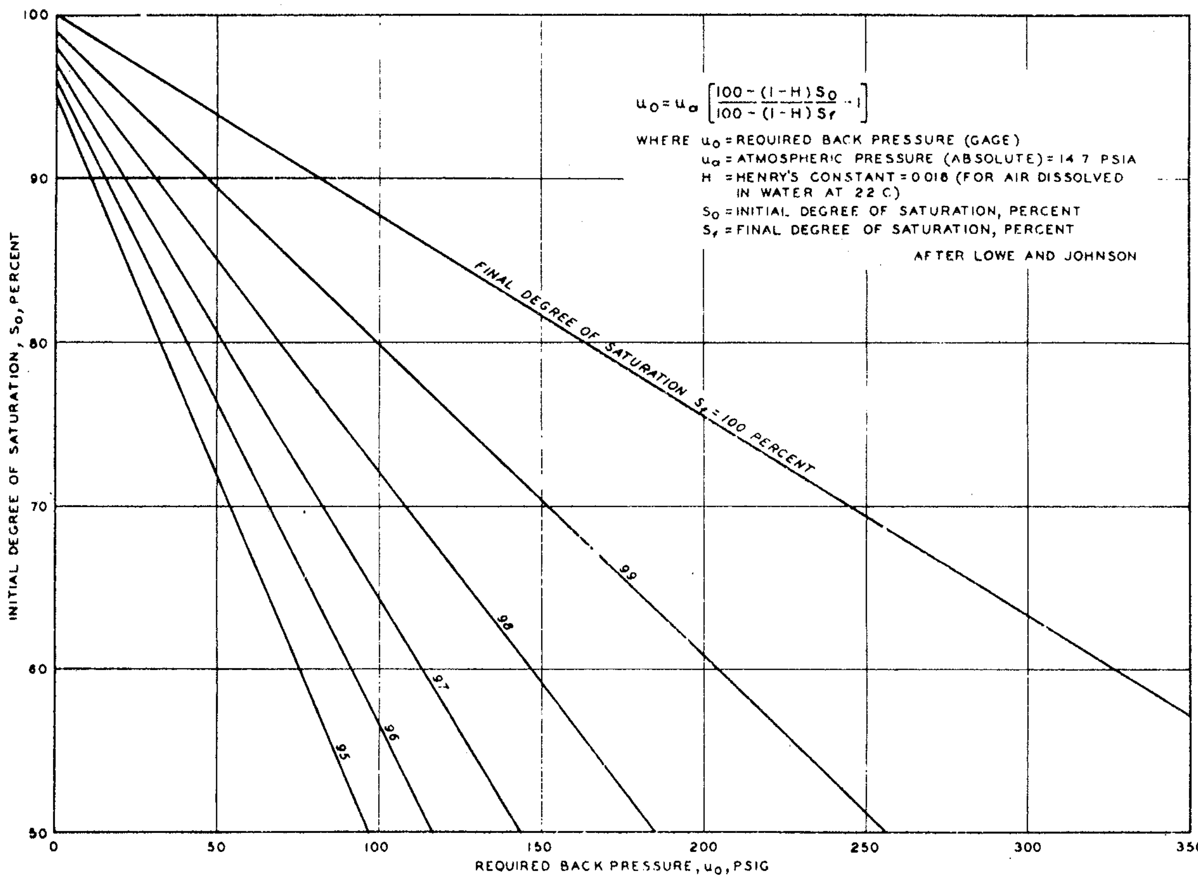
Figure 5-42 Schematic diagram of typical triaxial compression apparatus of R and S tests



5.7.7. R Test

All specimens must be completely saturated before application of the deviator stress in the R test. A degree of saturation over 98% can be considered to represent a condition of essentially complete saturation; if pore water pressures are to be measured during shear, however, the specimens must be 100% saturated. Computations of the degree of saturation based on changes of volume and water content are often imprecise, so complete saturation of a specimen should be assumed only when an increase of the chamber fluid pressure will cause an immediate and equal increase of pressure in the pore water of the specimen. In general, it is preferable to saturate the soil after the specimens have been prepared, encased in membranes, and placed within the compression chamber, using backpressure. A backpressure is an artificial increase of the pore water pressure that will increase the degree of saturation of a specimen by forcing pockets of air into solution in the pore water. The backpressure is applied to the pore water simultaneously with an equal increase of the chamber pressure so that the effective stress acting on the soil skeleton is not changed. In other words, the pressure differential across the membrane remains constant during the backpressure saturation phase. Thus, when the backpressure is increased sufficiently slowly to avoid an excessive pressure differential within the specimen itself, the degree of saturation will be increased while the volume of the specimen is maintained essentially constant. Figure 5-43 gives the backpressure theoretically required to produce a desired increase in saturation if there is no change in specimen volume. It is important to note that the relation shown in Figure 5-43 is based on an assumption that the water entering the specimen contains no dissolved air.

Figure 5-43 Backpressure required to attain various degrees of saturation



5.7.7.1. Apparatus

In addition to the apparatus described in the apparatus for the Q procedure, the following equipment are necessary for R tests utilizing back pressure for saturation:

(1) Air reservoir and regulator for controlling the backpressure, similar to those used to control the chamber pressure.

(2) Bourdon gage attached to the backpressure reservoir to measure the applied backpressure. As relatively large backpressures and chamber pressures are sometimes required, it is essential that these

two pressures be measured accurately to insure that the precise difference between them is known. A differential pressure gage[136] will permit this difference to be measured directly.

(3) Calibrated burette or standpipe capable of measuring volume changes to within 0.1 cm^3 for 1.4" diameter specimens, 0.5 cm^3 for 2.8" diameter specimens, and 1 cm^3 for 6" diameter specimens. This burette is connected in the backpressure line leading to the top of the specimen to measure the volume of water added to the specimen during saturation and volume changes of the specimen during consolidation. If the water added to the specimen becomes saturated with air, a higher backpressure will be required than that given in Figure 5-43. Therefore, precautions should be taken to minimize aeration of the saturation water by reducing the area of the air-water interface or by separating the air and water with a rolling rubber diaphragm.[137] A relatively long (over 6-foot) length of thick-walled, small-bore tubing between the burette and the specimen will also reduce the amount of air entering the specimen. Adequate safety precautions should be taken against breakage of the burette under high pressures.

(4) Electrical pressure transducer or no-flow indicator with which the pressure of the pore water at the bottom of the specimen can be measured without allowing a significant flow of water from the specimen. This is an extremely difficult measurement to make since even a minute flow of water will reduce the pressure in the pore water; yet, the measuring device must be sensitive enough to detect small changes in pressure. Electrical pressure transducers, while relatively expensive, offer almost complete protection against flow, are simple to operate, and lend themselves to the automatic recording of test data. Several types of manually balanced pressure-measuring systems employing a no-flow indicator are being used successfully, though a full discussion of their relative merits[138] and shortcomings is not possible here.

5.7.7.2. Procedure

The procedure for the R test utilizing backpressure for saturation shall consist of the following steps:

(1) Proceed as outlined in paragraphs (1) through (4) of the Q procedure, with the exception that specimen bases and caps with porous inserts and drainage connections should be used and backpressure equipment should be included as shown in Figure 5-42. Saturated strips of filter paper (such as Whatman's No. 54) placed beneath the membrane and extending from the base along three-fourths of the specimen length will reduce the time required for saturation and consolidation. These strips must neither overlap, forming a continuous circumferential coverage of the specimen, nor form a continuous path between the base and the cap. Place saturated filter paper disks having the same diameter as that of the specimen between the specimen and the base and cap; these disks will also facilitate removal of the specimen after the test. The drainage lines and the porous inserts should be completely saturated with deaired water. The drainage lines should be as short as possible and made

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[136] John Lowe, III, and Thaddeus C. Johnson, "Use of back pressure to increase degree of saturation of triaxial test specimens," *ASCE Research Conference on Shear Strength of Cohesive Soils*, University of Colorado Boulder, CO, June 1960).

[137] H. B. Seed, J. K. Mitchell, and C. K. Ghan, "The strength of compacted cohesive soils," *ASCE Research Conference on Shear Strength of Cohesive Soils*, University of Colorado (Boulder, Colo., June 1960).

[138] Bishop and Henkel, op. cit., pp. 52-63, 206-207.

A. Andersen, L. Bjerrum, E. DiBiagio, and B. Kjaernsli, Triaxial Equipment Developed at the Norwegian Geotechnical Institute, Publication No. 21, Norwegian Geotechnical Institute (Oslo, 1957).

A. Casagrande and R, C, Hirschfeld, First Progress Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays, Soil Mechanics Series No. 61, Harvard University (Cambridge, Mass., May 1960).

of thick-walled, small-bore tubing to insure minimum elastic changes in volume due to changes in pressure. Valves in the drainage lines (valves E, F, and G in Figure 5-42) should preferably be of a type that will cause no discernible change of internal volume when operated (such as the Teflon-packed ball valve made by the Whitey Research Tool Co.). While mounting the specimen in the compression chamber, care should be exercised to avoid entrapping any air beneath the membrane or between the specimen and the base and cap.

(2) Estimate the magnitude of the required backpressure by reference to Figure 5-43 or other theoretical relations. Specimens should be completely saturated before any appreciable consolidation is permitted, for ease and uniformity of saturation as well as to allow volume changes during consolidation to be measured with the burette; therefore, the difference between the chamber pressure and the back pressure should not exceed 5 psi during the saturation phase. To insure that a specimen is not prestressed during the saturation phase, the backpressure must be applied in small increments, with adequate time between increments to permit equalization of pore water pressure throughout the specimen.

(3) With all valves closed, adjust the pressure regulators to a chamber pressure of about 7 psi and a backpressure of about 2 psi. Record these pressures on the data sheet (Figure 11-15). Now open valve A to apply the preset pressure to the chamber fluid and simultaneously open valve F to apply the backpressure through the specimen cap. Immediately open valve G, read, and record the pore pressure at the specimen base. When the measured pore pressure becomes essentially constant, close valves F and G[139] and record the burette reading.

(4) Using the technique described in step (3), increase the chamber pressure and the backpressure in increments, maintaining the backpressure at about 5 psi less than the chamber pressure. The size of each increment might be 5, 10, or even 20 psi, depending on the compressibility of the soil specimen and the magnitude of the desired consolidation pressure. Open valve G and measure the pore pressure at the base immediately upon application of each increment of back pressure and observe the pore pressure until it becomes essentially constant. The time required for stabilization of the pore pressure may range from a few minutes to several hours depending on the permeability of the soil. Continue adding increments of chamber pressure and back pressure until, under any increment, the pore pressure reading equals the applied back pressure immediately upon opening valve G.

(5) Verify the completeness of saturation by closing valve F and increasing the chamber pressure by about 5 psi. The specimen shall not be considered completely saturated unless the increase in pore pressure immediately equals the increase in chamber pressure.

(6) When the specimen is completely saturated, hold the maximum applied backpressure constant and increase the chamber pressure until the difference between the chamber pressure and the backpressure equals the desired consolidation pressure. Open valve F and permit the specimen to consolidate (or swell) under the consolidation pressure. Valve E may be opened to allow drainage from both ends of the specimen. At increasing intervals of elapsed time (0.1, 0.2, 0.5, 1, 2, 4, 8, 15, and 30 minutes, 1, 2, 4, and 8 hours, etc.), observe and record (Figure 11-16) the burette readings and, if practicable, the dial indicator readings (it may be necessary to force the piston down into contact with the specimen cap for each reading). Plot the burette readings (and dial indicator readings, if taken) versus the logarithm of elapsed time, as shown in Figure 7-50. Allow consolidation to continue until a marked reduction in slope of the curve shows that 100% primary consolidation has been achieved.

[139] If an electrical pressure transducer is used to measure the pore pressure, valve G may be safely left open during the entire saturation procedure.

(7) Close valve G, unless pore pressure measurements are to be made during shear, and valves E and F, and proceed according to paragraphs 5.7.5.1(6) through 5.7.5.1(10), except use a rate of strain for the R test of about 0.5 percent/minute (for plastic materials) and about 0.3 percent/minute or less for brittle materials that achieve a maximum deviator stress at about 3 to 6% strain; the strain rate used should result in a time to maximum deviator stress of approximately 30 minutes. Relatively pervious soils may be sheared in 15 minutes. These rates of strain do not permit equalization of induced pore pressure throughout the specimen and are too high to allow satisfactory pore pressure measurements to be made at the specimen ends during shear.[140] Therefore, these rates of strain are applicable only to R tests in which no pore pressure measurements are made during shear. Where pore pressure measurements are made at the ends of the specimens as in R tests, the time to reach maximum deviator stress should generally be at least 120 minutes; considerably longer time may be required for materials of low permeability. For brittle soils (i.e., those in which the maximum deviator stress is reached at 6% axial strain or less), after the maximum deviator stress has been clearly defined, the rate of strain may be increased so that the remainder of the test is completed in the same length of time as that taken to reach maximum deviator stress. However, for each group of tests in a given test program, at least 20% of the samples should be tested to final axial strain at rates of strain outlined in the first sentence of this paragraph.

5.7.7.3. Computation

The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Figure 11-12) the initial water content, volume of solids, initial void ratio, initial degree of saturation, and initial dry density, using the formulas presented earlier.

(2) Compute the cross-sectional area of the specimen after completion of consolidation according to the following formula:

Equation 5-24: $$A_c = \frac{H_o - 2\Delta H_o}{H_o}$$

or if the specimen is or has been completely saturated during the test, use the more accurate formula

Equation 5-25: $$A_c = \frac{V_o - V_a - \Delta V_w}{H_o - \Delta H_o}$$

where

∞ A_c = area of the specimen after consolidation, cm_

∞ V_o = initial volume of specimen, cm_

∞ V_a = initial volume of air in specimen, cm_

∞ $V_o – V_s – V_w$ = initial volume of specimen minus volume of solids minus initial volume of water

∞ _V_w = change in water volume in the specimen during the saturation and consolidation phases of the test, cm_. This value may be computed from the change in weight of the specimen before and after the test or from the burette readings from the start of saturation on to the end of consolidation

∞ H_o = initial height of specimen, cm

[140] Bishop and Henkel, op. cit., pp. 192-204.

∞ ΔH_o = change in height of specimen during consolidation, cm

(3) Using the computed dimensions of the specimen after consolidation and assuming that the water content after consolidation is the same as the final water content, compute the void ratio and degree of saturation using formulas presented earlier.

(4) Compute and record on Figure 11-13 the axial strain, the corrected area, and the deviator stress at each increment of strain, using Equation 5-20, Equation 5-21 and Equation 5-22.

(5) Record the time to failure on the data sheet Figure 11-13.

(6) Correct the maximum deviator stress, if necessary, for the effect of membrane restraint (see paragraph 5b(4)).

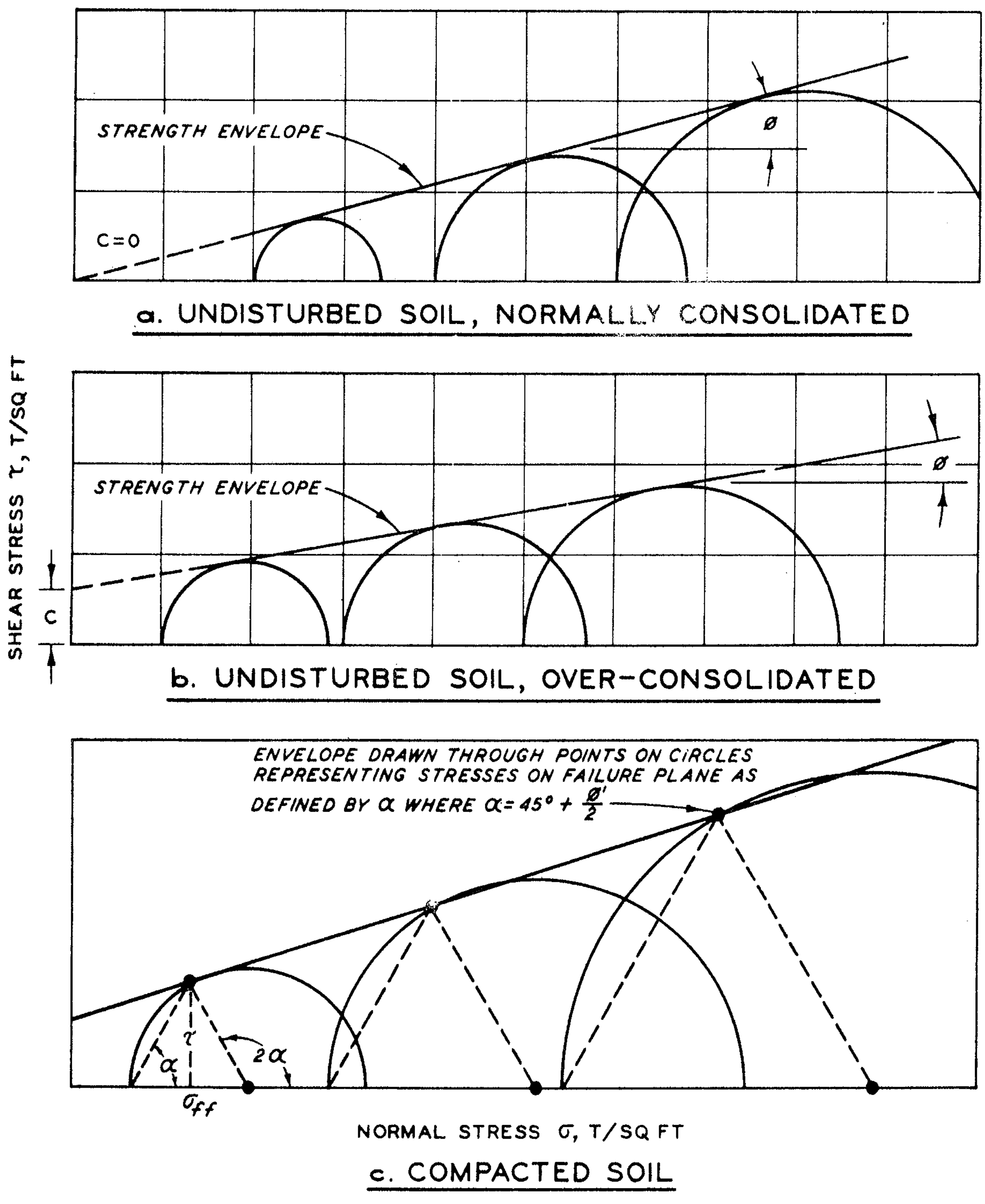
5.7.7.4. Presentation of results

The results of the R test shall be presented on the report form shown in Figure 11-14. A sketch of each specimen after failure should be shown above the Mohr circles. If pore pressure measurements were made during shear, plot the induced pore pressure versus axial strain for each specimen below the stress-strain curves. The procedures below should be followed in drawing strength envelopes:

(1) Undisturbed specimens. For undisturbed specimens, strength envelopes should be drawn tangent to the Mohr circles as shown in Figure 5-44a and Figure 5-44b.

(2) Compacted specimens. For compacted specimens, strength envelopes should be drawn through points on the Mohr circles representing stresses on the failure plane as shown in Figure 5-44c.

Figure 5-44 Examples of strength envelopes for R tests



5.7.8. S Test

The S test using triaxial equipment, as a rule, shall be performed only with relatively pervious soils. The consolidation of triaxial specimens of relatively impervious soils proceeds so slowly that the time required to complete an S triaxial test inhibits its use in routine laboratory work. Therefore, S tests of fine-grained impervious materials should normally be performed with direct shear equipment (see 5.5). However, if scheduling permits, it may be desirable to perform companion S triaxial tests of impervious soils to compare the results with those from S direct shear tests. If the soil to be tested is relatively impervious and contains gravel that would preclude the use of direct shear equipment, consideration should be given to using triaxial equipment with pore pressure measurements in order to obtain the drained shear strength parameters within a reasonable length of time. All specimens must be completely saturated before application of the deviator stress in the S test.

5.7.8.1. Apparatus

The apparatus used to perform the R test, as illustrated in Figure 5-42, will usually be satisfactory for the S test, though the equipment for saturation by back pressure will not be necessary for relatively pervious soils. In general, controlled-strain testing should be used for relatively pervious soils, and controlled-stress testing should be used for relatively impervious soils.

5.7.8.2. Procedure

The procedure for the S test shall consist of the following steps:

(1) For soils requiring saturation by backpressure, proceed as outlined for the R test. For pervious soils which can be effectively saturated by seepage, that is, by water percolating through the specimen under a small hydraulic head, omit the back pressure equipment described in paragraph 6a and proceed as follows:

   a. Proceed as outlined in paragraphs (1) through (4) of the Q test, with the exception that specimen bases and caps with porous inserts and drainage connections should be used and the apparatus should include a water supply container and a calibrated burette with a vacuum connection as shown in Figure 5-42. For specimens of cohesionless soil, the porous inserts and drainage lines (including the burette) should be dry, and a low vacuum (less than 5 psi) should be maintained at both the specimen base and cap (with valves D and G closed) to support the specimen while assembling and filling the triaxial chamber.

   b. Keeping valves A and C closed, adjust the pressure regulator for a chamber pressure of about 5 psi and then open valve A to apply this pressure to the chamber.

   c. With valves E and G closed, maintain a low vacuum through the burette to the specimen cap. Then open valve D and elevate the water supply container so that a hydrostatic head of 1 to 2 ft is applied to the base of the specimen.

   d. When the saturation water rises into the burette, disconnect the vacuum from the burette. Permit seepage under the small head to continue until the rate of flow into the burette is constant, and then close valve D.

(2) With valves E and F open (see Figure 5-42), lower the piston into contact with the specimen cap and increase the axial load at a relatively slow rate so that a fully drained condition exists at failure with controlled-strain loading or after each increment of load with controlled stress application. As for the direct shear test, considerable experience and judgment are generally required in determining the proper rate of axial load application (see 5.5). Theoretical formulas are also available[141] for estimating the time required for failure in S tests. Special precautions may be necessary for tests requiring an axial loading duration in excess of a few hours to insure that the chamber pressure (as well as the back pressure, if used) is maintained constant, that temperature fluctuations are minimized, and that evaporation or aeration of the water in the burette is reduced as much as

[141] Bishop and Henkel, op. cit., pp. 124-127, 204-206.

possible. Placing about 1 cm^3 of oil or dyed kerosene over the water surface in the burette will minimize evaporation.

(3) Record the dial indicator and burette readings at increasing intervals of elapsed time under each increment of load. For relatively impervious soils, plot either or both of these readings versus the logarithm of elapsed time, as shown in Figure 7-50, to establish when primary consolidation has been essentially completed for each increment of load. Record the final dial indicator and burette readings for each axial load increment on a form similar to Figure 11-17 prior to applying the next increment. With controlled strain loading, periodically observe and record (Figure 11-17) the resulting load and the dial indicator and burette readings; sufficient readings should be taken to completely define the shape of the stress-strain curve. Continue the test until an axial strain of 15% has been reached; however, when the deviator stress decreases after attaining a maximum value and is continuing to decrease at 15% strain, the test shall be continued to 20% strain (see Figure 5-39).

(4) Upon completion of axial loading, close valves E and F and proceed as outlined in paragraphs (8) through (10) of the Q procedure, except measure the specimen diameter after the compression chamber has been dismantled. While considerable difficulty may be encountered in measuring the diameter of the specimen after the test, such measurements will permit the most reliable computations of the specimen properties at failure.

5.7.8.3. Computation

The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Figure 11-12) the initial water content, volume of solids, initial void ratio, initial degree of saturation, and initial dry density using the formulas previously presented.

(2) Compute the cross-sectional area of the specimen, A_C, after completion of consolidation using the same formulas as with the Q procedure.

(3) Using the dimensions of the specimen after consolidation and the changes in volume as measured with the burette, compute the void ratio and degree of saturation after consolidation using the formulas previously presented.

(4) Compute and record on the data sheet Figure 11-17 the axial strain, the corrected area, and the deviator stress corresponding to the final readings under each increment of load for controlled-stress loading or for convenient intervals of strain for controlled-strain loading using the following equations:

Equation 5-26: $$\varepsilon = \frac{\Delta H}{H_c}$$

Equation 5-27: $$A'_{corr} = \frac{A_c}{1 - C_\varepsilon}$$

Equation 5-28: $$\sigma = 0.93\frac{P}{A'_{corr}}$$

where

- ε = axial strain
- A'_{corr} = Area of specimen corrected for strain and volume change, cm^2
- σ = deviator stress, ksf

- C = correction for volume change during shear = A_f/A_e
- A_f = area of specimen after test based on measurements = $\pi D_f^2/4$
- A_e = area of specimen at end of test computed on basis of constant volume = $A_c/(1-\varepsilon_e)$
- ε_e = axial strain at end of test = $(H_c - H_f)/H_c$
- P = net applied axial load, lbs. (see paragraph 5b(2))

(5) Record the time to failure on the data sheet Figure 11-17).

(6) Correct the maximum deviator stress, if necessary, for the effect of membrane restraint.

5.7.8.4. Presentation of Results

The results of the S test shall be presented on the report form shown in Figure 11-15. If volume changes of the specimens during shear were measured, plot the volumetric strain versus axial strain for each specimen below the stress-strain curves.

5.7.9. Possible Errors

Following are possible errors that would cause inaccurate determinations of strength and stress deformation characteristics:

5.7.9.1. Apparatus

(1) Leakage of chamber fluid into specimen. Such leakage might occur through or around the ends of the membrane or through the drainage connections and it would decrease the effective stress in a specimen during undrained shear, Very little leakage is needed to cause a very large change in effective stress, and the longer the period of undrained shear, the greater the amount of leakage. (Leakage will not influence the effective stress during periods of specimen drainage, but it will introduce errors in volume change measurements.)

(2) Leakage of pore water out of specimen. This leakage might occur through fittings or valves and it would increase the effective stress in a specimen during undrained shear.

(3) Permeability of porous inserts too low.

(4) Restraint caused by membrane and filter paper strips.

(5) Piston friction.

5.7.9.2. Preparation of Specimen

(1) Specimen disturbed while trimming. Disturbance of the natural soil structure does not always result in strength measurements which are too low, that is, on the safe side; disturbed specimens will consolidate more under the effective consolidation pressure in R or S tests and the measured strengths will be too high.

(2) Specimen disturbed while enclosing with membrane. The techniques of placing the membrane around the specimen illustrated in Figure 5-36 and Figure 5-37 may not be satisfactory for sensitive undisturbed soils since the specimen would tend to be flexed while binding the membrane to the unsupported cap. Alternatively, the specimen can be set upon an inverted cap clamped to a ring stand and the membrane placed over the specimen and bound to the cap; then the specimen and cap can be inverted onto the base and the lower end of the membrane secured.

(3) Specimen dimensions not measured precisely. Dial gages or micrometers are helpful in obtaining precise measurements. When the specimen diameter is measured after being enclosed by the

membrane, twice the thickness of the membrane must be subtracted from the measurement. The cross-section area of large specimens may be determined most satisfactorily from circumference measurements.

5.7.9.3. Q Test

(1) Changes in specimen dimensions upon application of chamber pressure. Partially saturated specimens will compress under the chamber pressure so the change in height, ΔH_o, due to the application of chamber height should be recorded. When this change in height is significant, the area of specimen before shear, A_C, should be computed according to the formula:

Equation 5-29: $$A_c = A_o \frac{H_o - 2\Delta H_o}{H_o}$$

(2) Rate of strain too fast.

(3) Water content determination after test not representative. Friction between the soil and the cap and base restrains the radial deformation at the ends of the specimen and this end restraint induces a nonuniform pore pressure distribution that, in turn, causes pore water migration within the specimen. For relatively impervious soils, a significant migration of pore water could occur only in a test of long duration (such as S and some R tests); however, for more pervious soils, an appreciable redistribution of water content can occur within the short duration of a Q test. Therefore, it may be desirable to determine the water content of the end sections (about 1/6 of the height at each end) separately from the middle portion. Correlations of strength with water content should be based on the water content of the middle portion, though the dry weight of the entire specimen is needed to compute the initial soil properties.[142]

5.7.9.4. R Test

(1) Back pressure increments too large in relation to effective consolidation pressure.

(2) Backpressure increments applied too rapidly.

(3) Chamber and backpressures not precisely maintained during consolidation phase. Variations in either or both of these pressures (often much larger than the difference between them) can result in overconsolidation of the specimen.

(4) Specimen not completely consolidated before shearing.

(5) Rate of strain too fast.

(6) Excessive variations in temperature during shear. An increase in temperature will decrease the effective stress in a specimen during undrained shear. This danger, obviously, increases with the duration of the test.

(7) Specimen absorbed water from porous inserts at end of test. As in a consolidation test or a direct shear test, the specimen will absorb water from the porous inserts and drainage lines at the end of the R or S test no matter how rapidly the apparatus is disassembled and the specimen removed. To obtain an accurate water content determination at the end of the test, the specimen should be allowed to swell completely under a small (2 or 3 psi) chamber pressure and the increase in volume measured

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[142] A. Casagrande and S. J. Poulos, Fourth Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays, Soil Mechanics Series No. 74, Harvard University (Cambridge, MA, October 1964).

by means of the burette. This volume change can then be used to correct the water content measured after the test.

5.7.9.5. S Test

(1) Rate of strain or rate of loading too fast.

(2) Inaccurate volume change measurements during shear. Where volume changes are measured using a burette, inaccuracies may result from incomplete saturation of the specimen, leakage, evaporation, or temperature fluctuations.

5.8. Unconfined Compression Test

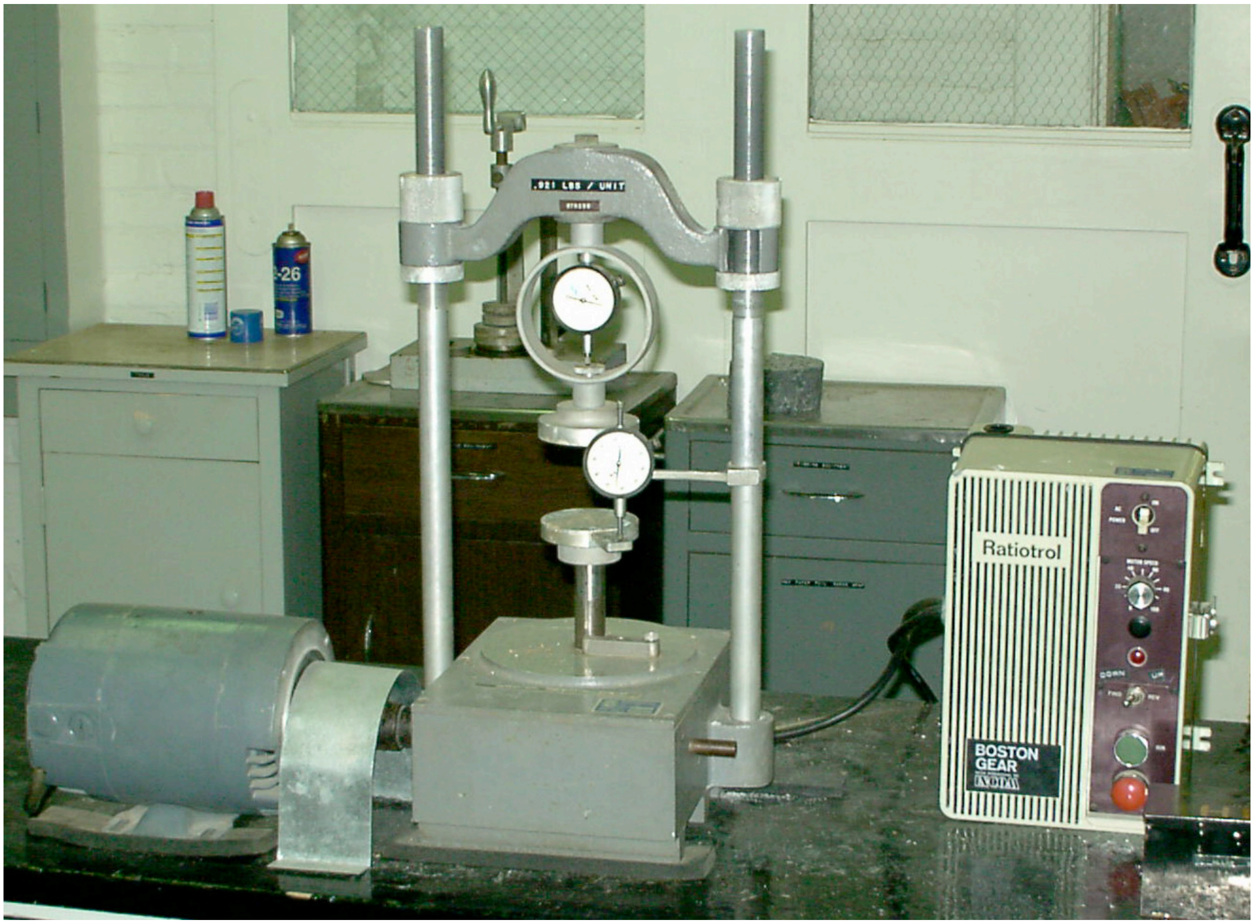
5.8.1. Apparatus

The apparatus consists of the following:

a. Equipment for Preparing Specimen. A trimming frame as described in 5.7.3, or a trimming cylinder with beveled cutting edges may be used for trimming specimens. The equipment should include wire saws and knives of various sizes and types for use with the trimming frame. A motorized soil lathe may be used advantageously under certain circumstances. A miter box or cradle is required to trim the specimen to a fixed length and to ensure that the ends of the specimen are parallel with each other and perpendicular to the vertical axis of the specimen.

b. Loading Device. A number of commercially available controlled-strain or controlled-stress types of loading devices are suitable for applying the axial loads in the unconfined compression test. In general, controlled- strain type loading devices are preferable, and the procedures described herein are based on the use of this type of equipment. If available, an automatic stress-strain recorder may be used to measure and record applied axial loads and displacements. A typical loading device is shown in Figure 5-45. Any equipment used should be calibrated so that the loads actually applied to the soil specimen can be determined. The required sensitivity of stress-measuring equipment for both controlled-stress and controlled-strain testing will vary with the strength characteristics of the soil. For relatively weak soils (compressive strengths less than 2 ksf), the unit load should be measurable to within 0.02 ksf. For soils with compressive strengths of 2 ksf or greater, the loads should be measurable to the nearest 0.1 ksf.

Figure 5-45 Typical Unconfined Compression Test Apparatus



c. Measuring equipment, such as dial indicators and calipers, suitable for measuring the dimensions and axial deformation of a specimen to the nearest 0.001".

d. Timing device, either a watch or clock with second hand.

e. Balances, sensitive to 0.1 g.

f. Other. Apparatus necessary to determine water content and specific gravity.

5.8.2. Preparation of Specimens

5.8.2.1. Specimen Size

Unconfined compression, specimens shall have a minimum diameter of 1.0" (preferably 1.4"), and the largest particle in any test specimen will be no greater than one-sixth the specimen diameter. The height-to-diameter ratio shall be not less than 2. Commonly used diameters of unconfined compression specimens are 1.4 and 2.8". Specimens of 1.4" diameter are generally used for testing cohesive soils that contain a negligible amount of gravel.

5.8.2.2. Undisturbed Specimens

Generally, undisturbed specimens are prepared from undisturbed tube or chunk samples of a larger size than the test specimen. Core or thin-wall tube samples of relatively small diameter may be tested without further trimming except for squaring the ends, if the condition of the soil requires this procedure.

Specimens must be handled carefully to prevent remolding, changes in cross section, or loss of moisture. To minimize disturbance caused by skin friction between samples and metal sampling tubes, the tubes should be cut into short lengths before ejecting the samples. Sample ejection should be accomplished with a smooth continuous and rapid motion in the same direction that the sample entered the tube. All specimens shall be prepared in a humid room to prevent evaporation of moisture. The specimen shall be prepared as follows:

(1) From the undisturbed sample cut a section somewhat larger in length and diameter than the desired specimen size. It is generally desirable to prepare duplicate specimens for unconfined compression testing, and selection of material for testing should be made with this in mind.

(2) Carefully trim the specimen to the required diameter using a trimming frame and various trimming tools (see Figure 5-34). Remove any small shells or pebbles encountered during the trimming operations. Carefully fill voids on the surface of the specimen with remolded soil obtained from the trimmings. Cut the specimen to the required length, using a miter box (see Figure 5-35). Where the presence of pebbles or crumbling results in excessive irregularity at the ends, cap the specimens with a minimum thickness of plaster of Paris, hydrostone, or other support material. Care must be taken to insure that the ends of the specimen are parallel with each other and perpendicular to the vertical axis of the specimen.

(3) From the soil trimmings obtain 200 g of material for specific gravity and water content determinations.

(4) Weigh the specimen to an accuracy of ± 0.01 g for 1.4" diameter specimens and ± 0.1 g for 2.8" diameter specimens. If specimens are to be capped, they should be weighed before capping.

(5) Measure the height of the specimen with calipers or a scale and the diameter with calipers or circumference measuring devices. If the specimen is cut to a fixed length in a miter box, the length of the miter box can be taken as the height of specimen for routine tests, and additional height measurements are not usually necessary. It is always advisable to measure the diameter of the specimen after trimming, even though specimens are cut to a nominal diameter in a trimming frame. Make all measurements to the nearest ± 0.01". Determine the average initial diameter, D_o, of the specimen using the diameters measured at the top, D_t, centre, D_c and bottom, D_b, of the specimen, as follows:

Equation 5-30: $$D_o = \frac{D_t + 2D_c + D_b}{4}$$

(6) If the specimen is not tested immediately after preparation, precautions must be taken to prevent drying and consequent development of capillary stresses. When drying before or during the test is anticipated, the specimen may be covered with a thin coating of grease such as petrolatum. This coating cannot be used if the specimen is to be used in a subsequent remolded test.

5.8.2.3. Remolded Specimens

Remolded specimens usually are prepared in conjunction with tests made on undisturbed specimens after the latter has been tested to failure. The remolded specimens are tested to determine the effects of remolding on the shear strength of the soil. The remolded specimen should have the same water content as the undisturbed specimen in order to permit a comparison of the results of the tests on the two specimens. The remolded specimen shall be prepared as follows:

(1) Place the failed undisturbed specimen in a rubber membrane and knead it thoroughly with the fingers to assure complete remolding of the specimen. Take reasonable care to avoid entrapping air in the specimen and to obtain a uniform density.

(2) Remove the soil from the membrane and compact it in a cylindrical mould with inside dimensions identical with those of the undisturbed specimen. The compaction effort is not critical since the water contents of soils subjected to remolded tests are always considerably wetter than optimum. Care must be taken, however, to insure uniform density throughout the specimen. A thin coat of petrolatum on the inside of the molding cylinder will assist in the removal of the specimen after compaction.

(3) Carefully remove the specimen from the mould, preferably by means of a close fitting piston, and plane off the top of the specimen. The specimen is then ready for testing.

(4) Follow the steps outlined in paragraphs (4) and (5) for undisturbed specimens.

5.8.3. Procedure

. The procedure shall consist of the following steps:

a. Record all identifying information for the sample such as project, boring number, visual classification, and other pertinent data on the data sheet (see Figure 11-18 which is a suggested form). The data sheet is also used for recording test observations described below.

b. Place the specimen in the loading device so that it is centered on the bottom platen; then adjust the loading device carefully so that the loading ram or upper platen barely is in contact with the specimen. If a proving ring is used for determining the axial load, contact of the platen and specimen is indicated by a slight deflection of the proving ring dial. Attach a dial indicator, sensitive to 0.001", to the loading ram to measure vertical deformation of the specimen. Record the initial reading of the dial indicator on the data sheet (Figure 11-18). Test the specimen at an axial strain rate of about 1 percent/minute. For very stiff or brittle materials which exhibit small deformations at failure, it may be desirable to test the specimen at a slower rate of strain.. Observe and record the resulting load corresponding to increments of 0.3% strain for the first 3% of strain and in increments of 1 or 2% of strain thereafter. Stop the test when the axial load remains constant or when 20% axial strain has been produced.

c. Record the duration of the test, in minutes, to peak strength (time to failure), type of failure (shear or bulge), and a sketch of specimen after failure on the data sheet (Figure 11-19).

d. After the test, place the entire specimen or a representative portion thereof in a container of known weight and determine the water content of the specimen.

5.8.4. Computations

The computations consist of the following steps:

a. From the observed data, compute and record on the data sheet (Figure 11-18) the water content, volume of solids, void ratio, degree of saturation and dry density, using the formulas presented in 3.4.2.

b. Compute and record on the data sheet the axial strain, the corrected area, and the compressive stress, at each increment of strain by using the following formulas:

Equation 5-31: $\varepsilon = \frac{\Delta H}{H_o}$

Equation 5-32: $A_{corr} = \frac{A_o}{1-\varepsilon}$

Equation 5-33: $\sigma = 0.93\frac{P}{A_{corr}}$

Where

- ε = Axial strain
- A_{corr} = Corrected area of specimen, cm^2
- σ = Compressive Stress, ksf
- ΔH = change in height of specimen during test, cm
- H_o = initial height of specimen, cm
- A_o = initial area of specimen, cm^2
- P = applied load, lb.

5.8.5. Presentation of Results

The results of the unconfined compression test shall be recorded on the report form shown as Figure 11-19. Pertinent information regarding the condition of the specimen, method of preparing the specimen, or any unusual features of each specimen (such as slickensides, stratification, shells, pebbles, roots or brittleness) should be shown under "Remarks." The applied compressive stress shall be plotted versus the axial strain in Figure 11-19. The unconfined compressive strength q_u of the specimen shall be taken as the maximum or peak compressive stress. For tests continued to 20% strain without reduction of axial loading occurring, the unconfined compressive strength as a rule shall be taken as the compressive stress at 15% strain.

Where the unconfined compressive strength of a specimen is also obtained after remolding, the sensitivity ratio, S_t, shall also be calculated and reported. The sensitivity ratio is defined as follows:

Equation 5-34: $S_t = \frac{q_{u_{undisturbed}}}{q_{u_{remoulded}}}$

5.8.6. Possible Errors

Following are possible errors that would cause inaccurate determinations of unconfined compressive strength:

a. Test not appropriate to type of soil.

b. Specimen disturbed while trimming.

c. Loss of initial water content. A small change in water content can cause a larger change in the strength of clay, so it is essential that every care be taken to protect the specimen against evaporation while trimming and measuring, during the test, and when remolding a specimen to determine the sensitivity.

d. Rate of strain or rate of loading too fast.

5.8.7. Use of Other Types of Equipment for Undrained Shear Strength Determinations

Various other types of laboratory equipment, such as cone penetrometers and vane shear apparatus, may be used advantageously in the laboratory as a supplement to the basic unconfined compression test equipment for determining the undrained shear strength of cohesive soils. The use of these testing devices generally results in savings in cost and time. However, the devices should be used with caution until sufficient data and procedural details are established to assure their successful application. Use of such testing apparatus, as a rule, should be preceded by careful correlations with the results of tests with the basic unconfined compression test equipment on the same type of soil, and correlations developed for a given type of soil should not be used indiscriminately for all soils.

§ 6. Seepage and Drainage

This section covers surface erosion, and analysis of flow quantity and groundwater pressures associated with underseepage. Requirements are given for methods of drainage and pressure relief.

Control of soil erosion must be considered in all new construction projects. Seepage pressures are of primary importance in stability analysis and in foundation design and construction. Frequently, drawdown of groundwater is necessary for construction. In other situations, pressure relief must be incorporated in temporary and permanent structures. For erosion analysis, the surface water flow characteristics, soil type, and slope are needed. For analysis of major seepage problems, determine permeability and piezometric levels by field observations.

6.1. Introduction to Permeability

6.1.1. Theory of Permeability using Darcy's Law

The flow of water through a soil medium is assumed to follow Darcy's law:

Equation 6-1: $$q = k\frac{\Delta h}{L}A$$

Where

- q = discharge (volume/time)
- A = cross-sectional area
- Δh = height of water drop, length
- L= length of water flow
- Δh/L = the hydraulic gradient (dimensionless; use of this as the hydraulic gradient is based on Bernoulli's equation)
- k = coefficient of permeability, expressed in length per unit time

If we define the hydraulic gradient as

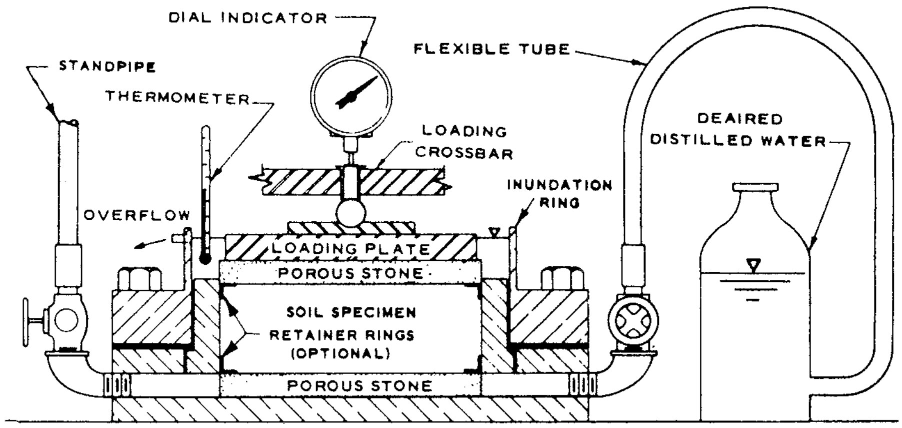
Equation 6-2: $$i = \frac{\Delta h}{L}$$

Equation 6-2 reduces to

Equation 6-3: $$q = kiA$$

The application of Darcy's law to a specimen of soil in the laboratory is illustrated in Figure 6-1.

Figure 6-1 Flow of water through soil



The coefficient of permeability, k, is defined as the rate of discharge of water at a temperature of 20º C under conditions of laminar flow through a unit cross-sectional area of a soil medium under a unit hydraulic gradient. The coefficient of permeability has the dimensions of velocity and is usually expressed in centimeters/second. The permeability of a soil depends primarily on the size and shape of the soil grains, the void ratio of the soil, the shape and arrangement of the voids, and the degree of saturation. Permeability computed based on Darcy's law is limited to the conditions of laminar flow and complete saturation of the voids.

In turbulent flow, the flow is no longer proportional to the first power of the hydraulic gradient. Under conditions of incomplete saturation, the flow is in a transient state and is time-dependent. The laboratory procedures presented herein for determining the coefficient of permeability are based on the Darcy conditions of flow. Unless otherwise required, the coefficient of permeability shall be determined for a condition of complete saturation of the specimen. Departure from the Darcy flow conditions to simulate natural conditions is sometimes necessary; however, the effects of turbulent flow and incomplete saturation on the permeability should be recognized and taken into consideration.

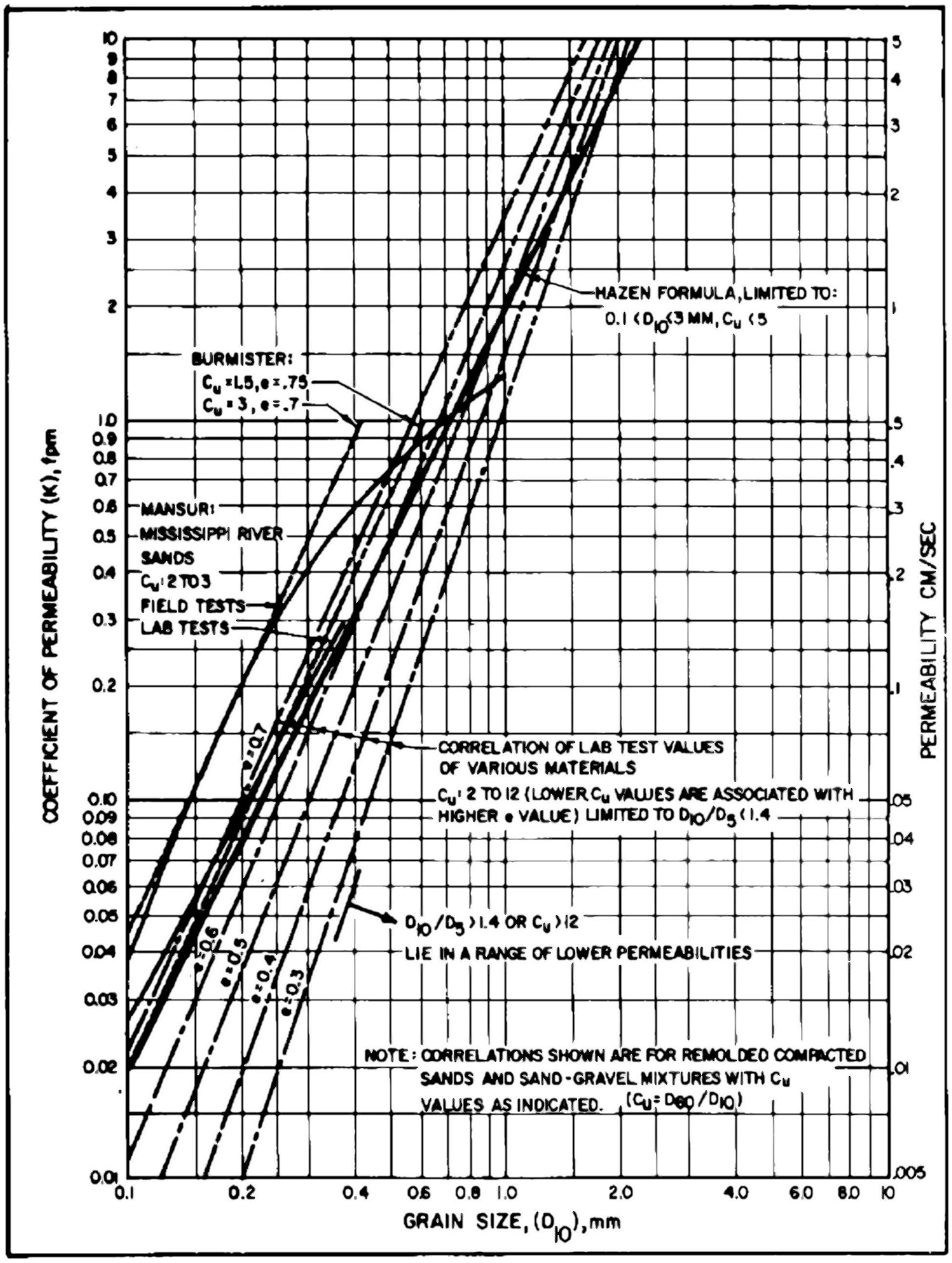
6.1.2. Application of Permeability Data

The permeability coefficient is used to compute the quantity and rate of water flow through soils in drainage and seepage analysis. Laboratory tests are appropriate for undisturbed samples of fine-grained materials and compacted materials in dams, filters, or drainage structures.

- Fine-Grained Soils. Permeability of fine-grained soils (undisturbed or compacted) generally is computed from consolidation test data or by direct measurement on consolidation or triaxial shear specimens. For soils with permeability less than 10^{-6} cm/sec, a sealant must be used between the specimen and the wall of the permeameter.
- Sand Drain Design. Sand drain design may require complete permeability data for soils to be stabilized, including determination of permeabilities in both vertical and horizontal direction.
- Field Permeability Tests. The secondary structure of *in situ* soils, stratification, and cracks has a great influence on the permeability. Results of laboratory tests should be interpreted with this in mind, and field permeability tests should be performed where warranted.

Permeability is the most variable of all the material properties commonly used in geotechnical analysis. A permeability spread of ten or more orders of magnitude has been reported for a number of different types of tests and materials. Coefficient of permeability is a property highly sensitive to sample disturbance, and shows a wide range of variation due to differences in structural characteristics.[143] Permeability of clean, coarse-grained samples is related to D_{10} size (Figure 6-2).

Figure 6-2 Permeability of Sand and Sand-Gravel Mixtures



6.1.3. Seepage Forces

The flow of water through soil exerts a force on the soil called a seepage force. The seepage pressure is this force per unit volume of soil and is equal to the hydraulic gradient times the unit weight of water.

[143] For typical values of permeability for a variety of soil types, see McCarthy, David, *Essentials of Soil Mechanics and Foundations*, 6th Edition. Upper Saddle River, NJ: Prentice-Hall, 2001.

Equation 6-4: $P_S = i\gamma_W$

where

- P_s = seepage pressure
- i = hydraulic gradient
- γ_w = unit weight of water

The seepage pressure acts in a direction at right angles to the equipotential lines (see Figure 6-3).

The seepage pressure is of great importance in analysis of the stability of excavations and slopes because it is responsible for the phenomenon known as boiling or piping.

6.1.3.1. Boiling

Boiling occurs when seepage pressures in an upward direction exceed the downward force of the soil. The condition can be expressed in terms of critical hydraulic gradient. A minimum factor of safety of 2 is usually required, i.e.,

Equation 6-5:

$$i_c = i_{critical} = \frac{\gamma_T - \gamma_W}{\gamma_W} = \frac{\gamma_b}{\gamma_W}$$

$$FS = \frac{i_c}{i} \geq 2$$

where

- i = actual hydraulic gradient
- γ_T = total unit weight of the soil
- γ_W = unit weight of water
- γ_b = buoyant unit weight of soil

6.1.3.2. Piping and Subsurface Erosion

Most piping failures are caused by subsurface erosion in or beneath dams. These failures can occur several months or even years after a dam is placed into operation.

In essence, water that comes out of the ground at the toe starts a process of erosion (if the exit gradient is high enough) that culminates in the formation of a tunnel-shaped passage (or "pipe") beneath the structure. When the passage finally works backward to meet the free water, a mixture of soil and water rushes through the passage, undermining the structure and flooding the channel below the dam. It has been shown that the danger of a piping failure due to subsurface erosion increases with decreasing grain size.

Similar subsurface erosion problems can occur in relieved dry-docks, where water is seeping from a free source to a drainage or filter blanket beneath the floor or behind the walls. If the filter falls or is defective and the hydraulic gradients are critical, serious concentrations of flow can result in large voids and eroded channels.

Potential passageways for the initiation of piping include: uniformly graded gravel deposits, conglomerate, open joints in bedrock, cracks caused by earthquakes or crustal movements, open joints in pipelines, hydraulic fracture, open voids in coarse boulder drains including French drains, abandoned wellpoint holes, gopher holes, cavities formed in levee foundations by rotting roots or buried wood, improper backfilling of pipelines, pipes without antiseepage collars, etc.

Failure by piping requires progressive movement of soil particles to a free exit surface. It can be controlled by adequately designed filters or relief blankets.[144]

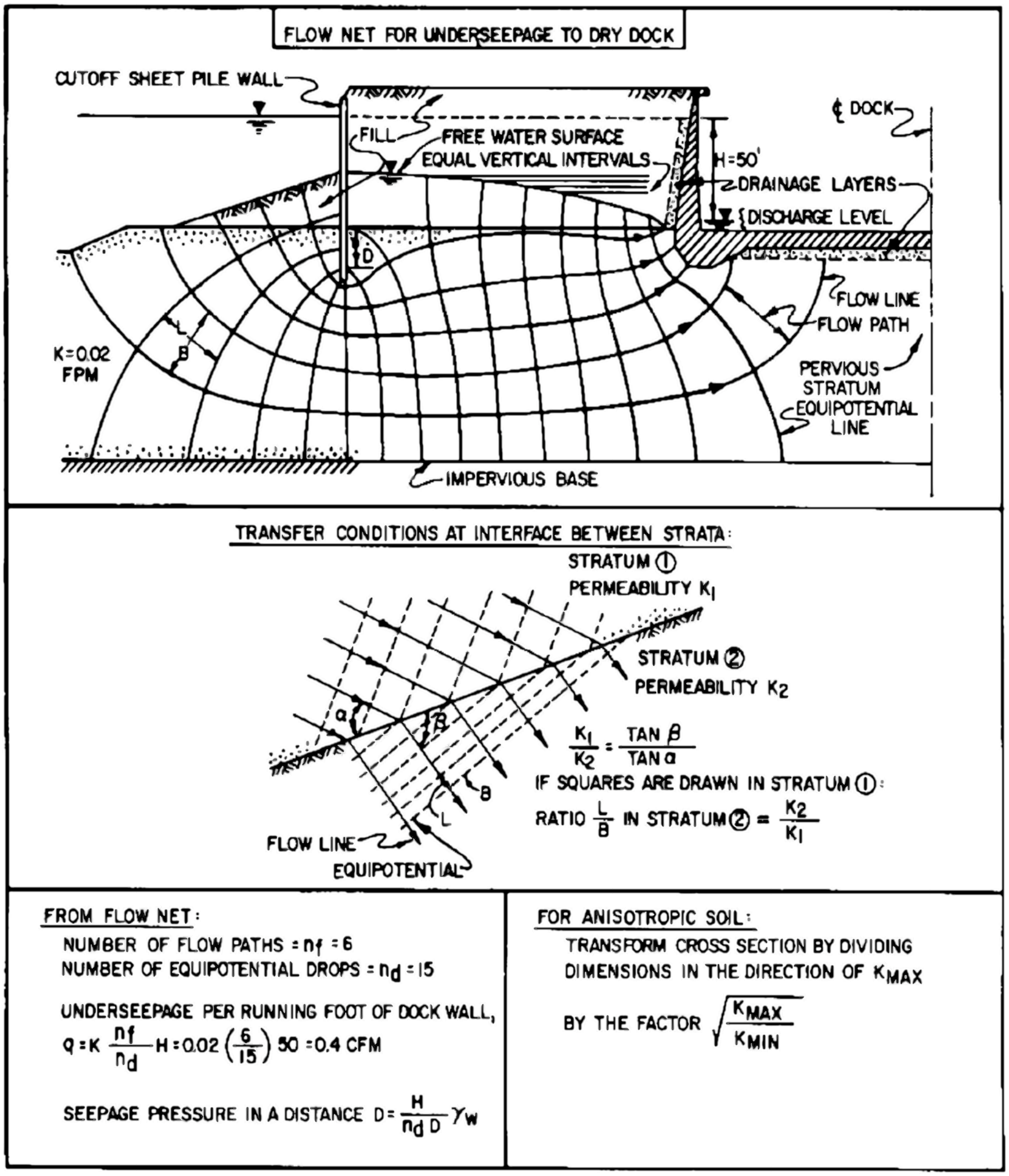
6.2. Seepage Analysis

6.2.1. Flow Net

Figure 6-3 shows an example of flow net construction. Use this procedure to estimate seepage quantity and distribution of pore water pressures in two-dimensional flow. Flow nets are applicable for the study of cut-off walls and wellpoints, or shallow drainage installations placed in a rectangular layout whose length in plan is several times its width. Flow nets can also be used to evaluate concentration of flow lines.

[144] Guidelines for preventing piping beneath dams may be found in Lee, E.W., Security from Under Seepage of Masonry Dams on Earth Foundations, Transactions, ASCE, Volume 100, Paper 1919, 1935.

Figure 6-3 Flow Net Construction and Seepage Analysis



6.2.1.1. Rules for flow net construction

1. When materials are isotropic with respect to permeability, the pattern of flow lines and equipotentials intersect at right angles. Draw a pattern in which square figures are formed between flow lines and equipotentials.
2. Usually it is expedient to start with an integer number of equipotential drops, dividing total head by a whole number, and drawing flow lines to conform to these equipotentials. In the general case, the outer flow path will form rectangular rather then square figures. The shape of these rectangles (ratio b/l) must be constant.
3. The upper boundary of a flow net that is at atmospheric pressure is a "free water surface". Integer equipotentials intersect the free water surface at points spaced at equal vertical intervals.

4. A discharge face through which seepage passes is an equipotential line if the discharge is submerged, or a free water surface if the discharge is not submerged. If it is a free water surface, the flow net figures adjoining the discharge face will not be squares.
5. In a stratified soil profile where ratio of permeability of layers exceeds 10, the flow in the more permeable layer controls. That is, the flow net may be drawn for more permeable layer assuming the less permeable layer to be impervious. The head on the interface thus obtained is imposed on the less pervious layer for construction of the flow net within it.
6. In a stratified soil profile where ratio of permeability of layers is less than 10, flow is deflected at the interface in accordance with the diagram shown above.
7. When materials are anisotropic with respect to permeability, the cross section may be transformed by changing scale as shown above and flow net drawn as for isotropic materials. In computing quantity of seepage, the differential head is not altered for the transformation.
8. Where only the quantity of seepage is to be determined, an approximate flow net suffices. If pore pressures are to be determined, the flow net must be accurate.
    a. Groundwater Pressures. For steady state flow, water pressures depend on the ratio of mean permeability of separate strata and the anistropy of layers. A carefully drawn flow net is necessary to determine piezometric levels within the flow field or position of the drawdown curve.
    b. Seepage Quantity. Total seepage computed from flow net depends primarily on differential head and mean permeability of the most pervious layer. The ratio of permeabilities of separate strata or their anisotropy has less influence. The ratio n_f/n_d in Figure 6-3 usually ranges from 1/2 to 2/3 and thus for estimating seepage quantity a roughly drawn flow net provides a reasonably accurate estimate of total flow. Uncertainties in the permeability values are much greater limitations on accuracy.

For special cases, the flow regime can be analyzed by the finite element method. Mathematical expressions for the flow are written for each of the elements, considering boundary conditions. A computer solves the resulting system of equations to obtain the flow pattern.

6.3. Seepage Control by Cut-off

Procedures for seepage control include cutoff walls for decreasing the seepage quantity and reducing the exit gradients, and drainage or relief structures that increase flow quantity but reduce seepage pressures or cause drawdown in critical areas. Table 6-1 shows various cut-off methods for seepage control.

Table 6-1 Cut-off Methods for Seepage Control

Method	Applicability	Characteristics and Requirements
Sheet pile cut off wall	Suited especially for stratified soils with high horizontal and low vertical permeability or pervious hydraulic fill materials. May be easily damaged by boulders or buried obstructions. Tongue and groove wood sheeting utilized for shallow excavation in soft to medium soils. Interlocking steel sheetpiling is utilized for deeper cutoff.	Steel sheeting must be carefully driven to maintain interlocks tight. Steel H-pile soldier beams may be used to minimize deviation of sheeting in driving. Some deviation of sheeting from plumb toward the side with least horizontal pressure should be expected. Seepage through interlocks is minimized where tensile force acts across interlocks. For straight wall sheeting an appreciable flow may pass through interlocks. Decrease interlock leakage by filling interlocks with sawdust, bentonite, cement grout, or similar material.
Compacted barrier of impervious soil	Formed by compacted backfill in a cutoff trench carried down to impervious material or as a core section in earth dams.	Layers or streaks of pervious material in the impervious zone must be avoided by careful selection and mixing of borrow materials, scarifying lifts, aided by sheepsfoot rolling. A drainage zone downstream of an impervious section of the embankment is necessary in most instances.

Method	Applicability	Characteristics and Requirements
Grouted or injected cutoff	Applicable where depth or character of foundation materials make sheetpile wall or cutoff trench impractical. Utilized extensively in major hydraulic structures. May be used as a supplement below cutoff sheeting or trenches.	A complete positive grouted cutoff is often difficult and costly to attain, requiring a pattern of holes staggered in rows with carefully planned injection sequence and pressure control. See DM-7.3, Chapter 2 for materials and methods.
Slurry trench method	Suited for construction of impervious cutoff trench below groundwater or for stabilizing trench excavation. Applicable whenever cutoff walls in earth are required. Is replacing sheetpile cutoff walls.	Vertical sided trench is excavated below groundwater as slurry with specific gravity generally between 1.2 and 1.8 is pumped back into the trench. Slurry may be formed by mixture of powdered bentonite with fine-grained material removed from the excavation. For a permanent cutoff trench, such as a foundation wall or other diaphragm wall, concrete is tremied to bottom of trench, displacing slurry upward. Alternatively, well graded backfill material is dropped through the slurry in the trench to form a dense mixture that is essentially an incompressible mixture; in working with coarser gravels (which may settle out), to obtain a more reliable key into rock, and a narrower trench, use a cement-bentonite mix.

Method	Applicability	Characteristics and Requirements
Impervious wall of mixed in-place piles.	Method may be suitable to form cofferdam wall where sheet pile cofferdam is expensive or cannot be driven to suitable depths, or has insufficient rigidity, or requires excessive bracing.	For a cofferdam surrounding an excavation, a line of overlapping mixed in-place piles are formed by a hollow shaft auger or mixing head rotated into the soil while cement grout is pumped through the shaft. Where piles cannot be advanced because of obstructions or boulders, supplementary grouting or injection may be necassary.
Freezing - ammonium brine or liquid nitrogen	All types of saturated soils and rock. Forms ice in voids to stop water. Ammonium brine is better for large applications of long duration. Liquid nitrogen is better for small applications of short duration where quick freezing is needed.	Gives temporary mechanical strength to soil. Installation costs are high and refrigeration plant is expensive. Some ground heave occurs.

6.3.1. Sheet Piling

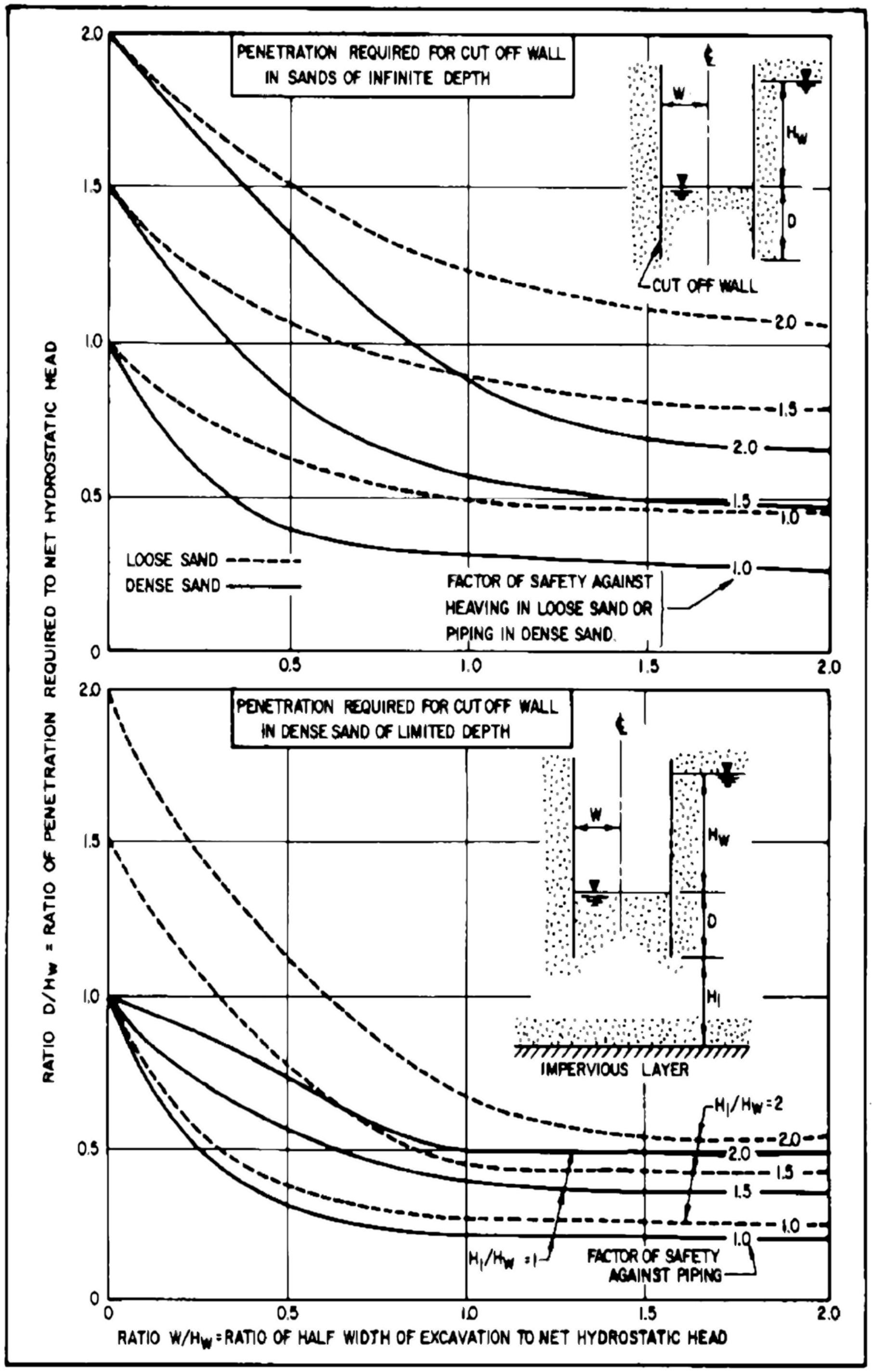
A driven line of interlocking steel sheeting may be utilized for a cut-off as a construction expedient or as a part of the completed structure.

- Applicability. The following considerations govern the use of sheet piling:

- Sheeting is particularly suitable in coarse-grained material with maximum sizes less than about 6 inches or in stratified subsoils with alternating fine grained and pervious layers where horizontal permeability greatly exceeds vertical.
- To be effective, sheeting must be carefully driven with interlocks intact. Boulders or buried obstructions are almost certain to damage sheeting and break interlock connections. Watertightness cannot be assumed if obstructions are present.
- Loss of head across a straight wall of intact sheeting depends on its watertightness relative to the permeability of the surrounding soil. In homogeneous fine-grained soil, head loss created by sheeting may be insignificant. In pervious sand and gravel, head loss may be substantial depending on the extent to which the flow path is lengthened by sheeting. In this case, the quantity of water passing through intact interlocks may be as much as 0.1 gpm per foot of wall length for each 10 feet differential in head across sheeting, unless special measures are taken to seal interlocks.

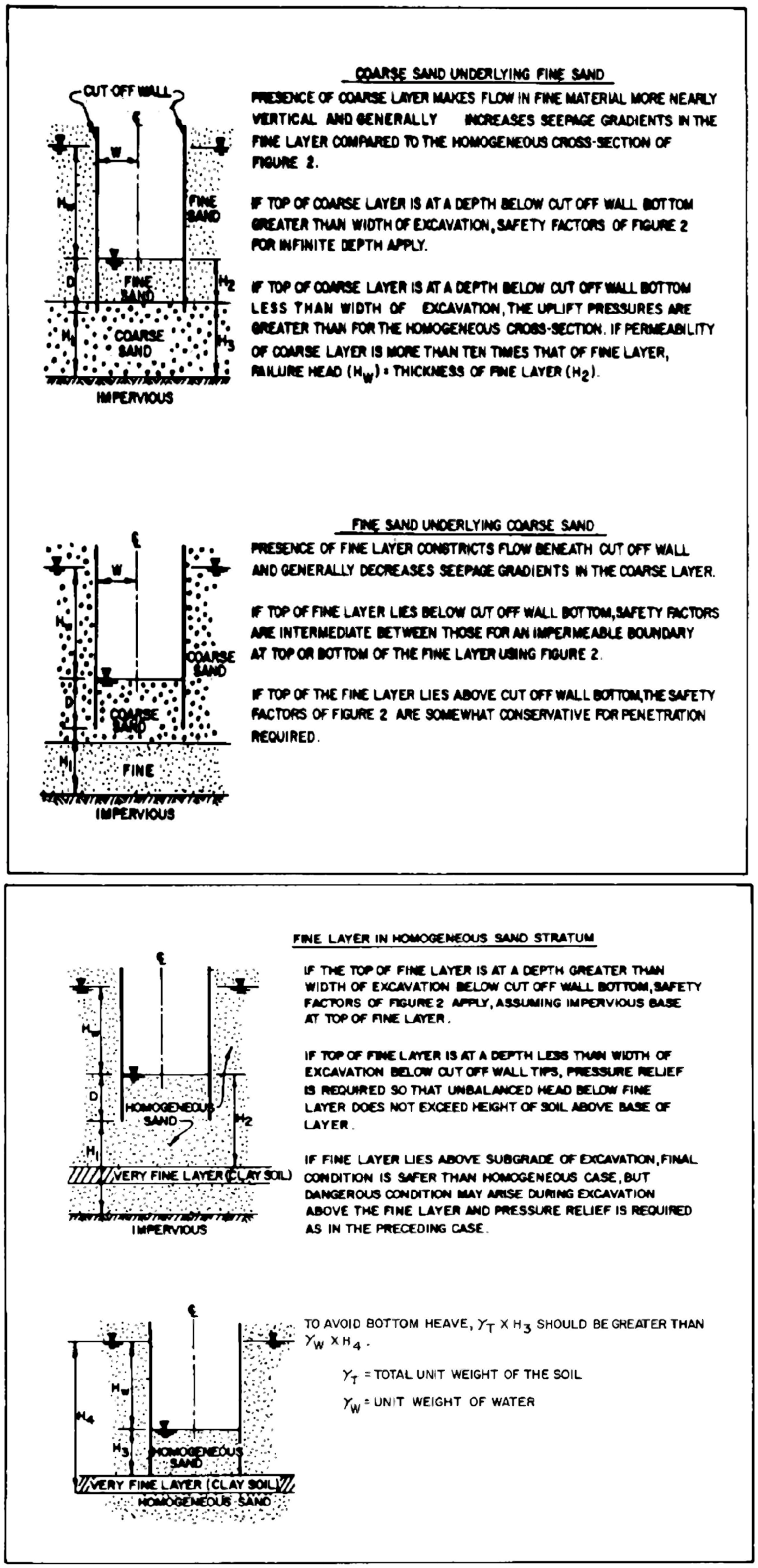
∞ Penetration Required. This paragraph and the next apply equally to all impervious walls listed in Table 6-1. Seepage beneath sheeting driven for partial cut-off may produce piping in dense sands or heave in loose sands. Heave occurs if the uplift force at the sheeting toe exceeds the submerged weight of the overlying soil column. To prevent piping or heave of an excavation carried below groundwater, sheeting must penetrate a sufficient depth below subgrade or supplementary drainage will be required at subgrade. See Figure 6-4 for sheeting penetration required for various safety factors against heave or piping in isotropic sands. For homogeneous but anisotropic sands, reduce the horizontal cross-section dimensions by the transformation factor of Figure 6-3 to obtain the equivalent cross section for isotropic conditions. See Figure 6-5 for sheeting penetration required in layered subsoils. For clean sand, exit gradients between 0.5 and 0.75 will cause unstable conditions for men and equipment operating on the subgrade. To avoid this, provide sheeting penetration for a safety factor of 1.5 to 2 against piping or heave.

Figure 6-4 Penetration of Cut-off Wall to Prevent Piping in Isotropic Sand[145]



145 Marsland, A., Model Experiments to Study the Influence of Seepage on the Stability of a Sheeted Excavation in Sand, Geotechnique, 1952-1953.

Figure 6-5 Penetration of Cut-off Wall Required to Prevent Piping in Stratified Sand



∞ Supplementary Measures. If it is uneconomical or impractical to provide required sheeting penetration, the seepage exit gradients may be reduced as follows:

- For homogeneous materials or soils whose permeability decreases with depth, place wellpoints, pumping wells, or sumps within the excavation. Wellpoints and pumping wells outside the excavation are as effective in some cases and do not interfere with bracing or excavation.
- For materials whose permeability increases with depth, ordinary relief wells with collector pipes at subgrade may suffice.
- A pervious berm placed against the sheeting, or a filter blanket at subgrade, will provide weight to balance uplift pressures. Material placed directly on the subgrade should meet filter criteria.
- An outside open water source may be blanketed with fines or bentonite dumped through water or placed as a slurry. See Table 6-2.

Evaluate the effectiveness of these measures by flow net analysis.

Table 6-2 Impermeable Reservoir Linings

Method	***Applicability and Procedures***
Buried Plastic Liner	Impervious liner formed of black coloured polyvinyl chloride plastic film. Where foundation is rough or rocky, place a layer 2 to 4 inches thick of fine-grained soil beneath liner. Seal liner sections by bonding with manufacturer's recommended solvent with 6" overlap at joints. Protect liner by 6" min. cover of fine-grained soil. On slopes, add a 6" layer of gravel and cobbles 3/4 to 3" size. Anchor liner in a trench at top of slope. Avoid direct contact with sunlight during construction before covering with fill and in completed installation. Usual thickness range of 20 to 45 mils (.020" to 045"). Items to be specified include Tensile Strength (ASTM D412), Elongation at Break (ASTM D412), Water Absorption (ASTM D471), Cold Bend (ASTM D2136), Brittleness Temperature (ASTM D746), Ozone Resistance (ASTM D1149), Heat Aging Tensile Strength and Elongation at Break (ASTM D412), Strength - Tear and Grab (ASTM D751).
Buried Synthetic Rubber Liner	Impervious liner formed by synthetic rubber, most often polyester reinforced. Preparation, sealing, protection, anchoring, sunlight, thickness, and ASTM standards are same as Buried Plastic Liner.
Bentonite Seal	Bentonite placed under water to seal leaks after reservoir filling. For placing under water, bentonite may be poured as a powder or mixed as slurry and placed into the reservoir utilizing methods recommended by the manufacturer. Use at least 0.8 pounds of bentonite for each square foot of area, with greater concentration at location of suspected leaks. For sealing silty or sandy soils, bentonite should have no more than 10% larger than 0.05 mm; for gravelly and rocky materials, bentonite can have as much as 40% larger than 0.05 mm. For sealing channels with flowing water or large leaks, use mixture of 1/3 each of sodium bentonite, calcium bentonite, and sawdust.
Earth Lining	Lining generally 2 to 4 feet thick of soils having low permeability. Used on bottom and sides of reservoir extending to slightly above operating water levels. Permeability of soil should be no greater than about $2x10^{-6}$ fpm for water supply linings and $2x10^{-7}$ fpm for pollution control facility linings.
Thin Compacted Soil Lining with Chemical	Dispersant is utilized to minimize thickness of earth lining required by decreasing permeability of the lining. Used where wave action is not liable to erode the lining. Dispersant, such as sodium tetraphosphate, is spread on a 6" lift of clayey silt or

Dispersant	clayey sand. Typical rate of application is 0.05 lbs/sf. Chemical and soil are mixed with a mechanical mixer and compacted by sheepsfoot roller. Using a suitable dispersant, the thickness of compacted linings may be limited to about 1 foot; the permeability of the compacted soil can be reduced to 1/10 of its original value.

6.3.2. Grouted Cut-off

Complete grouted cut-off is frequently difficult and costly to attain. Success of grouting requires careful evaluation of pervious strata for selection of appropriate grout mix and procedures. These techniques, in combination with other cut-off or drainage methods, are particularly useful as a construction expedient to control local seepage.

6.3.3. Impervious Soil Barriers

Backfilling of cut-off trenches with selected impervious material and placing impervious fills for embankment cores are routine procedures for earth dams.

- Compacted Impervious Fill. Properly constructed, these sections permit negligible seepage compared to the flow through foundations or abutments. Pervious layers or lenses in the compacted cut-off must be avoided by blending of borrow materials and scarifying to bond successive lifts.
- Mixed-in-Place Piles. Overlapping mixed-in-place piles of cement and natural soil forms a cofferdam with some shear resistance around an excavation.
- Slurry-filled Trench. Concurrent excavation of a straight-sided trench and backfilling with slurry of bentonite with natural soil is done. Alternatively, a cement bentonite mix can be used in a narrower trench where coarser gravel occurs. In certain cases, tremie concrete may be placed, working upward from the base of a slurry-filled trench, to form a permanent peripheral wall.

6.4. Design of Drainage Blanket And Filters

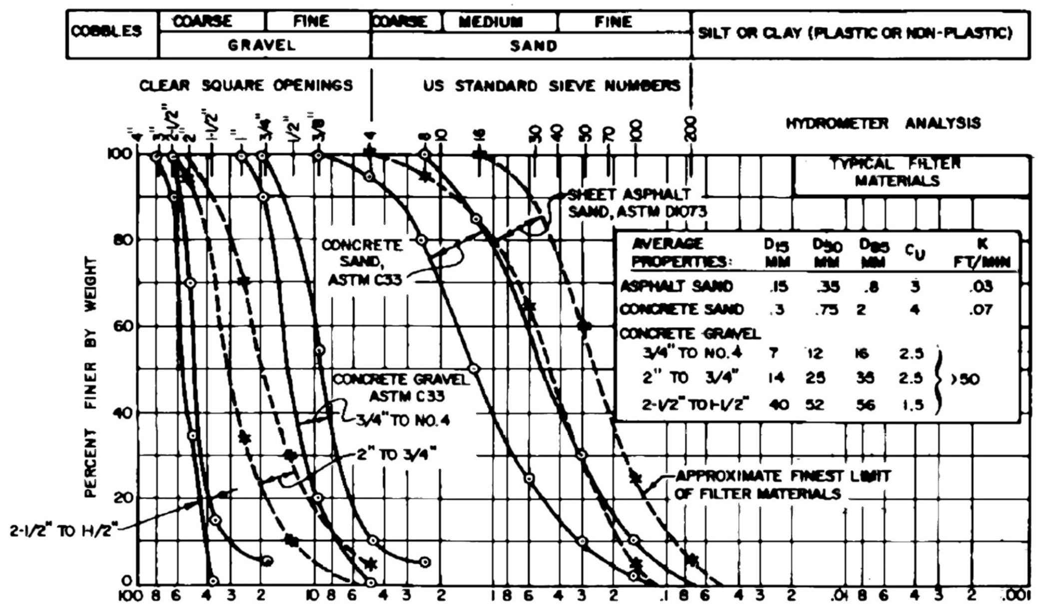
6.4.1. Filters

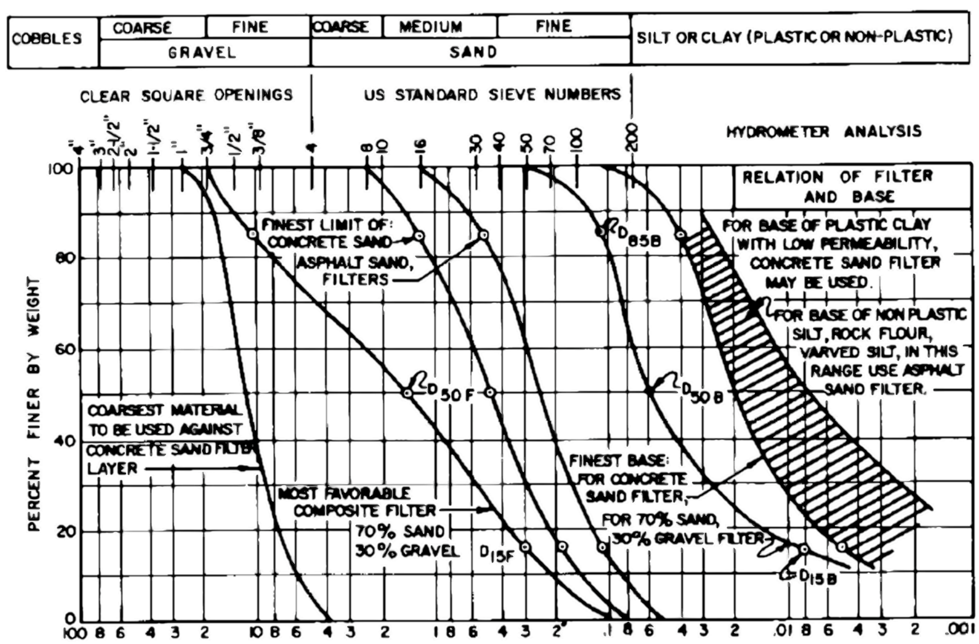
If water flows from silt to gravel, the silt will wash into the interstices of the gravel. This could lead to the following, which must be avoided:

1) The loss of silt may continue, causing creation of a cavity.
2) The silt may clog the gravel, stopping flow, and causing hydrostatic pressure build-up.

The purpose of filters is to allow water to pass freely across the interface (filter must be coarse enough to avoid head loss) but still be sufficiently fine to prevent the migration of fines. The filter particles must be durable, e.g., certain crushed limestones may dissolve. Filter requirements apply to all permanent subdrainage structures in contact with soil, including wells. See Figure 6-6 for protective filter design criteria.

Figure 6-6 Design Criteria for Protective Filters





GENERAL REQUIREMENTS:

1. TO AVOID HEAD LOSS IN FILTER: $\frac{D_{15F}}{D_{15B}} > 4$, AND PERMEABILITY OF FILTER MUST BE LARGE ENOUGH TO SUFFICE FOR PARTICULAR DRAINAGE SYSTEM.
2. TO AVOID MOVEMENT OF PARTICLES FROM BASE: $\frac{D_{15F}}{D_{85B}} < 5$, $\frac{D_{50F}}{D_{50B}} < 25$, $\frac{D_{15F}}{D_{15B}} < 20$

   FOR VERY UNIFORM BASE MATERIAL ($C_U < 1.5$): D_{15F}/D_{85B} MAY BE INCREASED TO 6

   FOR BROADLY GRADED BASE MATERIAL ($C_U > 4$): D_{15F}/D_{15B} MAY BE INCREASED TO 40
3. TO AVOID MOVEMENT OF FILTER IN DRAIN PIPE PERFORATIONS OR JOINTS: D_{85F}/SLOT WIDTH > (1.2 TO 1.4), D_{85F}/HOLE DIAMETER > (1.0 TO 1.2)
4. TO AVOID SEGREGATION, FILTER SHOULD CONTAIN NO SIZES LARGE THAN 3".
5. TO AVOID INTERNAL MOVEMENT OF FINES, FILTER SHOULD HAVE NO MORE THAN 5% PASSING No. 200 SIEVE.

The filter may be too fine grained to convey enough water, to provide a good working surface, or to pass the water freely without loss of fines to a subdrain pipe. For this condition, a second filter layer is placed on the first filter layer; the first filter layer is then considered the soil to be protected, and the second filter layer is designed. The finest filter soil is often at the base, with coarser layers above. This is referred to as reversed or inverted filters.

Concrete sand (ASTM C33, Specifications for Concrete Aggregates) suffices as a filter against the majority of fine-grained soils or silty or clayey sands. For non-plastic silt, varved silt, or clay with sand or silt lenses, use asphalt sand (ASTM D1073, Specifications for Fine Aggregates for Bituminous Paving Mixtures) but always check the criteria in Figure 6-6. Locally available natural materials are usually more economical than processed materials, and should be used where they meet filter criteria. The fine filter layer can be replaced with plastic filter cloths under the following conditions:[146]

- Non-woven filter cloths, or woven filter cloths with less than 4% open area should not be used where silt is present in sandy soils. A cloth with an equivalent opening size (EOS) equal to the No. 30 sieve and an open area of 36% will retain sands containing silt.
- When stones are to be dropped directly on the cloth, or where uplift pressure from artesian water may be encountered, the minimum tensile strengths (ASTM D1682, Tests for Breaking Load and Elongation of Textile Fabrics) in the strongest and weakest directions should be not less than 350 and 200 lbs. respectively. Elongation at failure should not exceed 35%. The minimum burst strength should be 520 psi (ASTM D751, Testing Coated Fabrics). Where the cloths are used in applications not requiring high strength or abrasion resistance, the strength requirements may be relaxed.
- Cloths made of polypropylene, polyvinyl chloride and polyethylene fibres do not deteriorate under most conditions, but they are affected by sunlight, and should be protected from the sun. Materials should be durable against ground pollutants and insect attack, and penetration by burrowing animals.
- Where filter cloths are used to wrap collection pipes or in similar applications, backfill should consist of clean sands or gravels graded such that the D85 is greater than the EOS of the cloth. When trenches are lined with filter cloth, the collection pipe should be separated from the cloth by at least six inches of granular material.
- Cloths should be made of monofilament yarns, and the absorption of the cloth should not exceed 1% to reduce possibility of fibres swelling and changing EOS and percent of open area.[147]

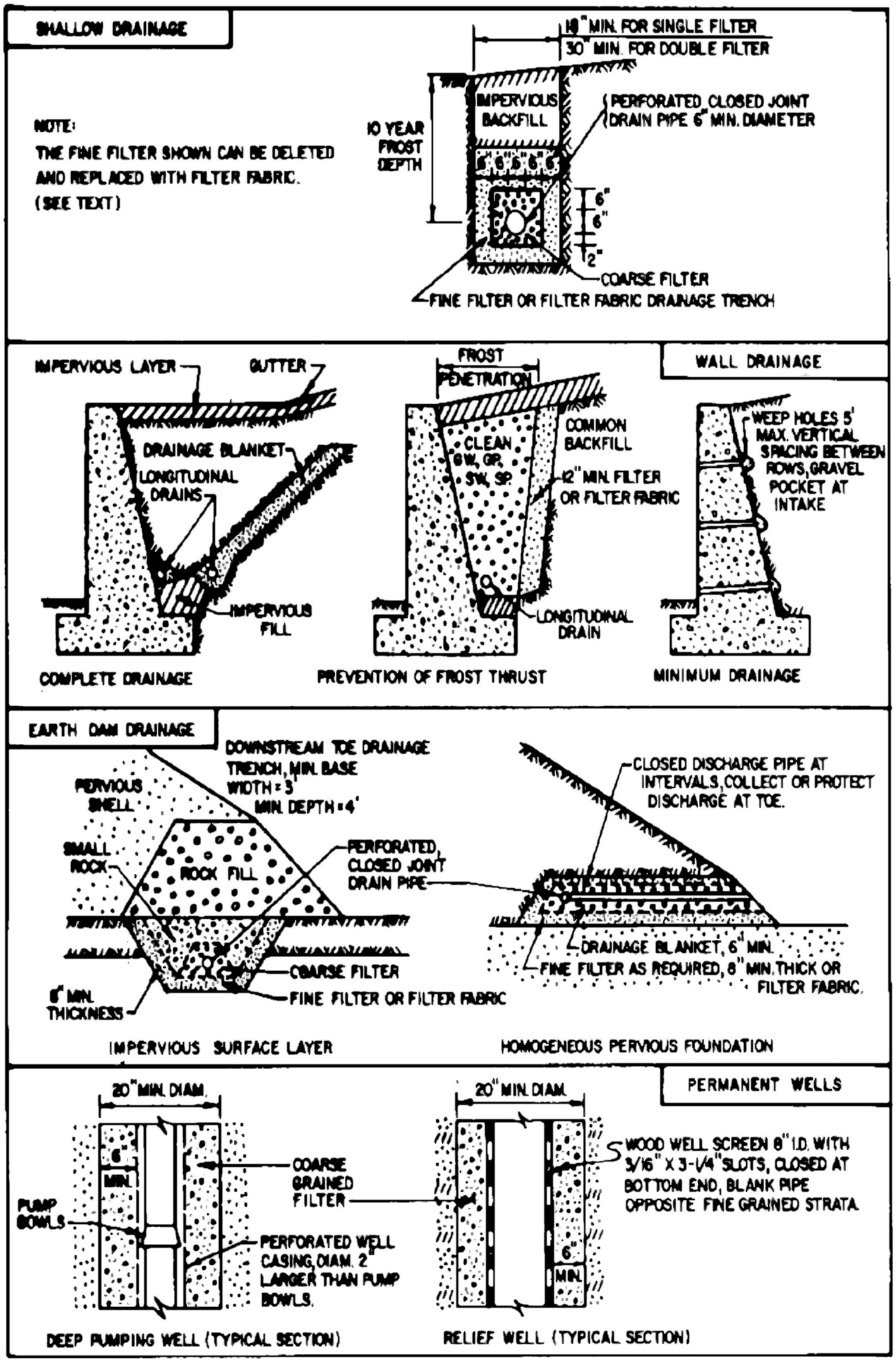
6.4.2. Drainage Blanket

Figure 6-7 shows typical filter and drainage blanket installations.

[146] Calhoun, C.C., Jr., Compton, J.R., Strohm, W.E. Jr., Performance of Plastic Filter Cloths as a Replacement for Granular Materials, Highway Research Record Number 373, Highway Research Board, 1971.

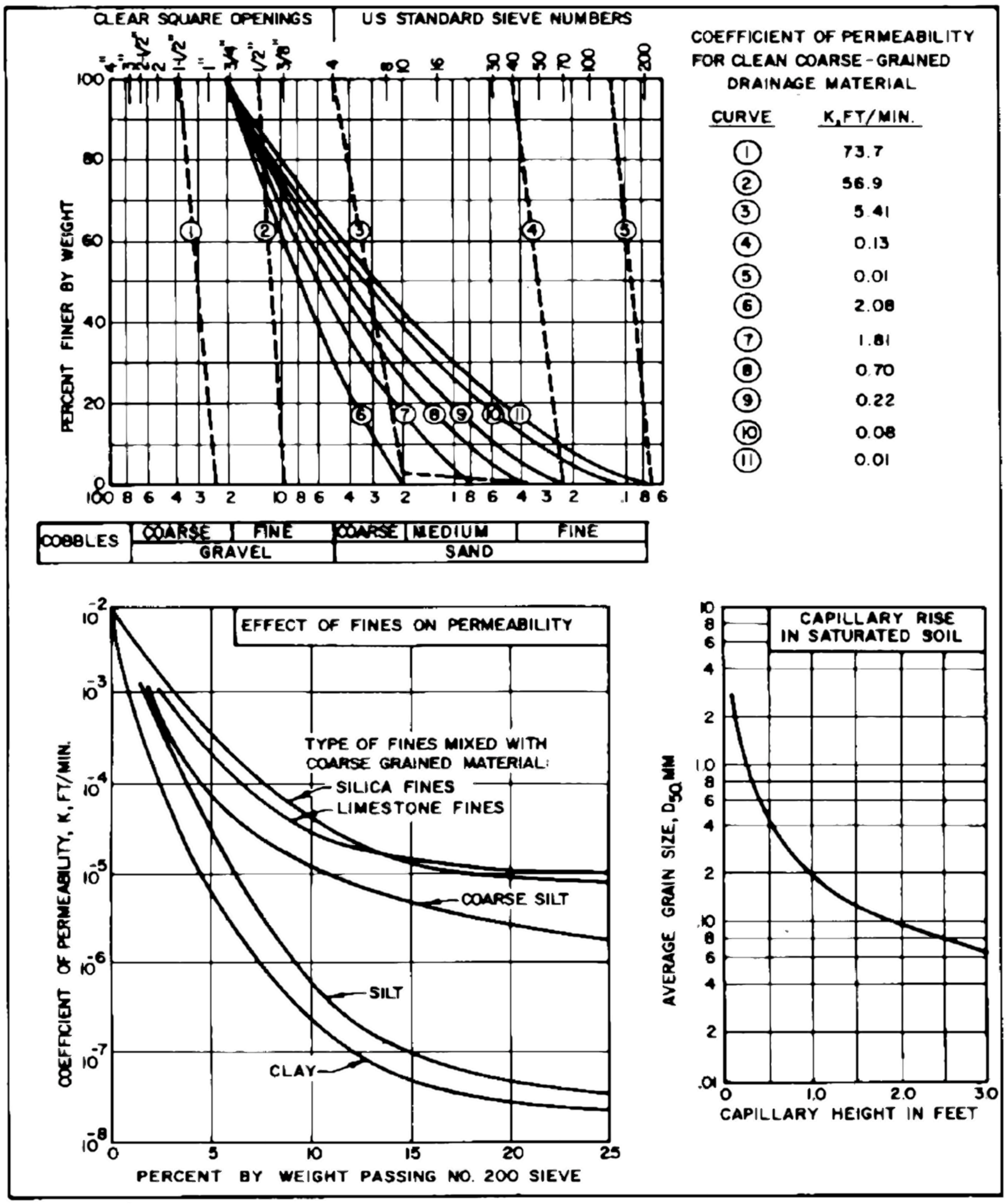
[147] For further guidance on types and properties of filter fabrics see Koerner, R.M. and Welsh, J.P., Construction and geotechnical engineering Using Synthetic Fabrics, John Wiley & Sons, Inc., 1980.

Figure 6-7 Typical Filter and Drainage Blanket Applications



- Permeability. Figure 6-8 gives typical coefficients of permeability for clean, coarse-grained drainage material and the effect of various percentages of fines on permeability. Mixtures of about equal parts gravel with medium to coarse sand have a permeability of approximately 1 fpm. Single sized, clean gravel has a permeability exceeding 50 fpm. For approximate relationship of permeability versus effective grain size D_{10}, see Figure 6-2.

Figure 6-8 Permeability and Capillarity of Drainage Materials[148]



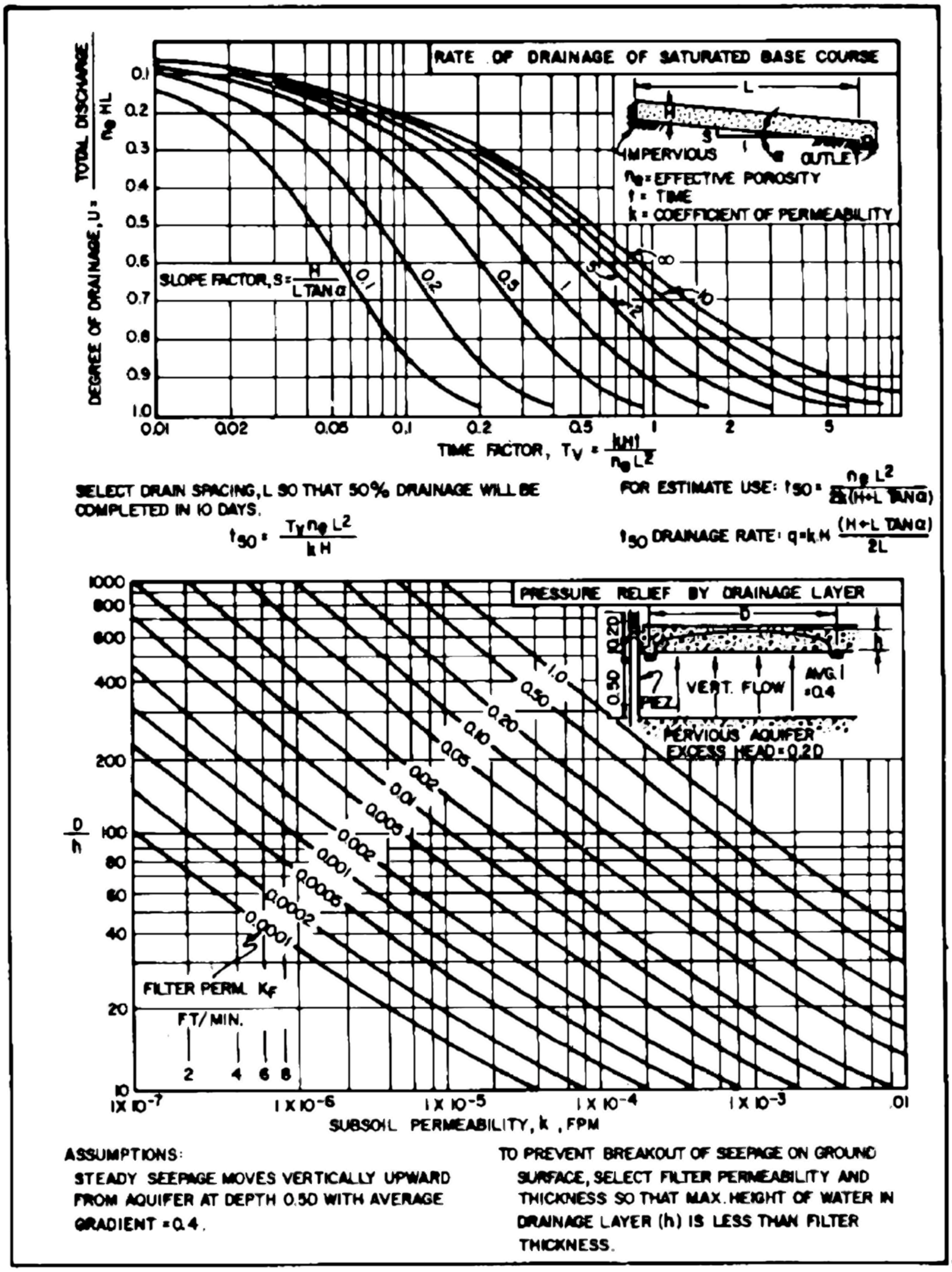
∞ Drainage Capacity. Estimate the quantity of water that can be transmitted by a drainage blanket using Equation 6-3. Uplift pressures that may be tolerated at the point farthest from the outlet of the

[148] Barber, E.W., Subsurface Drainage of Highways, Highway Research Board Bulletin 209, Highway Research Board, Washington, D.C.

drainage blanket limit the gradient. Increase gradients and flow capacity of the blanket by providing closer spacing of drainpipes within the blanket.

- Pressure Relief. See bottom panel of Figure 6-9 for combinations of drain pipe spacing, drainage course thickness, and permeability required for control of flow upward from an underlying aquifer under an average vertical gradient of 0.4.
- Rate of Drainage. See the top panel of Figure 6-9 for time rate of drainage of water from a saturated base course beneath a pavement. Effective porosity is the volume of drainable water in a unit volume of soil. It ranges from 25% for a uniform material such as medium to coarse sand, to 15% for a broadly graded sand-gravel mixture.

Figure 6-9 Analysis of Drainage Layer Performance[149]



- Drainage Blanket Design. The following guidelines should be followed:
  - Gradation. Design in accordance with Figure 6-6.

149 Cedergen, H.R., Seepage Requirements of Filters and Pervious Bases, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 86, No. SM5, 1960.

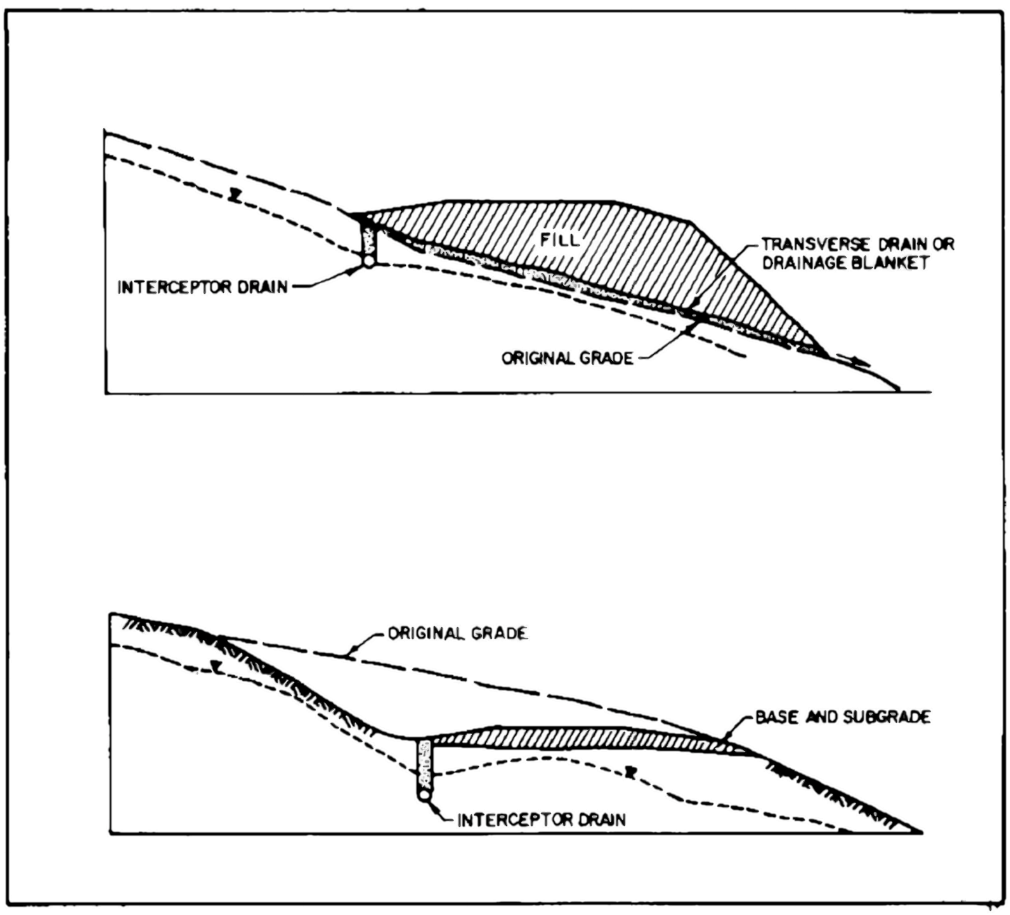
  - Thickness. Beneath, structures require a minimum of 12 inches for each layer with a minimum thickness of 24 inches overall. If placed on wet, yielding, uneven excavation surface and subject to construction operation and traffic, minimum thickness shall be 36 inches overall.

- Chemical Clogging. Filter systems (filter layers, fabrics, pipes can become chemically clogged by ferruginous (iron) and carbonate depositions and incrustations. Where the permanent subdrainage system is accessible, pipes with larger perforations (3/8”) and increased thickness of filter layers can be used. For existing facilities, a weak solution of hydrochloric acid can be used to dissolve carbonates.

6.4.3. Intercepting Drains

Intercepting drains consist of shallow trenches with collector pipes surrounded by drainage material, placed to intercept seepage moving horizontally in an upper pervious stratum. To design proper control drains, determine the drawdown and flow to drains by flow net analysis. Figure 6-10 shows typical placements of intercepting drains for roadways on a slope.

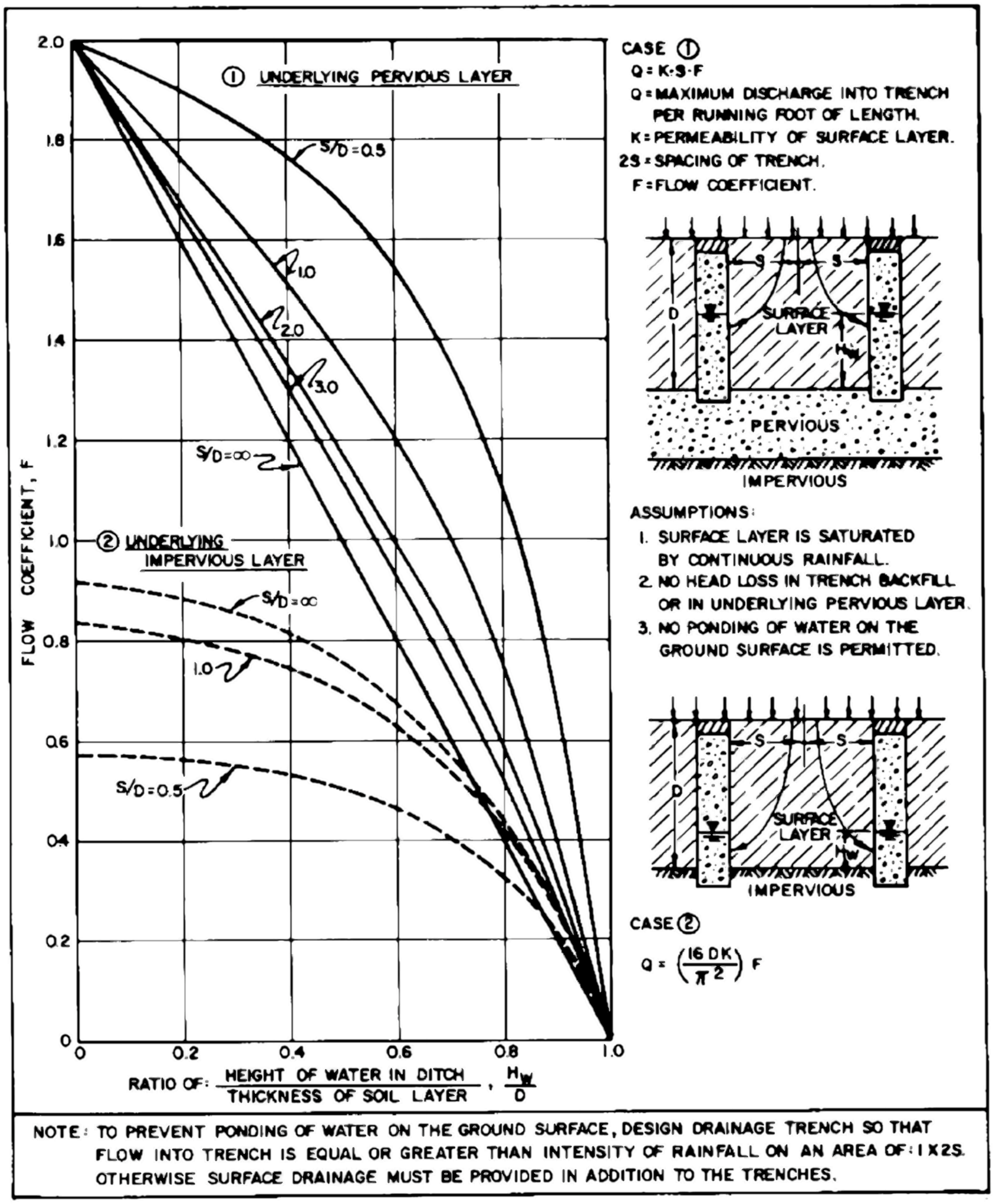
Figure 6-10 Intercepting Drains for Roadways on a Slope



6.4.4. Shallow Drains for Ponded Areas

Drains consisting of shallow stone trenches with collector pipes can be used to collect and control surface run-off. See Figure 6-11 for determination of rate of seepage into drainage trenches. If sufficient capacity cannot be provided in trenches, add surface drainage facilities.

Figure 6-11 Rate of Seepage into Drainage Trench[150]



6.4.5. Pipes for Drainage Blankets and Filters

Normally, perforated wall pipes of metal or plastic or porous wall concrete pipes are used as collector pipes. Circular perforations should generally not be larger than 3/8 inch. Filter material must be graded according to the above guidelines.

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[150] Kirkham, D., Seepage Into Ditches in the Case of a Plane Water Table and an Impervious Substratum, Transactions, American Geophysical Union, Washington, D.C., June, 1950, and Kirkham, D., Seepage Into Ditches From a Plane Water Table Overlying a Gravel Substratum, Journal of Geophysical Research, American Geophysical Union, Washington, D.C., April, 1960.

Pipes should be checked for strength. Certain deep buried pipes may need a cradle. Check for corrosiveness of soil and water; certain metal pipes may not be appropriate.

Since soil migration may occur, even in the best-designed systems, install cleanout points so that the entire system can be flushed and snaked.

6.5. Wellpoint Systems and Deep Wells

6.5.1. Methods

Excavation below groundwater in soils having permeability greater than 10^{-3} fpm generally requires dewatering to permit construction in the dry. For materials with a permeability between 10^{-3} and 10^{-5} fpm, the amount of seepage may be small but piezometric levels may need to be lowered in order to stabilize slopes or to prevent softening of subgrades. For intermediate depths, wellpoint systems or sumps normally accomplish drawdown.

Deep drainage methods include deep pumping wells, relief wells, and deep-sheeted sumps. These are appropriate when excavation exceeds a depth that can be dewatered efficiently by wellpoint systems alone or when the principal source of seepage is from lower permeable strata.

- Construction Controls. For important construction dewatering, install piezometers below the base of excavations and behind slopes or cofferdams to check on the performance and adequacy of drainage system.
- Settlement Effects. Where dewatering lowers the water levels in permeable strata adjacent to compressible soils, settlement may result. See 5.4 for methods of settlement evaluation.

6.5.2. Wellpoint Systems

Wellpoints consist of 1 1/2" or 2" diameter pipes with a perforated bottom section protected by screens. They are jetted or placed in a prepared hole and connected by a header pipe to suction pumps.

- Applicability. Wellpoints depend upon the water flowing by gravity to the well screen. Pumping methods for gravity drainage generally are not effective when the average effective grain size of a soil D_{10} is less than 0.05 mm. In varved or laminated soils where silty fine sands are separated by clayey silts or clay, gravity drainage may be effective even if the average material has as much as 50% smaller than 0.05 mm. Compressible, fine-grained materials containing an effective grain size less than 0.01 mm can be drained by providing a vacuum seal at the ground surface around the wellpoint, utilizing atmospheric pressure as a consolidating force. See Section 4 for limitations due to iron and carbonate clogging.
- Capacity. Wellpoints ordinarily produce a drawdown between 15 and 18 feet below the centre of the header. For greater drawdown, install well points in successive tiers or stages as excavation proceeds. Discharge capacity is generally 15 to 30 gpm per point. Points are spaced between 3 and 10 feet apart. In finely stratified or varved materials, use minimum spacing of points and increase their effectiveness by placing sand in the annular space surrounding the well point.
- Analysis. Well point spacing usually is so close that the seepage pattern is essentially two-dimensional. Analyse total flow and drawdown by flow net procedure. For fine sands and coarser material, the quantity of water to be removed controls well point layout. For silty soils, the quantity pumped is relatively small and the number and spacing of well points will be influenced by the time available to accomplish the necessary drawdown.

6.5.3. Sumps

For construction convenience or to handle a large flow in pervious soils, sumps can be excavated with soldier beam and horizontal wood lagging. Collected seepage is removed with centrifugal pumps placed within the sump. Analyse drawdown and flow quantities by approximating the sump with an equivalent circular well of large diameter.

Sheeted sumps are infrequently used. Unsheeted sumps are far more common, and are used primarily in dewatering open shallow excavations in coarse sands, clean gravels, and rock.

6.5.4. Electro-Osmosis

This is a specialized procedure utilized in silts and clays that are too fine-grained to be effectively drained by gravity or vacuum methods.

6.5.5. Pumping Wells

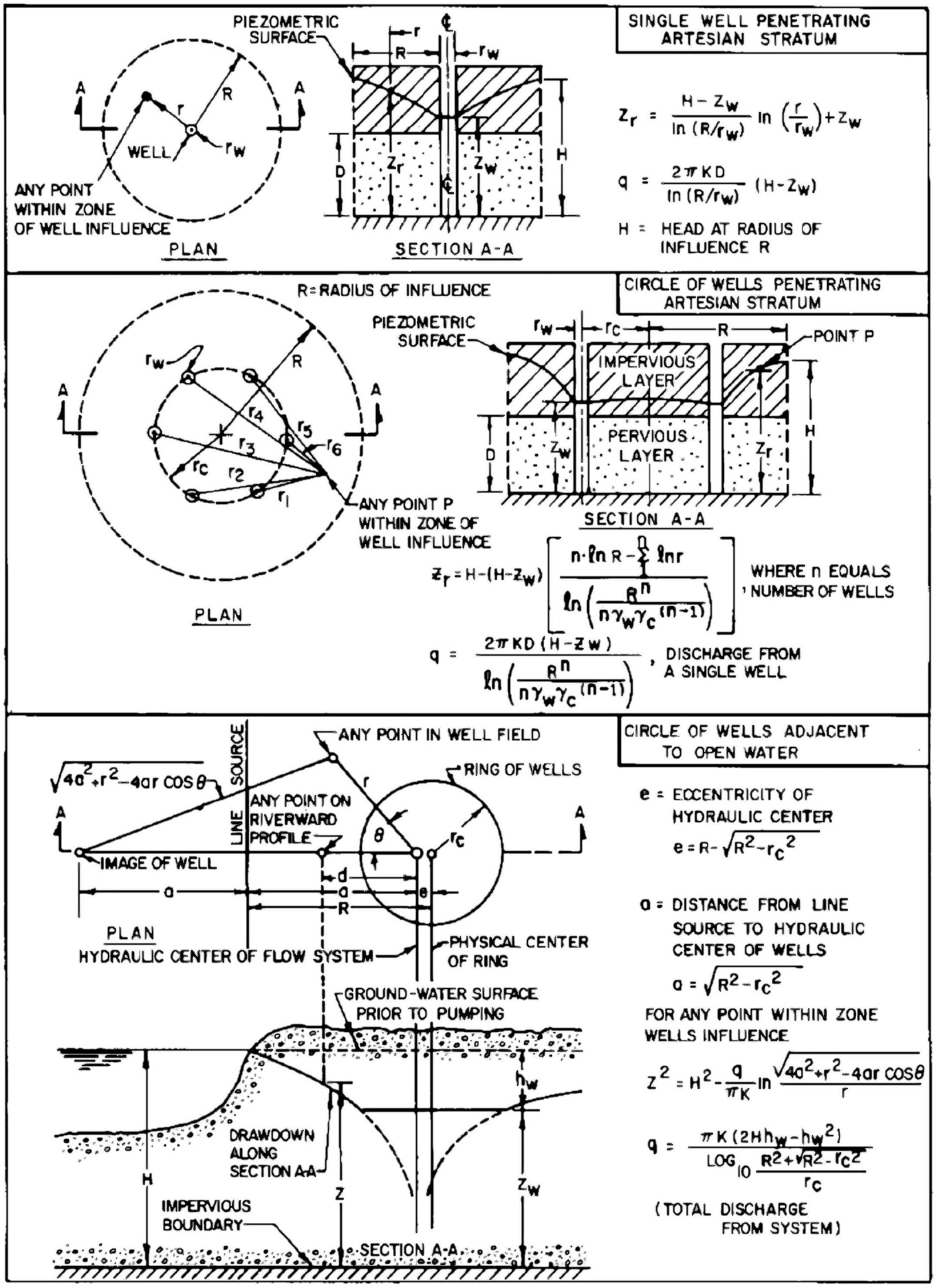
Drilling a hole of sufficient diameter to accommodate a pipe column and filter, installing a well casing, and placing filter material in the annular space surrounding the casing forms these wells. Pumps may be either the turbine type with a motor at the surface and pipe column with pump bowls hung inside the well, or a submersible pump placed within the well casing.

- Applications. Deep pumping wells are used if:
    a. dewatering installations must be kept outside the excavation area,
    b. large quantities are to be pumped for the full construction period, and
    c. pumping must commence before excavation to obtain the necessary time for drawdown.

See Figure 6-12 for analysis of drawdown and pumping quantities for single wells or a group of wells in a circular pattern. Deep wells may be used for gravels to silty fine sands, and water bearing rocks. See the previous section for limitations due to iron and carbonate clogging.

Bored shallow wells with suction pumps can be used to replace wellpoints where pumping is required for several months or in silty soils where correct filtering is critical.

Figure 6-12 Groundwater Lowering by Pumping Wells[151]



$$Z_r = \frac{H - Z_w}{\ln(R/r_w)} \ln\left(\frac{r}{r_w}\right) + Z_w$$

$$q = \frac{2\pi KD}{\ln(R/r_w)}(H - Z_w)$$

$$Z_r = H - (H - Z_w)\left[\frac{n \cdot \ln R - \sum^{n} \ln r}{\ln\left(\frac{R^n}{n \gamma_w \gamma_c^{(n-1)}}\right)}\right]$$

$$q = \frac{2\pi KD(H - Z_w)}{\ln\left(\frac{R^n}{n \gamma_w \gamma_c^{(n-1)}}\right)}$$

$$e = R - \sqrt{R^2 - r_c^2}$$

$$a = \sqrt{R^2 - r_c^2}$$

$$Z^2 = H^2 - \frac{q}{\pi K} \ln \frac{\sqrt{4a^2 + r^2 - 4ar\cos\theta}}{r}$$

$$q = \frac{\pi K (2Hh_w - h_w^2)}{\log_{10} \frac{R^2 + \sqrt{R^2 - r_c^2}}{r_c}}$$

- Special Methods. Ejector or eductor pumps may be utilized within wellpoints for lifts up to about 60 feet. The ejector pump has a nozzle arrangement at the bottom of two small diameter riser pipes that

[151] Avery, S.B., Analysis of Groundwater Lowering Adjacent to Open Water, Proceedings, ASCE, Vol. 77, 1951.

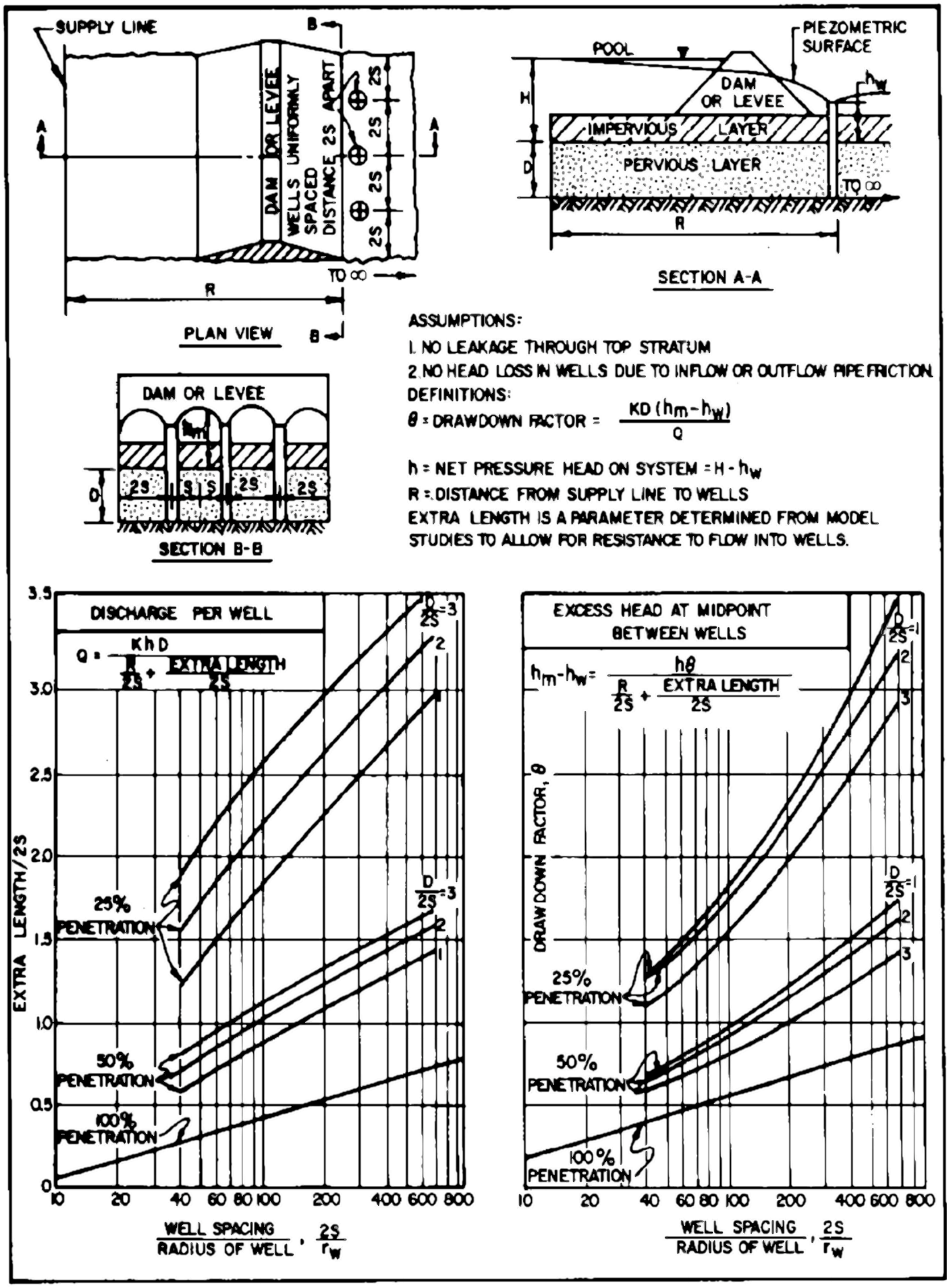
remove water by the Venturi principle. They are used in lieu of a multistage wellpoint system and if the large pumping capacity of deep wells is not required. Their primary application is for sands, but with proper control, they can also be used in silty sands and sandy silts.

6.5.6. Relief Wells

These wells are sand columns used to bleed water from underlying strata containing artesian pressures, and to reduce uplift forces at critical location. Relief wells may be tapped below ground by a collector system to reduce bank pressures acting in the well.

- Applications. Relief wells are frequently used as construction expedients, and in situations where a horizontal drainage course may be inadequate for pressure relief of deep foundations underlain by varved or stratified soils or soils whose permeability increases with depth.
- Analysis. See Figure 6-13 for analysis of drawdown produced by line of relief wells inboard of a long dike. To reduce uplift pressures h_m midway between the wells to safe values, vary the well diameter, spacing, and penetration to obtain the best combination.

Figure 6-13 Drainage of Artesian Layer by Line of Relief Wells[152]



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[152] Corps of Engineers, Soil Mechanics Design, Seepage Control, Engineering Manual, Civil Works Construction, Chapter I, Part CXIX, Department of the Army.

6.6. Linings for Reservoirs and Pollution Control Facilities

6.6.1. Purpose

Linings are used to reduce water loss, to minimize seepage that can cause instability in embankments, and to keep pollutants from migrating to groundwater sources as in holding ponds at sewage treatment and chemical facilities, and in sanitary landfills.[153]

6.6.2. Types

Table 6-2 lists types of linings appropriate where wave forces are insignificant. Where erosive forces are present, combine lining with slope protection procedure.

6.6.3. Subdrainage

If the water level in the reservoir may fall below the surrounding groundwater level, a permanent subdrainage system should be provided below the lining.

6.6.4. Investigation For Lining

Check any potential lining for reaction to pollutants (e.g., synthetic rubber is subject to attack by hydrocarbons), potential for insect attack (e.g., certain synthetic fabrics may be subject to termite attack), and the potential for burrowing animals breaching the lining.

6.7. Erosion Control

6.7.1. General

The design of erosion controls must consider the volume of runoff from precipitation, the runoff velocity, and the amount of soil loss.

- Volume of Runoff. The volume of runoff depends on the amount of precipitation, ground cover, and topography.[154]
- Amount of Soil Loss. Soil losses can be estimated using the Universal Soil Loss Equation developed by the Soil Conservation Service:

Equation 6-6: $$A = (EI)KLS$$

where

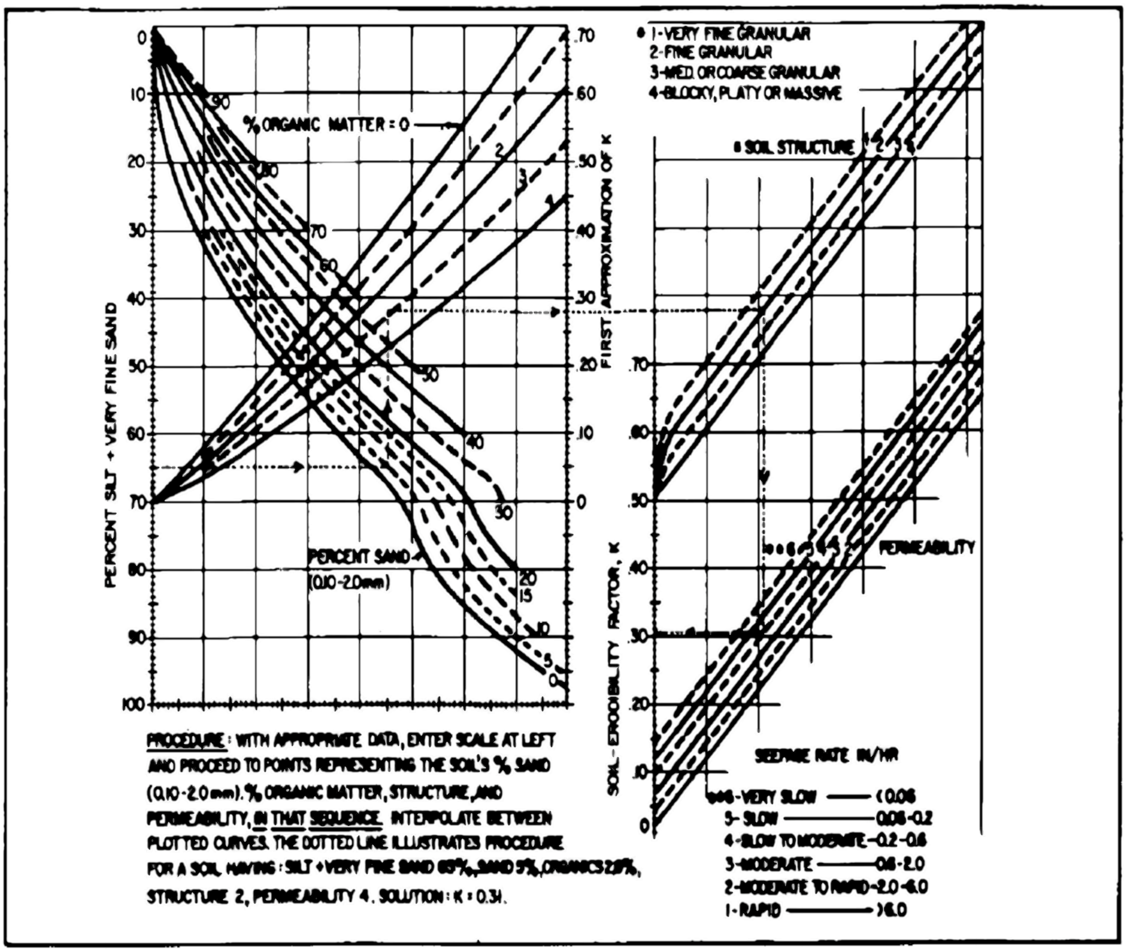
- A = computed soil loss per acre, in tons
- EI = rainfall erosion index
- K = soil erodibility factor
- L = slope length factor
- S = slope gradient factor

[153] For further guidance, see Cold Regions Research and Engineering Laboratory, Wastewater Stabilization Pond Linings, Special Report 28, Department of the Army, November 1978.

[154] For guidance on evaluating the volume of runoff, see Soil Conservation Service, U.S. Department of Agriculture, Urban Hydrology for Small Watersheds, Technical Release No. 55, Engineering Division, 1975.

EI, L, and S values should be obtained from local offices of the U.S. Soil Conservation Service. K values may be determined from published data on a particular locality. In the absence of such data, it may be roughly estimated from Figure 6-14.

Figure 6-14 Nomograph for Determining Soil Erodibility (K) for Universal Soil Loss Equation[155]



6.7.2. Investigation

Where erosion can be expected during earthwork construction, on-site investigations should include:

1) Field identification and classification for both agricultural textures and the Unified system,
2) Sampling for grain size distribution, Atterberg limits and laboratory classification, and
3) Determination of in-place densities.

6.7.3. Surface Erosion Control

∞ Construction Scheduling. Schedule construction to avoid seasons of heavy rains. Winds are also seasonal, but are negligible in impact compared to water erosion.

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155 Highway Research Board, Erosion Control on Highway Construction, National Cooperative Highway Research Program, Synthesis of Highway Practice 18, 1973.

- Soil Type. Avoid or minimize exposure of highly erodible soils. Sands easily erode but are easy to trap. Clays are more erosion resistant, but one eroded, are more difficult to trap.
- Slope Length and Steepness. Reduce slope lengths and steepness to reduce velocities. Provide benches on slopes at maximum vertical intervals of 30 feet.
- Cover. Cover quickly with vegetation, such as grass, shrubs and trees, or other covers such as mulches. Straw mulch applied at 2 tons/acre may reduce soil losses as much as 98% on gentle slopes. Other mulches include asphalt emulsion, paper products, jute, cloth, straw, wood chips, sawdust, netting of various natural and man-made fibres, and, in some cases, gravel.
- Soil Surface. Ridges perpendicular to flow and loose soil provide greater infiltration.
- Exposed Area. Minimize the area opened at any one time. Retain as much natural vegetation as possible. Leave vegetation along perimeters to control erosion and act as a sediment trap.
- Diversion. Minimize flow over disturbed areas, such as by placing a berm at the top of a disturbed slope.
- Sprinkling. Control dust by sprinkling of exposed areas.
- Sediment Basins. Construct debris basins to trap debris and silt before it enters streams.

6.7.4. Channel Linings

Table 6-3 presents guidelines for minimizing erosion of earth channels and grass covered channels.

Table 6-3 Limiting Flow Velocities to Minimise Erosion[156]

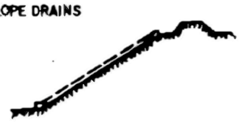
Soil Type	*Permissible Velocity for Bare Channel*	**PERMISSIBLE VELOCITY WITH CHANNEL VEGETATION**		
		6" to 10" in height	11" to 24" in height	Over 30" in height
Sand, Silt, Sandy loam, Silty loam	**1.5**	2.0 to 3.0	2.5 to 3.5	3.0 to 4.0
Silty clay loam, Silty clay	2.0	3.0 to 4.0	3.5 to 4.5	4.0 to 5.0
Clay	2.5	3.0 to 5.0	3.0 to 5.5	3.0 to 6.0

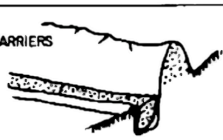
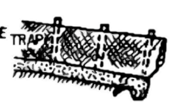
6.7.5. Sediment Control

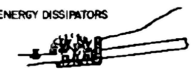
Typical sediment control practices are included in Table 6-4.

[156] Soil Conservation Service, U.S. Department of Agriculture, Minimizing Erosion in Urbanizing Areas, Madison, WI, 1972.

Table 6-4 Typical Erosion Control Practice

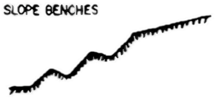
Treatment Practice	Advantages	Problems
FILL SLOPES		
BERMS AT TOP OF EMBANKMENT 	Prevent runoff from embankment surface from flowing over face of fill Collect runoff for slope drains or protected ditch Can be placed as a part of the normal construction operation and incorporated into fill or shoulders	Cooperation of construction operators to place final lifts at edge or shaping into berm Difficult to compact outside lift when work is resumed Sediment buildup and berm and slope failure
SLOPE DRAINS 	Prevent fill slope erosion caused by embankment surface runoff Can be constructed of full or half section pipe, bituminous, metal, concrete, plastic, or other waterproof material Can be extended as construction progresses May be either temporary or permanent	Permanent construction as needed may not be considered desirable by contractor Removal of temporary drains may disturb growing vegetation Energy dissipation devices are required at the outlets
FILL BERMS OR BENCHES 	Slows velocity of slope runoff Collects sediments Provides access for maintenance Collects water for slope drains May utilize waste	Requires additional fill material if waste is not available May cause sloughing Additional construction area may be needed
SEEDING/MULCHING 	Timely application of mulch and seeding decreases the period a slope is subject to severe erosion Mulch that is cut in or otherwise anchored will collect sediment. The furrows made will also hold water and sediment	Seeding season may not be favorable Not 100 percent effective in preventing erosion Watering may be necessary Steep slopes or locations with high velocities may require supplemental treatment.

Treatment Practice	Advantages	Problems
PROTECTION OF ADJACENT PROPERTY		
BRUSH BARRIERS 	Use slashing and logs from clearing operation Can be covered and seeded rather than removed Eliminates need for burning or disposal off right-of-way	May be considered unsightly in urban areas
STRAW BALE BARRIERS 	Straw is readily available in many areas When properly installed, they filter sediment and some turbidity from runoff	Requires removal Subject to vandal damage Flow is slow through straw requiring considerable area
SEDIMENT TRAPS 	Collect much of the sediment spill from fill slopes and storm drain ditches Inexpensive Can be cleaned and expanded to meet need	Does not eliminate all sediment and turbidity Space is not always available
SEDIMENT POOLS 	Can be designed to handle large volumes of flow Both sediment and turbidity are removed May be incorporated into permanent erosion control plan	Requires prior planning, additional construction area and/or flow easement If removal is necessary, can present a major effort during final construction stage Clean-out volumes can be large Access for clean-out not always convenient; Anti-seepage baffles required for permanent construction
FENCE TRAP 	Low cost Temporary measure can be erected with minimum supervision	Some maintenance needed depending on length of time in place

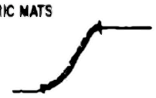
Treatment Practice	Advantages	Problems
PROTECTION OF ADJACENT PROPERTY (continued)		
ENERGY DISSIPATORS 	Slow velocity to permit sediment collection and to minimize channel erosion off project	Collects debris and requires cleaning Requires special design and construction of large shot rock or other suitable material from project
LEVEL SPREADERS 	Spreads channel or pipe flow to sheet flow Avoids channel easements and construction off project Simple to construct	Adequate spreader length may not be available Sodding of overflow berm is usually required Must be a part of the permanent erosion control effort Maintenance forces must maintain spreader until no longer required
PROTECTION OF STREAM		
CONSTRUCTION DIKE 	Permits work to continue during normal stream stages Controlled flooding can be accomplished during periods of inactivity	Usually requires pumping of work site water into sediment pond Subject to erosion from stream and from direct rainfall on dike
COFFERDAM 	Work can be continued during most anticipated stream conditions Clear water can be pumped directly back into stream No material deposited in stream	Expensive
TEMPORARY STREAM CHANNEL CHANGE	Prepared channel keeps normal flows away from construction	New channel usually will require protection Stream must be returned to old channel and temporary channel refilled
PROTECTION OF STREAM (continued)		
RIP RAP 	Sacked sand with cement or stone easy to stockpile and place Can be installed in increments as needed	Expensive
TEMPORARY CULVERTS FOR HAUL ROADS 	Eliminates stream turbulence and turbidity Provides unobstructed passage for fish and other aquatic life Capacity for normal flow can be provided with storm water flowing over the roadway	Space not always available without conflicting with permanent structure work May be expensive, especially for larger sizes of pipe Subject to washout
ROCK-LINED LOW-LEVEL CROSSING 	Minimizes stream turbidity Inexpensive May also serve as ditch check or sediment trap	May not be fordable during rainstorms During periods of low flow, passage of fish may be blocked

Treatment Practice	Advantages	Problems
DITCHES		
CHECK DAMS 	Maintains low velocities Catches sediment Can be constructed of logs, shot rock, lumber, masonry or concrete, gabions, sand bags	Close spacing on steep grades Require clean-out Unless keyed at sides and bottom, erosion may occur
SEDIMENT TRAPS/ STRAW BALE FILTERS 	Can be located as necessary to collect sediment during construction Clean-out often can be done with on-the-job equipment Simple to construct	Little direction on spacing and size Sediment disposal may be difficult Specification must include provisions for periodic clean-out May require seeding, sodding or pavement when removed during final cleanup
SODDING 	Easy to place with a minimum of preparation Can be repaired during construction Immediate protection May be used on sides of paved ditches to provide increased capacity	Requires water during first few weeks Sod not always available Will not withstand high velocity or severe abrasion from sediment load
SEEDING WITH MULCH AND MATTING 	Usually least expensive Effective for ditches with low velocity Easily placed in small quantities with inexperienced personnel	Will not withstand medium to high velocity Requires anchoring
PAVING, RIPRAP, RUBBLE	Effective for high velocities May be part of the permanent erosion control effort	Cannot always be placed when needed because of construction traffic and final grading and dressing Initial cost is high

Treatment Practice	Advantages	Problems
ROADWAY SURFACE		
CROWNING TO DITCH OR SLOPING TO SINGLE BERM 	Directing the surface water to a prepared or protected ditch minimizes erosion	Requires good construction procedures Can cause local stability problems (sloughing)
COMPACTION	The final lift of each day's work should be well compacted and bladed to drain to ditch or berm section Loose or uncompacted material is more subject to erosion	Requires good construction procedures
AGGREGATE COVER 	Minimizes surface erosion Permits construction traffic during adverse weather May be used as part of permanent base construction	Requires reworking and compaction if exposed for long periods of time Loss of surface aggregates can be anticipated
SEED/MULCH 	Minimizes surface erosion	Must be removed or is lost when construction of pavement is commenced
STONE FILLED GABION WALL 	Permits steeper slope No special backfill required Self draining	High cost Requires special techniques to install properly

Treatment Practice	Advantages	Problems
CUT SLOPES		
BERM AT TOP OF CUT 	Diverts water from cut Collects water for slope drains/paved ditches May be constructed before grading is started	Access to top of cut Difficult to build on steep natural slope or rock surface Concentrates water and may require channel protection or energy dissipation devices Can cause water to enter ground, resulting in sloughing of the cut slope
DIVERSION DIKE 	Collects and diverts water at a location selected to reduce erosion potential May be incorporated in the permanent project drainage	Access for construction May be continuing maintenance problem if not paved or protected Disturbed material or berm is easily eroded
SLOPE BENCHES 	Slows velocity of surface runoff Collects sediment Provides access to slope for seeding, mulching, and maintenance Collects water for slope drains or may divert water to natural ground	May cause sloughing of slopes if water infiltrates Requires additional construction area Not always possible due to poor material, etc. Requires maintenance to be effective Increases excavation quantities
SLOPE DRAINS (PIPE, PAVED, ETC) 	Prevents erosion on the slope Can be temporary or part of permanent construction Can be constructed or extended as grading progresses	Requires supporting effort to collect water Permanent construction is not always compatible with other project work Usually requires some type of energy dissipation

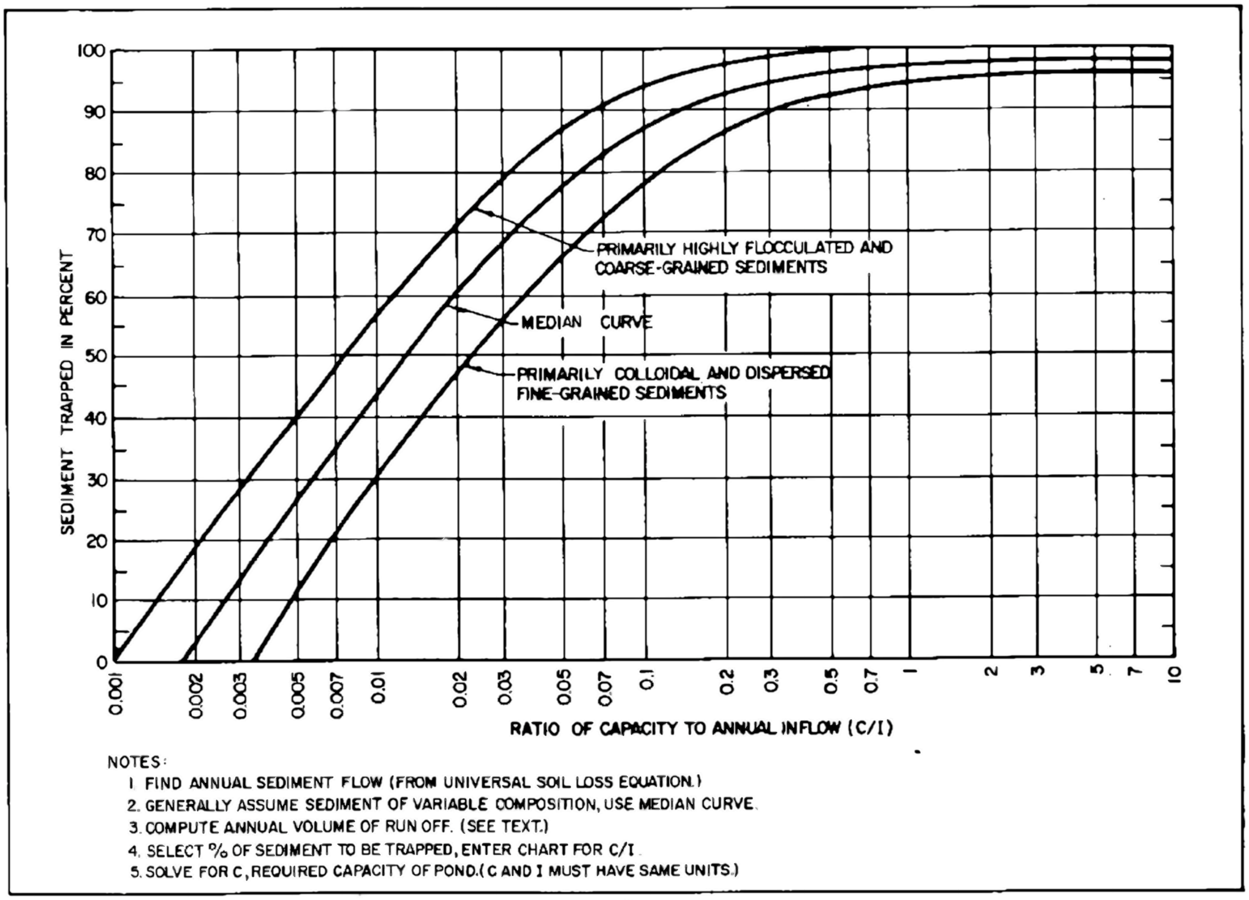
Treatment Practice	Advantages	Problems
CUT SLOPES (continued)		
SEEDING/MULCHING 	The end objective is to have a completely grassed slope. Early placement is a step in this direction. The mulch provides temporary erosion protection until grass is rooted. Temporary or permanent seeding may be used. Mulch should be anchored. Larger slopes can be seeded and mulched with smaller equipment if stage techniques are used.	Difficult to schedule high production units for small increments Time of year may be less desirable May require supplemental water Contractor may perform this operation with untrained or unexperienced personnel and inadequate equipment if stage seeding is required
SODDING 	Provides immediate protection Can be used to protect adjacent property from sediment and turbidity	Difficult to place until cut is complete Sod not always available May be expensive
SLOPE PAVEMENT, RIPRAP 	Provides immediate protection for high risk areas and under structures May be cast in place or off site	Expensive Difficult to place on high slopes May be difficult to maintain
TEMPORARY COVER 	Plastics are available in wide rolls and large sheets that may be used to provide temporary protection for cut or fill slopes Easy to place and remove Useful to protect high risk areas from temporary erosion	Provides only temporary protection Original surface usually requires additional treatment when plastic is removed Must be anchored to prevent wind damage

Treatment Practice	Advantages	Problems
CUT SLOPES (continued)		
SERRATED SLOPE 	Lowers velocity of surface runoff Collects sediment Holds moisture Minimizes amount of sediment reaching roadside ditches	May cause minor sloughing if water infiltrates Construction compliance
FABRIC MATS 	Effective for moderate to high embankment when crown vetch plantings are used Has lower cost features over other methods	Requires anchoring time to promote plant growth. May require periodic maintenance
BORROW AREAS		
SELECTIVE GRADING AND SHAPING	Water can be directed to minimize off-site damage Flatter slopes enable mulch to be cut into soil	May not be most economical work method for contractor
STRIPPING AND REPLACING OF TOPSOIL	Provides better seed bed Conventional equipment can be used to stockpile and spread topsoil	May restrict volume of material that can be obtained for a site Topsoil stockpiles must be located to minimize sediment damage Cost of rehandling material
DIKES, BERMS DIVERSION DITCHES SETTLING BASINS SEDIMENT TRAPS SEEDING & MULCH	See other practices	See other practices

- Traps. Traps are small and temporary, usually created by excavating and/or diking to a maximum height of five feet. Traps should be cleaned periodically.
- Ponds:
  - Size the outlet structure to accept the design storm.
  - Size the pond length, width and depth to remove the desired percentage of sediment. See Figure 6-15.[157]
  - If pond is permanent, compute volume of anticipated average annual sedimentation by the universal Soil Loss Equation. Multiply by the number of years between pond cleaning and by a factor of safety. This equals minimum required volume below water level. Dimensions of the pond can then be calculated based on the available area. The design depth of the pond should be approximately three to five feet greater than the calculated depth of sediment at the time of clearing.

[157] For design criteria see Gottschalk, L.C., Reservoir Sedimentation, Handbook of Applied Hydrology, Chow, Ed., Section 17-I, McGraw-Hill Book Company, 1964..

Figure 6-15 Capacity of Sediment Control Ponds[158]



Example Calculation:

- Annual soil loss in watershed = 0.9 acre-feet/year (from Universal Soil Loss Equation or other method, i.e. design charts)
- Desired pond efficiency = 70% or 0.63 acre-feet of sediment trapped each year.
- Annual volume of runoff from watershed draining into proposed pond = 400 acre-feet/yr.
- For 70% efficiency using median curve C/I = 0.032
- Required pond capacity C = 0.032 x 400 = 12.8 acre-feet.
- Assuming average depth of pond of 6 ft, required pond area about 2.1 acres. Pond should be cleaned when capacity reduced 50%.[159]
- Volume available for sediment = 50% x 12.8 = 6.4 acre-feet.
- Years between cleaning = 6.4/0.63 ≈ 10 years.

[158] Brune, G.M., Trap Efficiency of Reservoirs, Transactions, American Geophysical Union, Volume 34, No. 3, June, 1953.

[159] Trap efficiency decreases as volume of pond decreases; this has not been considered in the example.

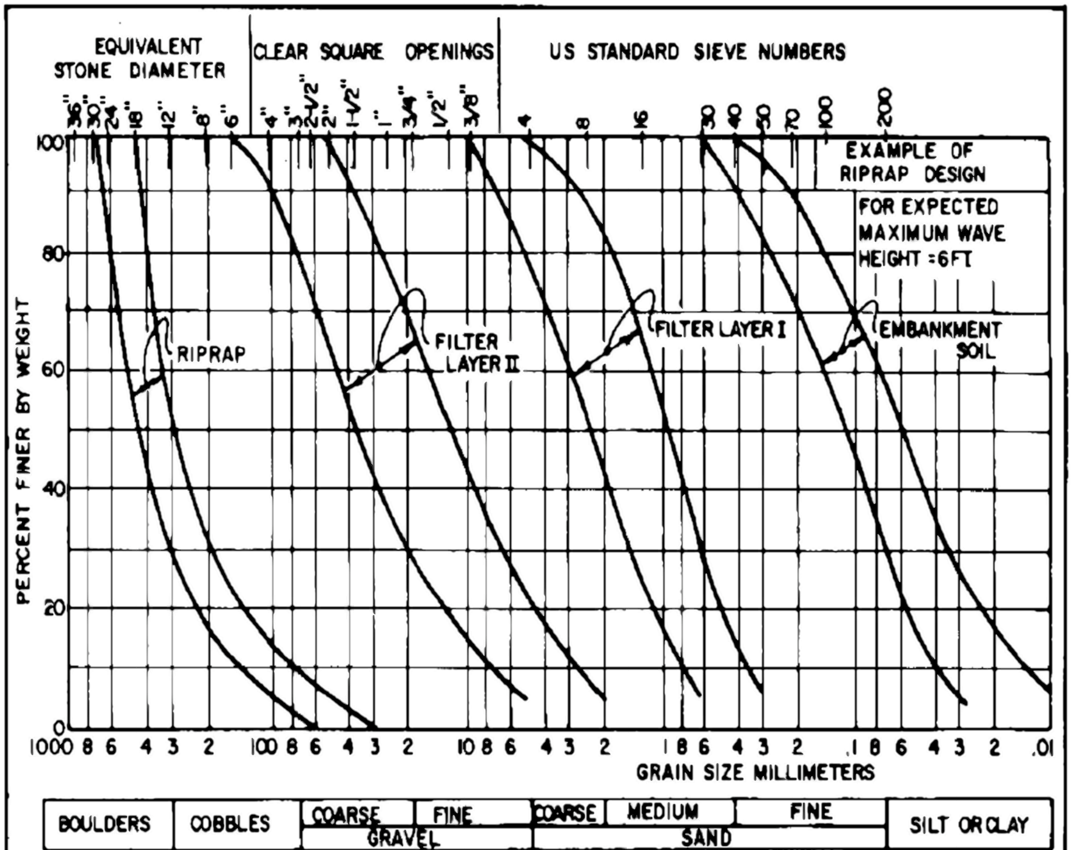
6.7.6. Riprap Protection

Frequently coarse rock is placed on embankments where erodible soils must be protected from fast currents and wave action. When coarse rock is used, currents and waves may wash soil out from under the rock and lead to undermining and failure. Soil loss under rock slopes can be prevented by the use of filter fabrics or by the placement of a filter layer of intermediate sized material between the soil and rock. In some cases soil loss can be prevented by the use of well-graded rock containing suitable fines that work to the bottom during placement.[160]

For determining rock sizes and filter requirements use Figure 6-16.

[160] For further guidance see Highway Research Board, Tentative Design Procedure for Rip-Rap - Lined Channels, National Cooperative Highway Research Program Report 108, Washington, D.C., 1970.

Figure 6-16 Design Criteria for Riprap and Filter on Earth Embankments[161]



GENERAL REQUIREMENTS:

1. FOR EMBANKMENT SLOPES BETWEEN 1:2 AND 1:4 DUMPED RIPRAP SHALL MEET THE FOLLOWING CRITERIA:

MAXIMUM WAVE HEIGHT FT.	AVERAGE ROCK SIZE D_{50}, IN.	MAXIMUM ROCK SIZE POUNDS	LAYER THICKNESS IN.
0 TO 1	8	100	12
1 TO 2	10	200	15
2 TO 4	12	500	18
4 TO 6	15	1500	24
6 TO 8	18	2500	30
8 TO 10	24	4000	36

2. RIPRAP SHALL BE WELL GRADED FROM A MAXIMUM SIZE AT LEAST 1.5 TIMES AVERAGE ROCK SIZE, TO 1 IN. SPALLS SUITABLE TO FILL VOIDS BETWEEN ROCKS.
3. RIPRAP BLANKET SHALL EXTEND TO AT LEAST 8 FT. BELOW LOWEST LOW WATER.
4. UNDER THE MOST EXTREME ICING AND TEMPERATURE CHANGES, ROCK SHOULD MEET SOUNDNESS AND DENSITY REQUIREMENTS FOR CONCRETE AGGREGATE. OTHERWISE, ANY UNWEATHERED ROCK WITH G > 2.60, OTHER THAN ARGILLACEOUS TYPES, ARE SUITABLE.
5. FILTER SHALL BE PROVIDED BETWEEN RIPRAP AND EMBANKMENT SOILS TO MEET THE FOLLOWING CRITERIA:

MAXIMUM WAVE HEIGHT, FT.	FILTER D_{85} SIZE AT LEAST:
0 TO 4	1 TO 1-1/2 IN.
4 TO 10	1-1/2 TO 2 IN.

NO FILTER IS NEEDED IF EMBANKMENT MEETS THE ABOVE REQUIREMENTS FOR D_{85} SIZE.

[161] Bureau of Reclamation, Design of Small Dams, U.S. Department of the Interior, U.S. Government Printing Office, 1973.

6.8. Laboratory Permeability Tests

6.8.1. Equipment

The apparatus used for permeability testing may vary considerably in detail depending primarily on the condition and character of the sample to be tested. Whether the sample is fine-grained or coarse-grained, undisturbed, remolded, or compacted, saturated or nonsaturated will influence the type of apparatus to be employed. The basic types of apparatus, grouped according to the type of specimen container (permeameter), are as follows:

(1) Permeameter cylinders

(2) Sampling tubes

(3) Pressure cylinders

(4) Consolidometers.

The permeability of remolded cohesionless soils is determined in permeameter cylinders, while the permeability of undisturbed cohesionless soils in a vertical direction can be determined using the sampling tube as a permeameter. The permeability of remolded cohesionless soils is generally used to approximate the permeability of undisturbed cohesionless soils in a horizontal direction. Pressure cylinders and consolidometers are used for fine-grained soils in the remolded, undisturbed, or compacted state. Fine-grained soils can be tested with the specimen oriented to obtain the permeability in either the vertical or horizontal direction. The above-listed devices are described in detail under the individual test procedures. Permeability tests utilizing the different types of apparatus, together with recommendations regarding their use, are discussed in the following paragraphs.

6.8.2. Constant-Head Permeability Test with Permeameter Cylinder

6.8.2.1. Use

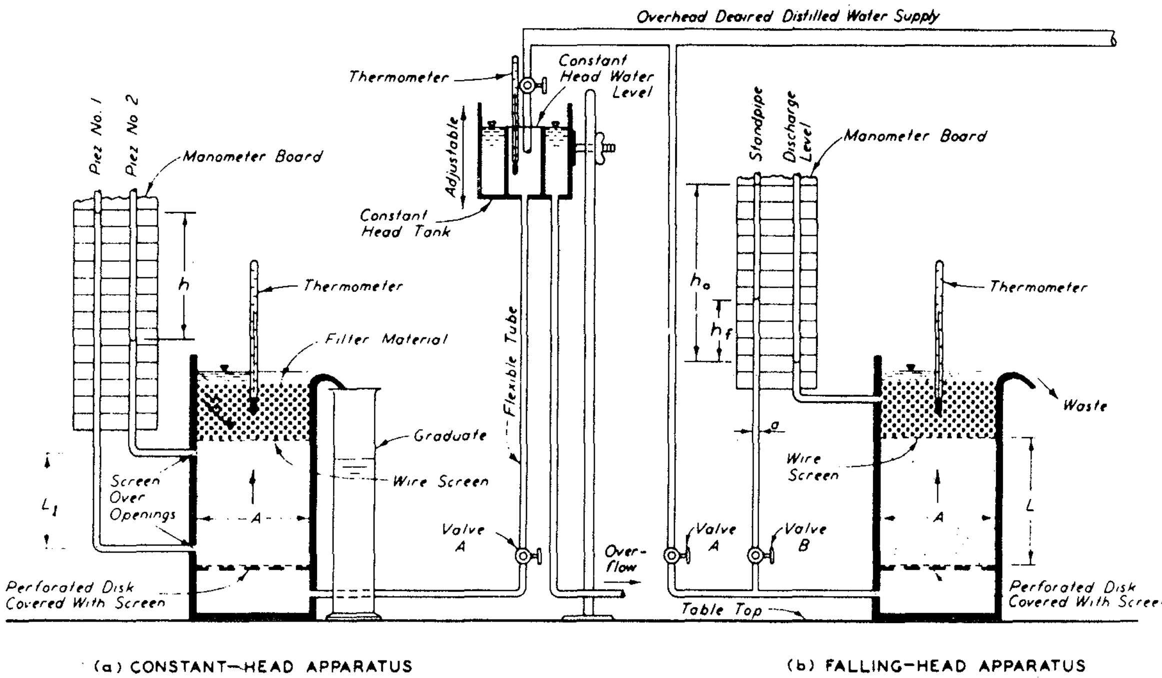
The constant-head permeability test with the permeameter cylinder shall in general be used for determining the permeability of remolded samples of coarse-grained soils such as clean sands and gravels having permeability greater than about 10 x 10^{-4} cm/sec.

6.8.2.2. Apparatus

The apparatus and accessory equipment should consist of the following:

(1) A permeameter cylinder similar to that shown schematically in Figure 6-17. The permeameter cylinder should be constructed of a transparent plastic material, The inside diameter of the cylinder should be not less than about 10 times the diameter of the largest soil particles, except when the specimen is encased in a rubber membrane as in the permeability test with pressure chamber, in which case the diameter of the cylinder should be at least six times the diameter of the largest soil particles. Piezometer taps along the side of the permeameter within limits to be occupied by the sample are advantageous in that the head loss within the sample is always measured across a fixed distance and rapid determination of hydraulic gradient can be made.

Figure 6-17 Schematic diagram of constant-head and falling -head permeability apparatus



(2) Perforated metal or plastic disks and circular wire screens, 35 to 400 mesh, cut for a close fit inside the permeameter.

(3) Glass tubing, rubber or plastic tubing, stoppers, screw clamps, etc., necessary to make connections as shown in Figure 6-17a.

(4) Filter materials such as Ottawa sand, coarse sand, and gravel of various gradations.

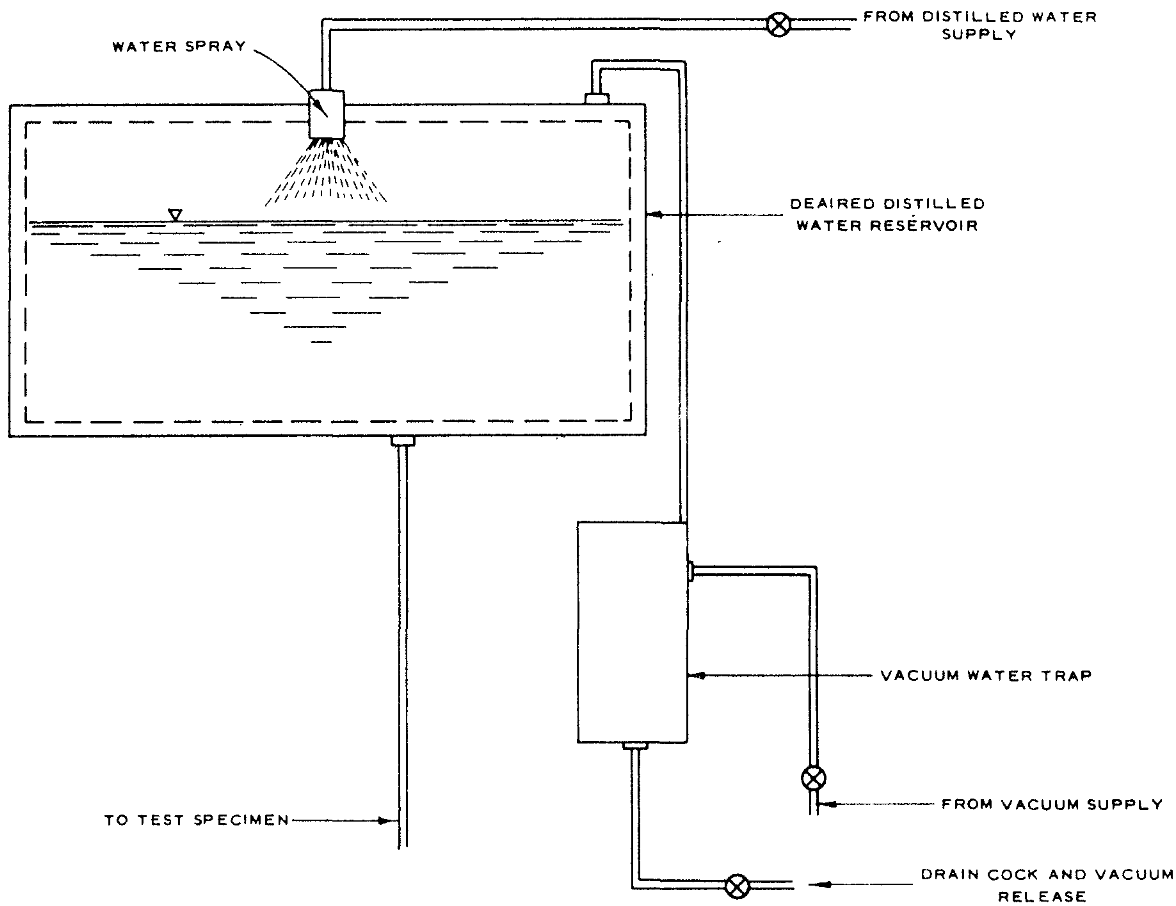
(5) A device for maintaining a constant-head water supply.

(6) Deaired distilled[162] water. Tap water contains dissolved air and gases that separate from solution in the initial layers of a test specimen of soil in the form of small bubbles. These bubbles reduce the permeability of the soil by decreasing the void space available for the flow of water. The most common method for removing dissolved air from water is by boiling the water and then cooling it at reduced pressures. This method is applicable only with small quantities of water. Freshly distilled water also has a very negligible amount of air. Large quantities of deaired distilled water may be prepared and retained for subsequent use by spraying distilled water in a fine stream into a container from which the air has been evacuated (see Figure 6-18). Permeability tests on saturated specimens should show no significant decrease in permeability with time if properly desired distilled water is used. However, if such a decrease in permeability occurs during a test, then a prefilter, consisting of a layer of the same material as the test specimen, should be used between the deaired distilled water reservoir and the test specimen to remove the air remaining in solution.[163]

[162] Demineralized water or tap water when it is known to be relatively free of minerals may be used in place of distilled water.

[163] G. E. Bertram, An Experimental Investigation of Protective Filters, Soil Mechanics Series No. 7, Harvard University (Cambridge, Mass., January 1940, reprinted May 1959).

Figure 6-18 Schematic diagram of apparatus for preparing deaired distilled water



(7) Manometer board with tubing leading from the piezometer taps. If piezometer taps are not provided, equipment to measure the distance between the constant-head source and tail water is required.

(8) Timing device, a watch or clock with second hand.

(9) Graduated cylinder, 100 ml capacity.

(10) Centigrade thermometer, range 0 to 50º C, accurate to 0.1º C.

(11) Balance, sensitive to 0.1 g.

(12) Oven (see 3.4.1.1).

(13) Scale, graduated in centimetres.

6.8.2.3. Placement and Saturation of Specimen

Placement and saturation of the specimen shall be done in the following steps:

(1) Record all identifying information for the specimen such as project, boring number, sample number, or other pertinent data, on a data sheet (Figure 11-20 is a suggested form).

(2) Oven-dry the specimen. Allow it to cool and weigh to the nearest 0.1 g. Record the oven-dry weight of material on the data sheet opposite W_s. The amount of material should be sufficient to provide a

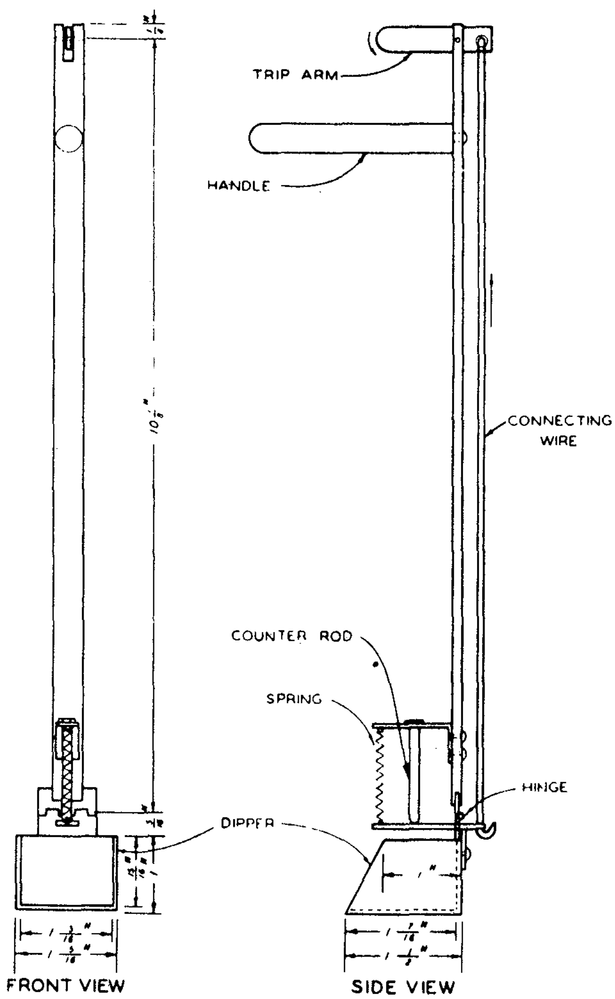
specimen in the permeameter having a minimum length of about one to two times the diameter of the specimen.

(3) Place a wire screen with openings small enough to retain the specimen over a perforated disk near the bottom of the permeameter above the inlet. The screen openings should be approximately equal to the 10% size of the specimen.

(4) Allow deaired distilled water to enter the water inlet of the permeameter to a height of about _" above the bottom of the screen, taking care that no air bubbles are trapped under the screen.

(5) Mix the material thoroughly and place in the permeameter to avoid segregation. The material should be dropped just at the water surface, keeping the water surface about 1/2" above the top of the soil during placement. A funnel or a special spoon as shown in Figure 6-19 is convenient for this purpose.

Figure 6-19 Spoon for placing Cohesionless Soils



(6) The placement procedure outlined above will result in a saturated specimen of uniform density although in a relatively loose condition. To produce a higher density in the specimen, the sides of the permeameter containing the soil sample are tapped uniformly along its circumference and length with a rubber mallet to produce an increase in density; however, extreme caution should be exercised so that fines are not put into suspension and segregated within the sample. As an alternative to this

procedure, the specimen may be placed in the in the dry using a funnel or spoon which permits the material to fall a constant height. The desired density may be achieved by vibrating the specimen to obtain a specimen of predetermined height. Compacting the specimen in layers is not recommended as a film of dust may be formed at the surface of the compacted layer which might affect the permeability results, After placement, apply a vacuum to the top of the specimen and permit water to enter the evacuated specimen through the base of the permeameter.

(7) After the specimen has been placed, weigh the excess material, if any, and the container. The specimen weight is the difference between the original weight of sample and the weight of the excess material. Care must be taken so that no material is lost during placement of the specimen. If there is evidence that material has been lost, oven-dry the specimen and weigh after the test as a check.

(8) Level the top of the specimen, cover with a wire screen similar to that used at the base, and fill the remainder of the permeameter with a filter material.

(9) Measure the length of the specimen and inside diameter of the permeameter to the nearest 0.3 cm and record on the data sheet as initial height and diameter of specimen.

(10) Test the specimen at the estimated natural void ratio or at a series of different void ratios, produced by increasing the amount of vibration after each permeability determination.

(11) Measure and record the length (height) of specimen in the permeameter prior to each determination. Permeability determinations at three different void ratios are usually sufficient to establish the relation of void ratio to permeability.

6.8.2.4. Procedure

The procedure shall consist of the following steps:

(1) Measure the distance, L_1, between the centres of the piezometer taps to the nearest 0.01 cm and record on the data sheet.

(2) Adjust the height of the constant-head tank to obtain the desired hydraulic gradient. The hydraulic gradient should be selected so that the flow through the specimen is laminar. The range of laminar flow conditions can be determined by plotting discharge versus hydraulic gradient. A straight-line relation indicates laminar flow, while deviations from the straight-line at high gradients indicate turbulent flow. Laminar flow for fine sands is limited to hydraulic gradients less than approximately 0.3. It is usually not practicable to achieve laminar flow for coarser soils, and the tests generally should be run at the hydraulic gradient anticipated in the field.

(3) Open valve A (see Figure 6-17a) and record the initial piezometer readings after the flow has become stable. Exercise care in building up heads in the permeameter so that the specimen is not disturbed.

(4) After allowing a few minutes for equilibrium conditions to be reached, measure by means of a graduate the quantity of discharge corresponding to a given time interval. Measure the piezometric heads and the water temperature in the permeameter.

(5) Record the quantity of flow, piezometer readings, water temperature, and the time interval during which the quantity of flow was measured on the data sheet, Figure 11-20.

(6) Repeat steps (4) and (5) several times over a period of about 1 hour, and compute the coefficient of permeability corresponding to each set of measured data. If there is no substantial change in the permeability, then the computed permeability is probably reliable. If there is a slight decrease in the permeability, then the permeability computed from the initial measurements, rather than the average, should be reported, so long as a plot of permeability versus time shows that the initial measurements are consistent with the subsequent measurements; a difference in permeability may result from a change in density caused by inadvertent jarring of the specimen in the permeameter. If there is any

substantial decrease of the permeability with time, a pre-filter should be used between the water reservoir and the permeameter. The criterion for judging whether a change in the computed permeability is "substantial" depends on the desired accuracy of the coefficient of permeability.

(7) If desired, reduce the void ratio as previously described and repeat the constant-head test.

6.8.2.5. Computation

The computations consist of the following steps:

(1) Compute the test void ratios in accordance with 3.4.2, The specific gravity shall be estimated or determined in accordance with 3.4.3.

(2) Compute the coefficient of permeability, k, by means of the following equation:

Equation 6-7: $$k_{20} = \frac{QLR_T}{hAt}$$

Where

- k_{20} = coefficient of permeability, cm/sec at 20º C
- Q= quantity of flow, cm^3
- L = length of specimen over which head loss is measured, cm. If piezometer taps are used, L = L_1 = distance between piezometer taps, cm
- R_T = temperature correction factor for viscosity of water obtained from Table 6-5.
- h = loss of head in length, L, or difference in piezometer readings = h_1 - h_2, cm
- A = cross-sectional area of specimen, cm^2
- t = elapsed time, sec.

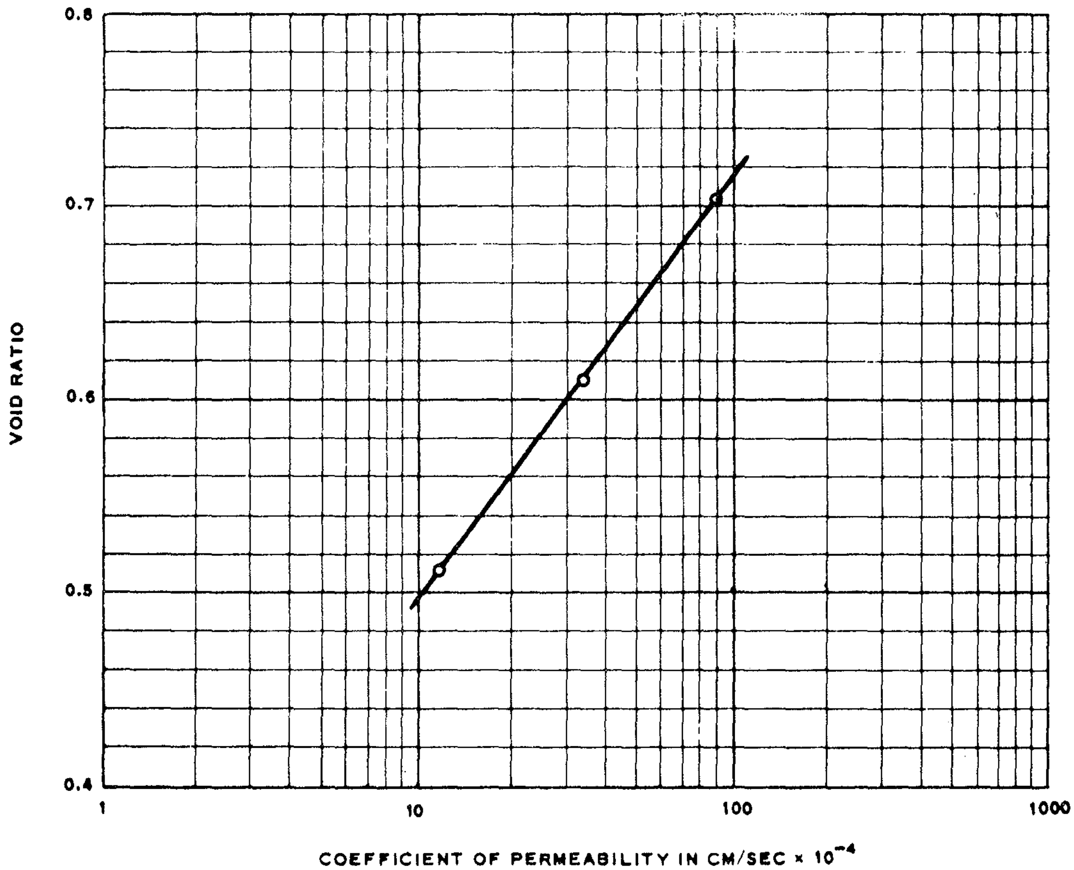
Table 6-5 Correction Factor R_T for Viscosity of Water at Various Temperatures

Temperature	Tenths of Degrees									
Degrees C	0	1	2	3	4	5	6	7	8	9
0.0	1.783	1.777	1.771	1.765	1.759	1.753	1.747	1.741	1.735	1.729
1.0	1.723	1.717	1.711	1.705	1.699	1.694	1.688	1.682	1.676	1.670
2.0	1.664	1.659	1.654	1.648	1.643	1.638	1.632	1.627	1.622	1.616
3.0	1.611	1.606	1.601	1.596	1.590	1.585	1.580	1.575	1.570	1.565
4.0	1.560	1.555	1.550	1.545	1.540	1.535	1.531	1.526	1.521	1.516
5.0	1.511	1.507	1.502	1.498	1.493	1.488	1.484	1.479	1.475	1.470
6.0	1.465	1.461	1.457	1.452	1.448	1.443	1.439	1.435	1.430	1.426
7.0	1.421	1.417	1.413	1.409	1.404	1.400	1.396	1.392	1.388	1.383
8.0	1.379	1.375	1.371	1.367	1.363	1.359	1.355	1.351	1.347	1.343
9.0	1.339	1.336	1.332	1.328	1.324	1.320	1.317	1.313	1.309	1.305
10.0	1.301	1.298	1.294	1.290	1.287	1.283	1.279	1.276	1.272	1.269
11.0	1.265	1.262	1.258	1.255	1.251	1.248	1.244	1.241	1.237	1.234
12.0	1.230	1.227	1.223	1.220	1.217	1.213	1.210	1.207	1.203	1.200
13.0	1.197	1.194	1.190	1.187	1.184	1.181	1.178	1.175	1.171	1.168
14.0	1.165	1.162	1.159	1.156	1.153	1.150	1.147	1.144	1.141	1.138
15.0	1.135	1.132	1.129	1.126	1.123	1.120	1.117	1.114	1.111	1.108
16.0	1.106	1.103	1.100	1.097	1.094	1.091	1.089	1.086	1.083	1.080
17.0	1.077	1.075	1.072	1.069	1.067	1.064	1.061	1.059	1.056	1.053
18.0	1.051	1.048	1.045	1.043	1.040	1.038	1.035	1.033	1.030	1.027
19.0	1.025	1.022	1.020	1.017	1.015	1.012	1.010	1.007	1.005	1.002
20.0	1.000	0.998	0.995	0.993	0.990	0.988	0.986	0.983	0.981	0.979
21.0	0.976	0.974	0.972	0.969	0.967	0.965	0.962	0.960	0.958	0.955
22.0	0.953	0.951	0.949	0.947	0.944	0.942	0.940	0.938	0.936	0.933
23.0	0.931	0.929	0.927	0.925	0.923	0.920	0.918	0.916	0.914	0.912
24.0	0.910	0.908	0.906	0.904	0.901	0.899	0.897	0.895	0.893	0.891
25.0	0.889	0.887	0.885	0.883	0.881	0.879	0.877	0.875	0.873	0.871
26.0	0.869	0.867	0.866	0.864	0.862	0.860	0.858	0.856	0.854	0.852
27.0	0.850	0.848	0.847	0.845	0.843	0.841	0.839	0.837	0.836	0.834
28.0	0.832	0.830	0.828	0.826	0.825	0.823	0.821	0.819	0.818	0.816
29.0	0.814	0.812	0.810	0.809	0.807	0.805	0.804	0.802	0.800	0.798
30.0	0.797	0.795	0.793	0.792	0.790	0.788	0.787	0.785	0.783	0.782
31.0	0.780	0.778	0.777	0.775	0.774	0.772	0.770	0.769	0.767	0.766
32.0	0.764	0.763	0.761	0.759	0.758	0.756	0.755	0.753	0.752	0.750
33.0	0.749	0.747	0.746	0.744	0.743	0.741	0.739	0.738	0.736	0.735
34.0	0.733	0.732	0.731	0.729	0.728	0.726	0.725	0.723	0.722	0.720
35.0	0.719	0.718	0.716	0.715	0.713	0.712	0.711	0.709	0.708	0.706
36.0	0.705	0.704	0.702	0.701	0.699	0.698	0.697	0.695	0.694	0.693
37.0	0.691	0.690	0.689	0.687	0.686	0.685	0.683	0.682	0.681	0.679
38.0	0.678	0.677	0.675	0.674	0.673	0.672	0.670	0.669	0.668	0.666
39.0	0.665	0.664	0.663	0.661	0.660	0.659	0.658	0.656	0.655	0.654
40.0	0.653	0.652	0.650	0.649	0.648	0.647	0.646	0.644	0.643	0.642
41.0	0.641	0.639	0.638	0.637	0.636	0.635	0.634	0.632	0.631	0.630
42.0	0.629	0.628	0.627	0.626	0.624	0.623	0.622	0.621	0.620	0.619
43.0	0.618	0.616	0.615	0.614	0.613	0.612	0.611	0.610	0.609	0.608
44.0	0.607	0.606	0.604	0.603	0.602	0.601	0.600	0.599	0.598	0.597
45.0	0.596	0.595	0.594	0.593	0.592	0.591	0.590	0.588	0.587	0.586
46.0	0.585	0.584	0.583	0.582	0.581	0.580	0.579	0.578	0.577	0.576
47.0	0.575	0.574	0.573	0.572	0.571	0.570	0.569	0.568	0.567	0.566
48.0	0.565	0.564	0.564	0.563	0.562	0.561	0.560	0.559	0.558	0.557
49.0	0.556	0.555	0.554	0.553	0.552	0.551	0.550	0.549	0.548	0.548

6.8.2.6. Presentation of Results

The coefficient of permeability shall be reported in units with coefficients of 1.0, 1 x 10^{-4}, and 1 x 10^{-9} cm/sec. The void ratio of the specimen shall be reported with all values of k. The coefficient of permeability, k, is logarithmically dependent upon the void ratio of the soil. Where k is determined at several void ratios, the test results shall be presented on a semilogarithmic chart as shown in Figure 6-20 in which k is plotted on the abscissa (logarithmic scale) and the void ratio is plotted on the ordinate (arithmetic scale.)

Figure 6-20 Relation between permeability and void ratio for cohesionless soils

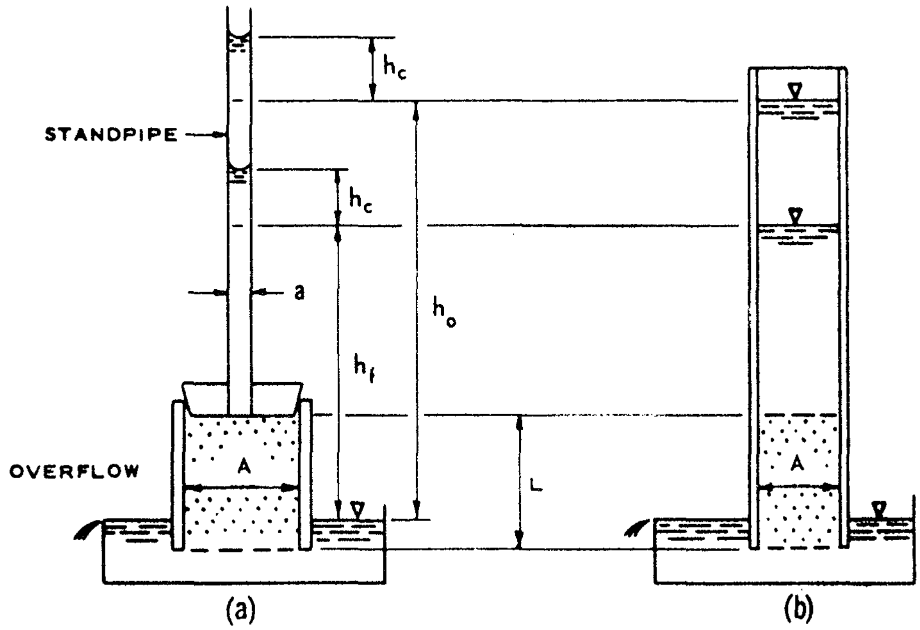


6.8.3. Falling-Head Permeability Test with Permeameter Cylinder

6.8.3.1. Use

The falling-head test with the permeameter cylinder should in general be used for determining the permeability of remolded samples of fine-grained soils having permeability less than about 10 x 10^{-4} cm/sec. The principle of this test is shown in Figure 6-21.

Figure 6-21 Principle of Falling Head Test



USING SETUP SHOWN IN (a), THE COEFFICIENT OF PERMEABILITY IS DETERMINED AS FOLLOWS:

$$k = \frac{La}{At} \ln \frac{h_o}{h_f} = 2.303 \frac{La}{At} \log_{10} \frac{h_o}{h_f}$$

USING SETUP SHOWN IN (b), THE COEFFICIENT OF PERMEABILITY IS DETERMINED AS FOLLOWS:

$$k = \frac{L}{t} \ln \frac{h_o}{h_f} = 2.303 \frac{L}{t} \log_{10} \frac{h_o}{h_f}$$

WHERE: h_c = HEIGHT OF CAPILLARY RISE
a = INSIDE AREA OF STANDPIPE
A = CROSS-SECTIONAL AREA OF SPECIMEN
L = LENGTH OF SPECIMEN
h_o = HEIGHT OF WATER IN STANDPIPE ABOVE DISCHARGE LEVEL MINUS h_c AT TIME, t_o
h_f = HEIGHT OF WATER IN STANDPIPE ABOVE DISCHARGE LEVEL MINUS h_c AT TIME, t_f
t = ELAPSED TIME, $t_f - t_o$

6.8.3.2. Apparatus

The apparatus and accessory equipment should consist of the following:

(1) A permeameter cylinder similar to that shown schematically in Figure 6-17b, or modified versions thereof. The permeameter cylinder should be constructed of a transparent plastic material. The

inside diameter of the cylinder should be not less than about 10 times the diameter of the largest soil particles. The use of two piezometer taps, as shown by Figure 6-17b, connected to a standpipe and discharge level tube eliminates the necessity for taking into account the height of capillary rise which would be necessary in the case of a single standpipe of small size. The height of capillary rise for a given tube and condition can be measured simply by standing the tube upright in a beaker full of water. The size of standpipe to be used is generally based on experience with the equipment used and soils tested. In order to accelerate testing, air pressure may be applied to the standpipe to increase the hydraulic gradient.

(2) Perforated metal or plastic disks and circular wire screens, 35 to 100 mesh, cut for a close fit inside the permeameter.

(3) Glass tubing, rubber or plastic tubing, stoppers, screw clamps, etc., necessary to make connections as shown in Figure 6-17b.

(4) Filter materials such as Ottawa sand, coarse sand, and gravel of various gradations.

(5) Deaired distilled water.

(6) Manometer board or suitable scales for measuring levels in piezometers or standpipe.

(7) Timing device, a watch or clock with second hand.

(8) Centigrade thermometer, range 0 to 50º C, accurate to 0.1º C.

(9) Balance, sensitive to 0.1 g.

(10) Oven (see 3.4.1.1).

(11) Scale, graduated in centimeters.

6.8.3.3. Placement and Saturation of Specimen

Placement and saturation of the specimen shall be done as described in paragraph 3c. Identifying information for the sample and test data shall be entered on a data sheet similar to Figure 11-21.

6.8.3.4. Procedure

The procedure shall consist of the following steps:

(1) Measure and record the height of the specimen, L, and the cross-sectional area of the specimen, A.

(2) With valve B open (see Figure 6-17b), crack valve A and slowly bring the water level up to the discharge level of the permeameter.

(3) Raise the head of water in the standpipe above the discharge level of the permeameter. The difference in head should not result in an excessively high hydraulic gradient during the test. Close valves A and B.

(4) Begin the test by opening valve B. Start the timer. As the water flows through the specimen, measure and record the height of water in the standpipe above the discharge level, h_o, in centimeters, at time t_o, and the height of water above the discharge level, h_f, in centimeters, at time t_f.

(5) Observe and record the temperature of the water in the permeameter.

(6) Repeat the determination of permeability, and if the computed values differ by an appreciable amount, repeat the test until consistent values of permeability are obtained.

6.8.3.5. Computation

The computations consist of the following steps:

(1) Compute the test void ratios.

(2) Compute the coefficient of permeability, k, by means of the following equation[164]:

$$\textbf{Equation 6-8:}\ k = \frac{aL}{At}\left(\ln\frac{h_o}{h_f}\right)R_T$$

Where

- a = inside area of standpipe, cm^2
- A = cross-sectional area of specimen, cm^2
- L = length of specimen, cm
- t = elapsed time (t_f - t_o), sec
- h_o = height of water in standpipe above discharge level at time t_o, cm
- h_f = height of water in standpipe above discharge level at time t_f, cm
- R_T = temperature correction factor for viscosity of water obtained from Table 6-5, degrees C.

If a single standpipe of small diameter is used as shown in Figure 6-17, the height of capillary rise, h_c, should be subtracted from the standpipe readings to obtain h_o and h_f.

6.8.3.6. Presentation of Results

The results of the falling-head permeability test shall be reported as described in paragraph 3f.

6.8.4. Permeability Tests with Sampling Tubes

Permeability tests may be performed directly on undisturbed samples without removing them from the sampling tubes. The sampling tube serves as the permeameter cylinder. The method is applicable primarily to cohesionless soils that cannot be removed from the sampling tube without excessive disturbance. The permeability obtained is in the direction in which the sample was taken, i.e., generally vertical. The permeability obtained in a vertical direction may be substantially less than that obtained in a horizontal direction.

Permeability tests with sampling tubes may be performed under constant-head or falling-head conditions of flow, depending on the estimated permeability of the sample. The equipment should be capable of reproducing the conditions of flow in the constant head or falling-head tests. It is important that all disturbed material or material containing drilling mud he removed from the top and bottom of the sample. Screens held in place by perforated packers should protect the ends of the sample. The test procedure and computations are the same as those described previously for each test.

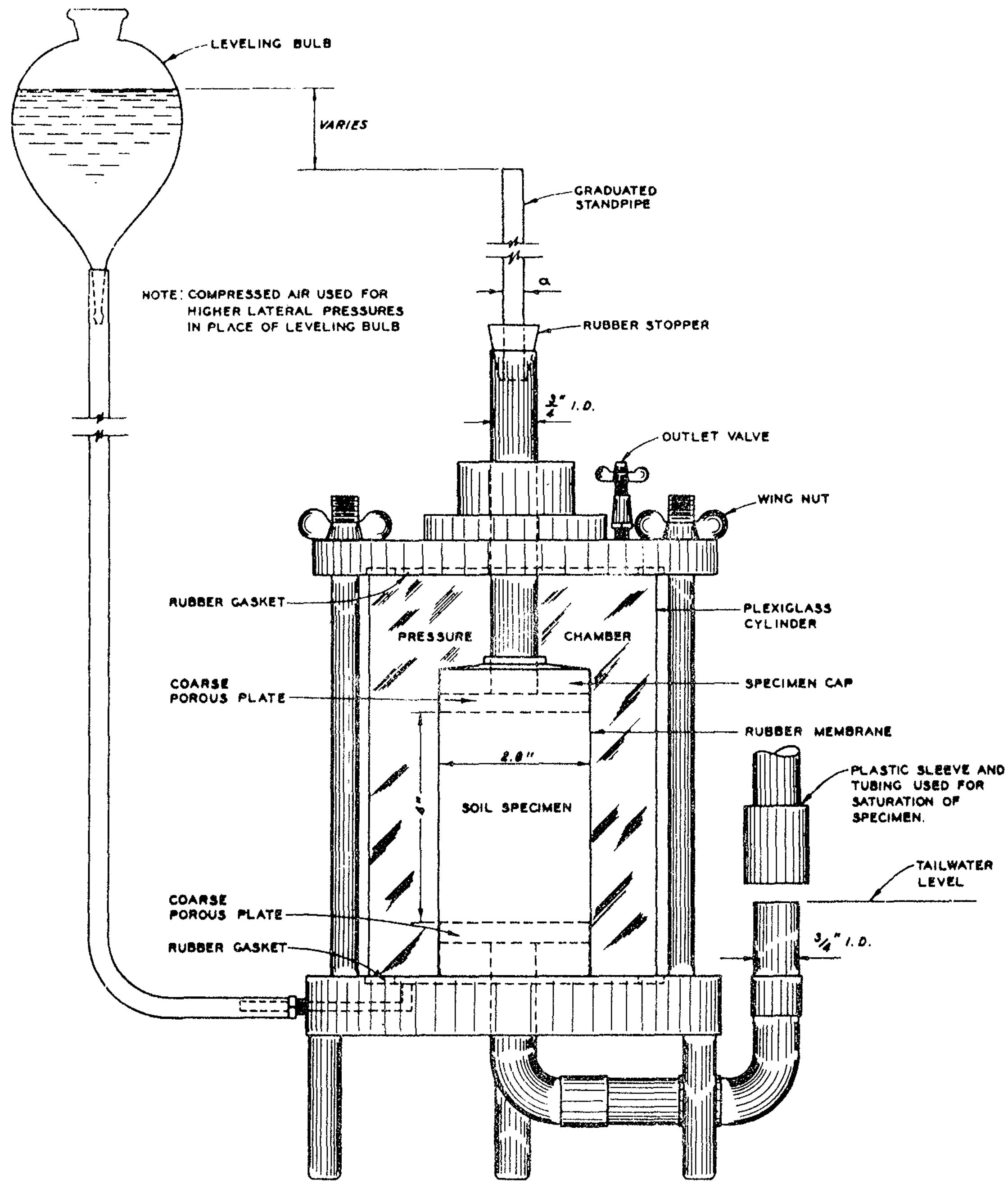
6.8.5. Permeability Test with Pressure Chamber

In the permeability test with a pressure chamber, see Figure 6-22, a cylindrical specimen is confined in a rubber membrane and subjected to an external hydrostatic pressure during the permeability test. The advantages of this type of test are: (a) leakage along the sides of the specimen, which would occur if the specimen were tested in a permeameter, is prevented, and (b) the specimen can be tested under conditions

[164] Equation 6-8 has traditionally been stated using common logarithms and a constant factor. This was useful when it was necessary to look up logarithms in tables. In view of the wide availability of calculators and computer programs, the use of natural logarithms – which makes possible the elimination of the constant – seems a more sensible approach.

of 1oading expected in the field. The test is applicable primarily to cohesive soils in the undisturbed, remolded or compacted state. Complete saturation of the specimen, if it is not fully saturated initially, is practically impossible. Consequently, this test should be used only for soils that are fully saturated, unless values of permeability are purposely desired for soils in an unsaturated condition. The permeability test with the pressure chamber is usually performed as a falling-head test.

Figure 6-22 Pressure chamber for permeability test



The permeability specimens for use in the pressure chamber generally should be 2.8” in diameter, as rubber membranes and equipment for cutting and trimming specimens of this size are available for triaxial testing apparatus. A specimen length of about 4" is adequate. (The dimensions of a test specimen may be varied if equipment and supplies are available to make a suitable test set-up.) The pressure in the chamber should not be less than the maximum head on the specimen during the test. The other test procedure and computations are the same as those described for the falling-head test. The linear relation between permeability and void ratio on a semilogarithmic plot as shown in Figure 6-20 is usually not applicable to fine-grained soils, particularly when compacted. Other methods of presenting permeability-void ratio data may be desirable.

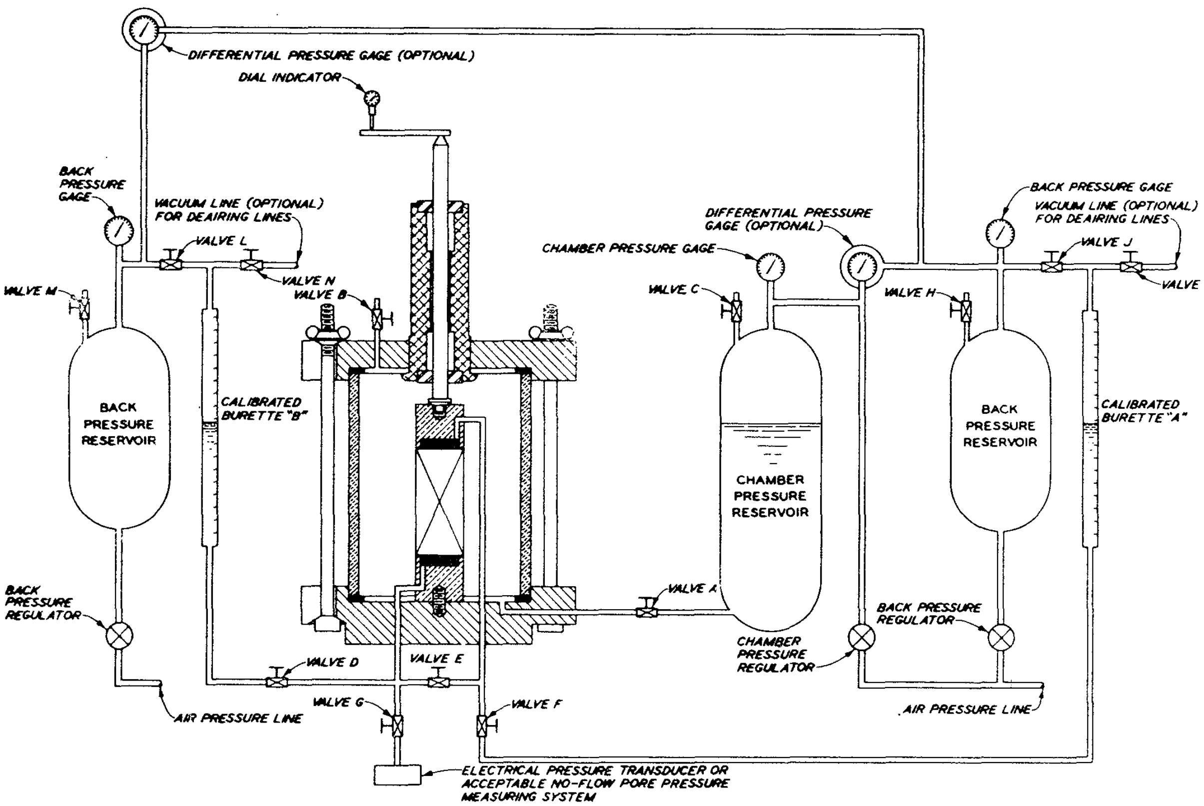
6.8.6. Permeability Tests with Back Pressure

6.8.6.1. Description

Gas bubbles in the pores of a compacted or undisturbed specimen of fine-grained soil will invalidate the results of the permeability tests described in the preceding paragraphs. It is known that an increase in pressure will cause a reduction in volume of gas bubbles and an increased weight of gas dissolved in water. To each degree of saturation, there corresponds a certain additional pressure (back pressure) that, if applied to the pore fluid of the specimen, will cause complete saturation. The permeability test with back pressure is performed in a pressure chamber such as that shown in Figure 6-23, utilizing equipment that permits increasing the chamber pressure and pore pressure simultaneously, maintaining their difference constant The method is generally applicable to fine-grained soils that are not fully saturated.[165]

[165] Apparatus and procedures have been described by A. Casagrande "Third Progress Report on Investigation of Stress Deformation and Strength Characteristics of Compacted Clays," Soil Mechanics Series No. 70, Nov 1963, Harvard University, Cambridge, MA, pp 30 and 31, and L. Bjerrum and J. Huder, "Measurement of the Permeability of Compacted Clays," Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 1, Aug 1957, pp 6-8.

Figure 6-23 Schematic diagram of falling-head device for permeability tests with backpressure



6.8.6.2. Procedure

See Figure 6-23. The procedure shall, consist of the following steps:

(1) After having determined the dimensions and wet weight of the test specimen, place it in the triaxial apparatus, using the same procedure as for setting up a specimen for an R triaxial test with pore pressure measurements except that filter strips should not be used.

(2) Saturate the specimen and verify 100% saturation. Burette "A" is utilized during this operation.

(3) With the drainage valves closed, increase the chamber pressure to attain the desired effective consolidation pressure (chamber pressure minus back pressure). At zero elapsed time, open valves E and F.

(4) Record time, dial indicator reading, and burette reading at elapsed times of 0, 15, and 30 sec, 1, 2, 4, 8, and 15 minutes, and 1, 2, 4, and 8 hours, etc. Plot the dial indicator readings and burette readings on an arithmetic scale versus elapsed time on a log scale. When the consolidation curves indicate that primary consolidation is complete close valves E and F.

(5) Apply a pressure to burette B greater than that in burette A. The difference between the pressures in burettes B and A is equal to the head loss h; h divided by the height of the specimen after consolidation, L, is the hydraulic gradient. The difference between the two pressures should be kept as small as practicable, consistent with the requirement that the rate of flow be large enough to make accurate measurements of the quantity of flow within a reasonable period. Because the difference in the two pressures may be very small in comparison to the pressures at the ends of the specimen, and because the head loss must be maintained constant throughout the test, the difference between the pressures within the burettes must be measured accurately; a differential pressure gage is very useful

for this purpose. The difference between the elevations of the water within the burettes should also be considered (1" of water = 0.036 psi of pressure).

(6) Open valves D and F. Record the burette readings at any zero elapsed time. Make readings of burettes A and B and of temperature at various elapsed times (the interval between successive readings depends upon the permeability of the soil and the dimensions of the specimen). Plot arithmetically the change in readings of both burettes versus time. Continue making readings until the two curves become parallel and straight over a sufficient length of time to accurately determine the rate of flow (slope of the curves).

(7) If it is desired to determine the permeability at several void ratios, steps 3 through 6 can be repeated, using different consolidation pressures in step 3.

(8) At the end of the permeability determinations, close all drainage valves and reduce the chamber pressure to zero; disassemble the apparatus.

(9) Determine the wet and dry weights of the specimen.

6.8.6.3. Computations

The computations consist of the following steps.

(1) Compute the test void ratios.

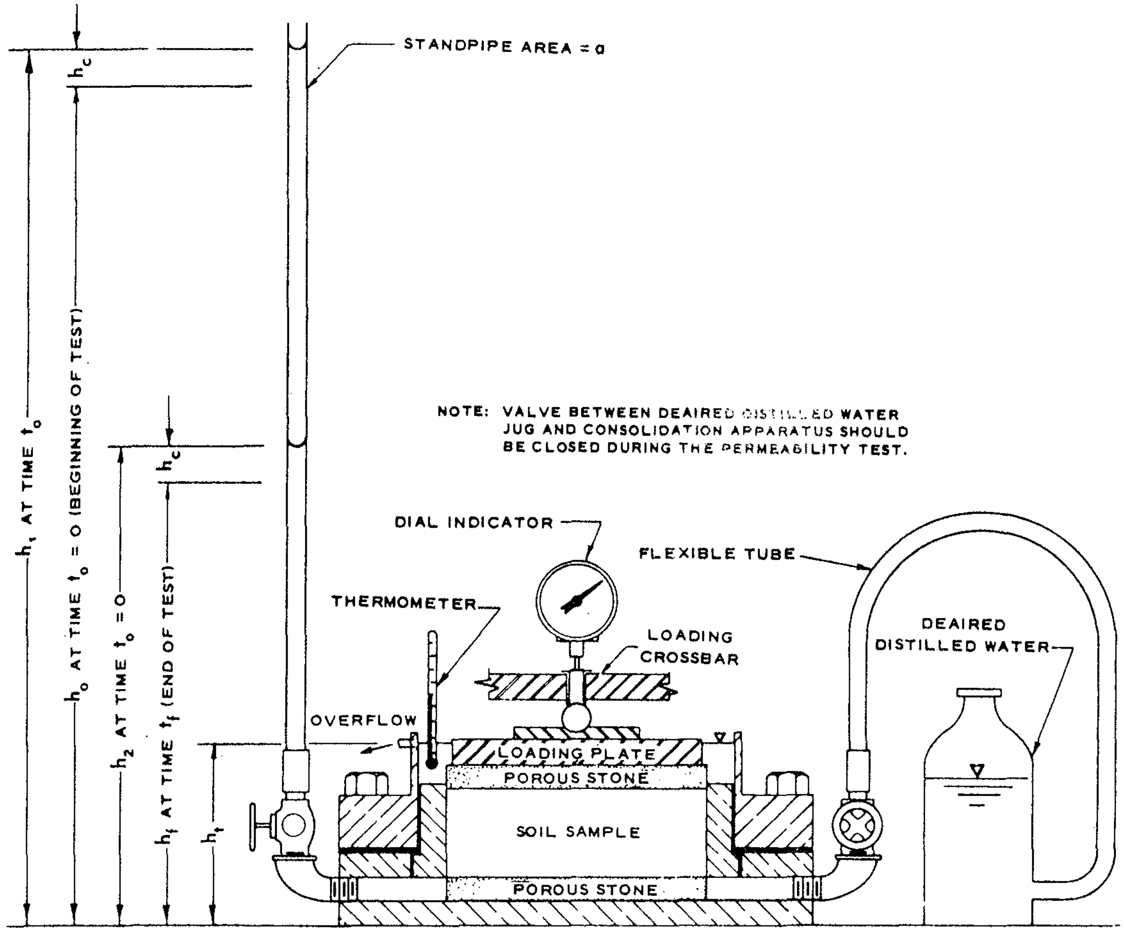
(2) Computations of coefficients of permeability are the same as those described for the constant-head permeability test.

6.8.7. Permeability Tests with Consolidometer

A permeability test in a consolidometer is essentially similar to that conducted in a pressure chamber, except that the specimen is placed within a relatively rigid ring and is loaded vertically. The test can be used as an alternate to the permeability test in the pressure chamber. The test is applicable primarily to cohesive soils in a fully saturated condition. Testing is usually performed under falling-head conditions.

A schematic diagram of the consolidation apparatus set up for a falling-head permeability test is shown in Figure 6-24. Identifying information for the specimen and subsequent test data are entered on a data sheet (Figure 11-22 is a suggested form). The specimen should be placed in the specimen ring and the apparatus assembled. The specimen is consolidated under the desired load and the falling-head test is performed as previously described.

Figure 6-24 Schematic Diagram of Falling Head Device for Permeability Test in Consolidometer



The net head on the specimen may be increased by use of air pressure; however, the pressure on the pore water should not exceed 25 to 30% of the vertical pressure under which the specimen has consolidated. Dial indicator readings are observed before and after consolidation to permit computation of void ratios. The determination of the coefficient of permeability may be made in conjunction with the consolidation test, in which case the test is performed at the end of the consolidation phase under each load increment. Computations are similar to those described for the falling-head test with the permeameter cylinder. The permeability may also be determined indirectly from computations using data obtained during the consolidation test; however, the assumptions on which the method is based are seldom satisfied, and consequently, the direct determination of permeability should be employed where reliable values of permeability are required.

6.8.8. Possible Errors

Following are possible errors that would cause inaccurate determinations of the coefficient of permeability:

a. Stratification or nonuniform compaction of cohesionless soils. If the specimen is compacted in layers, any accumulation of fines at the surface of the layers will reduce the measured coefficient of permeability.

b. Incomplete initial saturation of specimen.

c. Excessive hydraulic gradient. Darcy’s law is applicable only to conditions of laminar flow.

d. Air dissolved in water. No other source of error is as troublesome as the accumulation of air in the specimen from the flowing water. As water enters the specimen, small quantities of air dissolved in the water will tend to collect as fine bubbles at the soil-water interface and reduce the permeability at this interface with increasing time. (It should be noted that air accumulation would not affect the coefficient of permeability determined by the constant-head test if piezometer taps along the side of the specimen are used to measure the head loss.)

e. Leakage along side of specimen in permeameter. One major advantage to the use of the triaxial compression chamber for permeability tests is that a flexible membrane that is pressed tightly against the specimen by the chamber pressure confines the specimen.

§ 7. Analysis of Settlement and Volume Expansion

7.1. Introduction

7.1.1. Overview

Soil is a nonhomogeneous porous material consisting of three phases: solids, fluid (normally water), and air. Soil deformation may occur by change in stress, water content, soil mass, or temperature. Vertical displacements and settlement caused by change in stress and water content are described in this section.

Difficult soils can create serious problems in both settlement and volume expansion. These are discussed in detail in 2.5.

This section excludes settlement caused by the following:

- Subsidence and undermining by tunnels
- Subsidence due to buried karst features or cavities
- Thermal effects of structures on permafrost
- Effects of frost heave
- Loss in mass from erosion
- Loss of ground from rebound and lateral movement in adjacent excavations
- Loss of support caused by lateral soil movement from landslides, downhill creep, and shifting retaining walls.
- Horizontal deformation of structures associated with vertical deformations may also occur, but such analysis is complex and beyond the scope of this book.
- Deep foundations are driven piles and drilled shafts used to transmit foundation loads to deeper strata capable of supporting the applied loads.
- Settlements of domestic and hazardous landfills are unpredictable and cannot be readily estimated using techniques presented in this book.

Soil movements may be minimized by treating the soil prior to construction by numerous methods such as removal of poor soil and replace with suitable soil, precompression of soft soil, dynamic consolidation of cohesionless soil, and chemical stabilization or wetting of expansive or collapsible soil. Foundations may be designed to tolerate some differential movements. Remedial techniques such as underpinning with piles, grouting, and slabjacking are available to stabilize and repair damaged foundations.

7.1.2. General Considerations and Definitions

Placement of an embankment load or structure on the surface of a soil mass introduces stress in the soil that causes the soil to deform and leads to settlement of the structure. It is frequently necessary to estimate the differential and total vertical soil deformation caused by the applied loads. Differential movement affects the structural integrity and performance of the structure. Total deformation is significant relative to connections of utility lines to buildings, grade and drainage from structures, minimum height specifications of dams (i.e., freeboard), and railroad and highway embankments. Soils and conditions described in Table 2-12 require special considerations to achieve satisfactory design and performance. Early recognition of these problems is essential to allow sufficient time for an adequate field investigation and preparation of an appropriate design.

7.1.2.1. Approach Embankment Settlement

Approach embankment settlement is the most prevalent foundation problem in highway construction. Unlike stability problems, the results are seldom catastrophic but the cost of perpetual maintenance of continuing settlement is immense. The difficulty in preventing these problems is not as much a lack of technical expertise as it is a lack of communication between personnel involved in the roadway design and those involved in the structure design.

The design of a roadway embankment can utilize a wide range of soil materials and permit substantial amounts of settlement without affecting the performance of the highway. Roadway designers necessarily permit such materials to reduce project costs by utilizing cheap locally available soils. Structures are necessarily designed for little or no settlement to maintain specified highway clearances and to insure integrity of structural members. The approach embankment must affect a transition between roadway and structure while providing adequate structural foundation support. In most agencies, the responsibility for approach embankment design is not defined, which results in roadway criteria being used across the structure. This is wrong; the approach embankment requires special materials and placement criteria to prevent internal consolidation and to moderate external consolidation.

7.1.2.2. Sources of Stress

Sources of stress in soil occur from soil weight, surface loads, and environmental factors such as desiccation from drought, wetting from rainfall, and changes in depth to groundwater. Stress in soils is discussed in more detail in § 5.

7.1.2.2.1. Soil weight

Soil strata with different unit weights alter the stress distribution. Any change in total stress results in changes in effective stress and pore pressure. In a saturated soil, any sudden increase in applied total stress results in a corresponding pore pressure increase, Equation 5-1. This increase may cause a flow of water out of the soil deposit, a decrease in pore pressure, and an increase in effective stress. Changes in pore water pressure such as the raising or lowering of water tables also lead to a reduction or increase in effective stress.

7.1.2.2.2. Surface loads

Loads applied to the surface of the soil mass increase the stress within the mass. The pressure bulb concept, Figure 5-5, illustrates the change in vertical stress within the soil mass. Placement of a uniform pressure over a foundation with a minimum width much greater than the depth of the soil layer will cause an increase of vertical stress in the soil approximately equal to the applied pressure.

7.1.2.2.3. Applicability to settlement calculations

The ability to predict settlements using elastic theory depends much more strongly on the *in situ* nonlinearity and material inhomogeneity than errors in the distribution of stresses. These settlements directly depend on the assumed constitutive material law and on the magnitude of the required soil parameters. Refer to 3.3 for further information on elasticity theory.

7.1.2.2.4. Rules of thumb for static loads

Preliminary settlement analyses are sometimes performed before the structural engineer and architect are able to furnish the design load conditions.

a) Some rules of thumb for line and column loads for buildings described in Table 7-1 are based on a survey of some engineering firms. Tall multi-storey structures may have column loads exceeding

1000 tons. Column spacings are often 20 to 25 ft or more. The average pressure applied per story of a building often varies from 0.1 to 0.2 tsf.

b) Vertical pressures from embankments may be estimated from the unit wet weight times height of the fill.

c) Vertical pressures from locks, dams, and retaining walls may be estimated by dividing the structure into vertical sections of constant height and evaluating the unit weight times the height of each section.

Table 7-1 Some Typical Loads on Building Foundations

Structure	*Line Load, tons/ft*	*Column Load, tons*
Apartments	0.5 to 1	30
Individual housing	0.5 to 1	< 5
Warehouses	1 to 2	50
Retail Spaces	1 to 2	40
Two-story buildings	1 to 2	40
Multi-storey buildings	2 to 5	100
Schools	1 to 3	50
Administration buildings	1 to 3	50
Industrial facilities		50

7.2. Limitations of Settlement

7.2.1. General

Significant aspects of settlement from static and dynamic loads are total and differential settlement. Total settlement is the magnitude of downward movement. Differential settlement is the difference in vertical movement between various locations of the structure and distorts the structure. Conditions that cause settlement are described in Table 2-12. Limitations to total and differential settlement depend on the function and type of structure.

7.2.2. Total Settlement

Many structures can tolerate substantial downward movement or settlement without cracking, Table 7-2; however, total settlement should not exceed 2 inches for most facilities. A typical specification of total settlement for commercial buildings is 1 inch. Structures such as solid reinforced concrete foundations supporting smokestacks, silos, and towers can tolerate larger settlements up to 1 ft.

Table 7-2 Maximum Allowable Average Settlement of Some Structures[166]

Type of Structure	*Settlement, inches*
Plain brick walls, Length/Height ≥ 2.5	3

[166] Polshin, D. E. and Tokar, R. A. 1957. "Maximum Allowable Non-uniform Settlement of Structures," Proceedings Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol 1, pp 402-405. Available from Butterworths Publications, Ltd.,88 Kingsway, London, WC2, England.

Plain brick walls, Length/Height ≤ 1.5	4
Framed structure	4
Reinforced brick walls and brick walls with reinforced concrete	6
Solid reinforced concrete foundations supporting smokestacks, silos, towers, etc	12

Some limitations of total settlement are as follows:

7.2.2.1. Utilities

Total settlement of permanent facilities can harm or sever connections to outside utilities such as water, natural gas, and sewer lines. Water and sewer lines may leak contributing to localized wetting of the soil profile and aggravating differential displacement. Leaking gas from breaks caused by settlement can lead to explosions.

7.2.2.2. Drainage

Total settlement reduces or interferes with drainage of surface water from permanent facilities, contributes to wetting of the soil profile with additional differential movement, and may cause the facility to become temporarily inaccessible.

7.2.2.3. Serviceability

Relative movement between the facility and surrounding soil may interfere with serviceability of entryways.

7.2.2.4. Freeboard

Total settlement of embankments, levees and dams reduces freeboard and volume of water that may be retained. The potential for flooding is greater during periods of heavy rainfall. Such settlement also alters the grade of culverts placed under roadway embankments.

7.2.3. Differential Settlement

Differential settlement, which causes distortion and damages in structures, is a function of the uniformity of the soil, stiffness of the structure, stiffness of the soil, and distribution of loads within the structure. Limitations to differential settlement depend on the application. Differential settlements should not usually exceed 1/2 inch in buildings, otherwise cracking and structural damage may occur. Differential movements between monoliths of dams should not usually exceed 2 inches; otherwise leakage may become a problem. Embankments, dams, one or two story facilities, and multi-storey structures with flexible framing systems are sufficiently flexible such that their stiffness often need not be considered in settlement analysis. Pavements may be assumed to be completely flexible.

7.2.3.1. Types of Damages

Differential settlement may lead to tilting that can interfere with adjacent structures and disrupt the performance of machinery and people. Differential settlement can cause cracking in the structure, distorted and jammed doors and windows, uneven floors and stairways, and other damages to houses and buildings. Differential movement may lead to misalignment of monoliths and reduce the efficiency of water stops. Widespread cracking can impair the structural integrity and lead to collapse of the structure, particularly during earthquakes. The height of a wall for a building that can be constructed on a beam or foundation without cracking is related to the deflection/span length Δ/L ratio and the angular distortion β described below.

7.2.3.2. Deflection Ratio

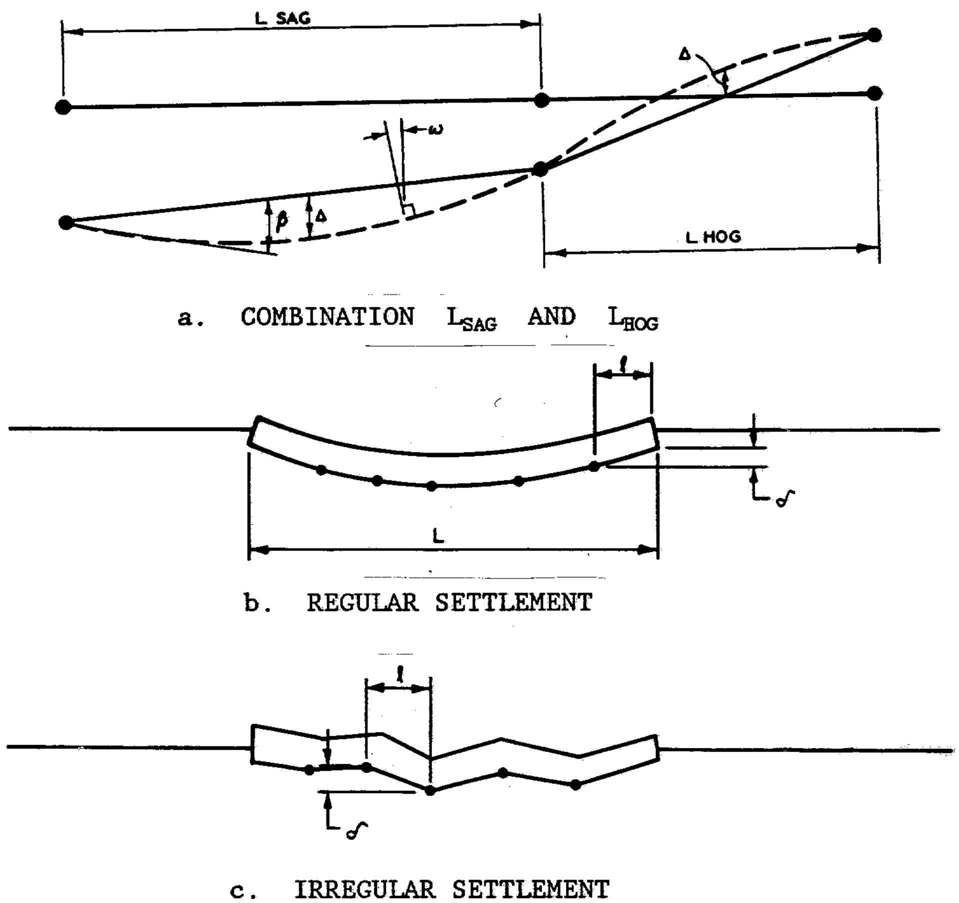
The deflection ratio Δ/L is a measure of the maximum differential movement Δ in the span length L, Figure 7-1. The span length may be between two adjacent columns, L_{SAG} or L_{HOG}, Figure 7-1a.

(1) Table 7-3 provides limiting observed deflection ratios for some buildings.

(2) Design Δ/L ratios are often greater than 1/600, but the stiffness contributed by the components of an assembled brick structure, for example, helps maintain actual differential displacement/span length ratios near those required for brick buildings, Table 7-3, to avoid cracking.

(3) Circular steel tanks can tolerate Δ/L ratios greater than 1/200 depending on the settlement shape[167].

Figure 7-1 Schematic illustration of angular distortion $\beta = \delta/\ell$ and deflection ratio Δ/L for settling (sagging) and heaving (hogging) profiles



[167] D'Orazio, T. B. and Duncan, J. M. 1987. "Differential Settlement in Steel Tanks," Journal of the geotechnical engineering Division, Vol 113, pp 967-983. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Table 7-3 Some Limiting Deflection Ratios[168]

Structure	Deflection Ratio, Δ/L, Sand and Hard Clay	Deflection Ratio, Δ/L, Plastic Clay
Buildings with plain brick walls, Length/Height ≥ 3	1/3333	1/2500
Buildings with plain brick walls, Length/Height ≥ 5	1/2000	1/1500
One story mills; between columns for brick clad column frames	1/1000	1/1000
Steel and concrete frame	1/500	1/500

7.2.3.3. Angular Distortion

Angular distortion $\beta = \delta/\ell$ is a measure of differential movement δ between two adjacent points separated by the distance, Figure 7-1.

7.2.3.3.1. Initiation of damage

Table 2-3 illustrates limits to angular distortion for various types of structures without cracking based on field surveys of damage.

a) A safe limit for no cracking in properly designed and constructed steel or reinforced concrete frame structures is angular distortion β = 1/500. Cracking should be anticipated when β exceeds 1/300. Considerable cracking in panels and brick walls and structural damage is expected when β is less than 1/150.

b) Tilting can be observed if $\omega > 1/250$ and must be limited to clear adjacent buildings, particularly in high winds. The angle of tilt is indicated by ω , Figure 7-1a.

c) Slower rates of settlement increase the ability of structures to resist cracking.

d) Unreinforced concrete masonry unit construction is notably brittle and cracks at relatively low angular distortion values as shown in Table 7-4.

Such structures must be properly detailed and constructed to provide acceptable service at sites with even moderate differential movement potential.

Consideration should be given to using a less crack-susceptible material at expansive soil sites and any other site having a significant differential movement potential.

Table 7-4 Limiting Angular Distortions to Avoid Potential Damages

Situation	Length/Height	Allowable Angular Distortion, $\beta = \delta/\ell$
Hogging of unreinforced load-bearing walls		1/2000

[168] Feld, J. 1965. "Tolerance of Structures to Settlement," Journal of the Soil Mechanics and Foundations Division, Vol 91, pp 63-77. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Load bearing brick, tile, or concrete block walls	≥ 5	1/1250
	≤ 3	1/2500
Sagging of unreinforced load-bearing walls		1/1000
Machinery sensitive to settlement		1/750
Frames with diagonals		1/600
No cracking in buildings; tilt of bridge abutments; tall slender structures such as stacks, silos, and water tanks on a rigid mat		1/500
Steel or reinforced concrete frame with brick, block, plaster or stucco finish	≥ 5	1/500
	≤ 3	1/1000
Circular steel tanks on flexible base with floating top; steel or reinforced concrete frames with insensitive finish such as dry wall, glass, panels		1/300 - 1/500
Cracking in panel walls; problems with overhead cranes		1/300
Tilting of high rigid buildings		1/250
Structural damage in buildings; flexible brick walls with length/height ratio > 4		1/150
Circular steel tanks on flexible base with fixed top; steel framing with flexible siding	1/125	

7.2.3.3.2. Influence of architecture

Facades, siding, and other architectural finishes are usually placed after a portion of the settlement has occurred.

Most settlement, for example, may have already occurred for facilities on cohesionless soil; whereas, very little settlement may have occurred for facilities on compressible, cohesive soil when the facade is to be placed.

a) Larger angular distortions than those shown in Table 7-4 can be accommodated if some of the settlement has occurred before installation of architectural finishes.

b) The allowable angular distortion of the structure, Table 7-4, should be greater than the estimated maximum angular distortion of the foundation, Table 7-5, to avoid distress in the structure.

Table 7-5 Empirical Correlations Between Maximum Distortion (Δ) and Angular Distortion β (From Table 5-3, TM 5-818-1)

Soil	***Foundation***	***Approximate β for Δ = 1 inch***[169]
Sand	Mats	1/750

[169] β increases roughly in proportion with Δ. For $\Delta = 2$ inches, β is about twice as large as those shown; for $\Delta = 3$ inches, three times as large, etc.

	Spread footings	1/600
Varved silt	Rectangular mats	1/1000 to 1/2000
	Square mats	1/2000 to 1/3000
	Spread footings	1/600
Clay	Mats	1/1250
	Spread footings	1/1000

7.2.3.4. Estimation of the Maximum Angular Distortion

The maximum angular distortion for uniformly loaded structures on laterally uniform cohesive soil profiles occurs at the corner, Figure 7-1b. The maximum angular distortion may be estimated from the lateral distribution of calculated settlement. The maximum angular distortion for structures on sand, compacted fill, and stiff clay often occurs anywhere on the foundation because the settlement profile is usually erratic, Figure 7-1c.

(1) The maximum angular distortion at a corner of a foundation shaped in a circular arc on a uniformly loaded cohesive soil for the Boussinesq stress distribution, Figure 5-5, is approximately

Equation 7-1: $$\beta_{\max} = \frac{3\rho_{\max}}{(N_{co1} - 1)\ell}$$

Where

- ρ_{max} = maximum settlement in centre of mat, ft
- N_{col} = number of columns in a diagonal line on the foundation
- ℓ = Distance between adjacent columns on the diagonal line, ft

The maximum settlement may be calculated from loads on soil beneath the centre of the foundation using methodology of § 5.

(2) When the potential for soil heave and nonuniform soil wetting exists, the maximum angular distortion may be the sum of the maximum settlement ρ_{max} without soil wetting and maximum potential heave S_{max} of wetted soil divided by the minimum distance between ρ_{max} and S_{max}. S_{max} may occur beneath the most lightly loaded part of the foundation such as the midpoint between diagonal columns. ρ_{max} may occur beneath the most heavily loaded part of the structure. ρ_{max} will normally only be the immediate elastic settlement ρ_i; consolidation is not expected in a soil with potential for heave *in situ*. Leaking water, sewer, and drain lines may cause nonuniform soil wetting.

(3) When the potential for soil heave and uniform wetting occurs, the maximum angular distortion will be the difference between the maximum and minimum soil heave divided by the minimum distance between these locations. The maximum and minimum heave may occur beneath the most lightly and heavily loaded parts of the structure, respectively. Uniform wetting may occur following construction of the foundation through elimination of evapo-transpiration from the ground surface.

(4) When the potential for soil collapse exists on wetting of the subgrade, the maximum angular distortion will be the difference between the maximum settlement of the collapsible soil ρ_{col} and ρ_{min} divided by the distance between these points or adjacent columns. ρ_{min} may be the immediate settlement assuming collapse does not occur (no soil wetting) beneath a point. See 7.5 for further details on heaving and collapsing soil and 7.3 for details on calculating immediate settlement.

7.2.3.5. Correlations between Deflection Ratio and Angular Distortion

The deflection ratio Δ/L may be estimated from the maximum angular distortion or slope at the support by[170]

Equation 7-2: $$\frac{\Delta}{L} = \frac{\beta}{3}\left[\frac{1+3.9\left(\frac{H_w}{L}\right)^2}{1+2.6\left(\frac{H_w}{L}\right)^2}\right]$$

Where

- ∞ Δ = differential displacement, ft
- ∞ L = span length or length between columns, ft
- ∞ H_w = wall height, ft
- ∞ β = angular distortion

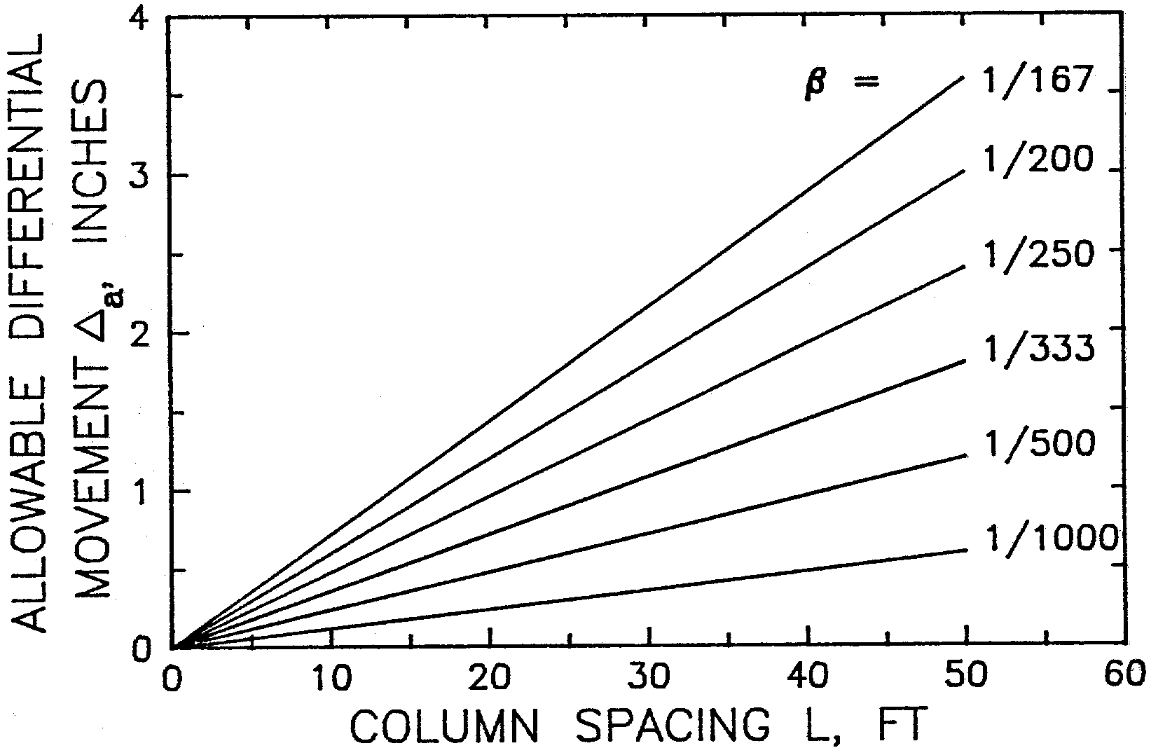
The deflection ratio Δ/L is approximately 1/3 of the angular distortion β for short, long structures or L/H_w greater than 3.

(1) Table 7-5 illustrates empirical correlations between the maximum deflection Δ and angular distortion β for uniformly loaded mats and spread footings on homogeneous sands, silts, and clays.

(2) Figure 7-2 illustrates a relationship between the allowable differential settlement Δ_a, column spacing L, and the angular distortion β.

[170] Wahl, H. E. 1981. "Tolerable Settlement of Buildings," Journal of the geotechnical engineering Division, Vol 107, pp 1489-1504. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Figure 7-2 Allowable differential movement for buildings



7.3. Evaluation of Settlement for Static Loads

7.3.1. General

This section presents the evaluation of immediate settlement in cohesionless and cohesive soils and consolidation settlement of soil for static loads. Settlement is denoted as a positive value to be consistent with standard practice.

7.3.2. Components of Settlement

Total settlement ρ in feet, which is the response of stress applied to the soil, may be calculated as the sum of three components

Equation 7-3: $\rho = \rho_i + \rho_c + \rho_s$

Where

- ρ_i = immediate or distortion settlement, ft
- ρ_c = primary consolidation settlement, ft
- ρ_s = secondary compression settlement, ft

Primary consolidation and secondary compression settlements are usually small if the effective stress in the foundation soil applied by the structure is less than the maximum effective past pressure of the soil, paragraph 1-5a.

7.3.2.1. Immediate Settlement

Immediate settlement ρ_i is the change in shape or distortion of the soil caused by the applied stress.

(1) Calculation of immediate settlement in cohesionless soil is complicated by a nonlinear stiffness that depends on the state of stress. Empirical and semi-empirical methods for calculating immediate settlement in cohesionless soils are described in 7.3.3.

(2) Immediate settlement in cohesive soil may be estimated using elastic theory, particularly for saturated clays, clay shales, and most rocks. Methods for calculating immediate settlement in cohesive soil are described in 7.3.4.

7.3.2.2. Primary Consolidation Settlement

Primary consolidation settlement ρ_c occurs in cohesive or compressible soil during dissipation of excess pore fluid pressure, and it is controlled by the gradual expulsion of fluid from voids in the soil leading to the associated compression of the soil skeleton. Excess pore pressure is pressure that exceeds the hydrostatic fluid pressure. The hydrostatic fluid pressure is the product of the unit weight of water and the difference in elevation between the given point and elevation of free water (phreatic surface). The pore fluid is normally water with some dissolved salts. The opposite of consolidation settlement (soil heave) may occur if the excess pore water pressure is initially negative and approaches zero following absorption and adsorption of available fluid.

(1) Primary consolidation settlement is normally insignificant in cohesionless soil and occurs rapidly because these soils have relatively large permeabilities.

(2) Primary consolidation takes substantial time in cohesive soils because they have relatively low permeabilities. Time for consolidation increases with thickness of the soil layer squared and is inversely related to the coefficient of permeability of the soil. Consolidation settlement determined from results of one-dimensional consolidation tests includes some immediate settlement ρ_i. Methods for calculating primary consolidation settlement are described in 7.3.5.

7.3.2.3. Secondary Compression Settlement

Secondary compression settlement is a form of soil creep that is largely controlled by the rate at which the skeleton of compressible soils, particularly clays, silts, and peats, can yield and compress. Secondary compression is often conveniently identified to follow primary consolidation when excess pore fluid pressure can no longer be measured; however, both processes may occur simultaneously. Methods for calculating secondary compression settlement are described in 7.3.6.

7.3.3. Immediate Settlement of Cohesionless Soil for Static Loads

Settlement in cohesionless soil (see 5.2.7 for definition) is normally small and occurs quickly with little additional long-term compression. Six methods described below for estimating settlement in cohesionless soil are based on data from field tests (i.e., Standard Penetration Test (SPT), Cone Penetration Test (CPT), Dilatometer Test (DMT) and Pressuremeter Test (PMT). Undisturbed samples of cohesionless soil are normally not obtainable for laboratory tests. The first four empirical and semi-empirical methods - Alpan, Schultze and Sherif, Modified Terzaghi and Peck, and Schmertmann approximations - were shown to provide estimates from about 1/4 to 2 times the measured settlement for 90% confidence based on the results of a statistical analysis[171]. Penetration tests may not be capable of sensing effects of prestress or overconsolidation and can underestimate the stiffness that may lead to overestimated settlements[172].

7.3.3.1. Alpan Approximation

This procedure estimates settlement from a correlation of (SPT) data with settlement of a 1-ft square loading plate. The settlement of a footing of width B in feet is[173]

Equation 7-4: $$\rho_i = m'\left[\frac{2B}{1+B}\right]^2 \frac{\alpha_o}{12} q$$

Where

- ρ_i = immediate settlement, ft
- m' = shape factor, $(L/B)^{0.39}$
- L = length of footing, ft
- B = width of footing, ft
- α_o = parameter from Figure 7-3a using an adjusted blow count N' from Figure 7-3b, inches/tsf
- q = average pressure applied by footing on soil, tsf

7.3.3.1.1. Blow count N

The converted blow count N_{60} from Equation 4-2 is entered in Figure 7-3a with the calculated effective overburden pressure σ'_o at the base of the footing to estimate the relative density D_r. The relative density is adjusted to 100% using the Terzaghi-Peck curve and the adjusted blow count N' read for D_r = 100%.

For example, if σ'_o = 0.3 tsf and N = 10, then the relative density D_r = 67%, Figure 7-3a. The adjusted N' is determined as 31 for D_r = 100%.

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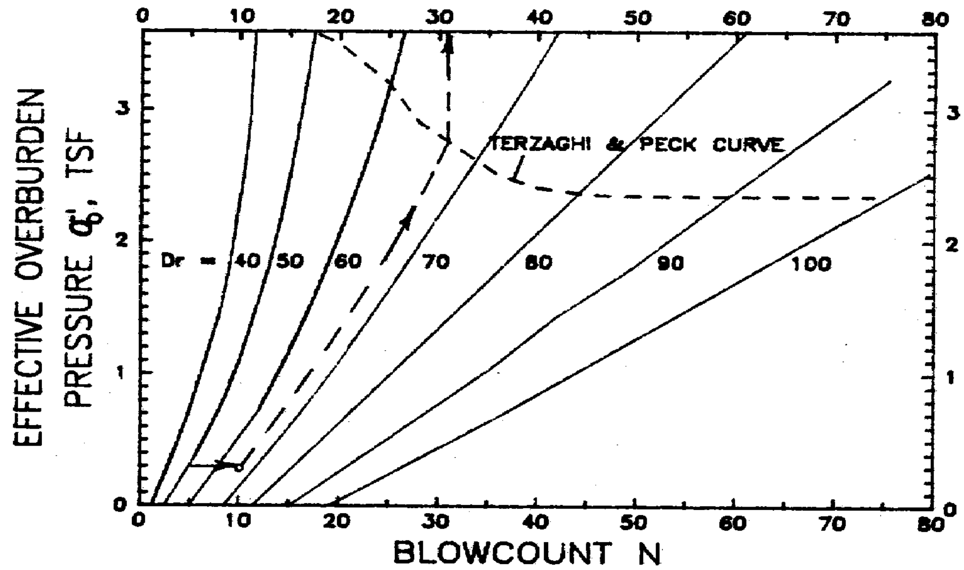
[171] Jeyapalan, J. and Boehm, R. 1986. "Procedures for Predicting Settlement in Sands," Settlement of Shallow Foundations on Cohesionless Soils: Design and Performance, Geotechnical Special Publication No. 5, pp 1-22.

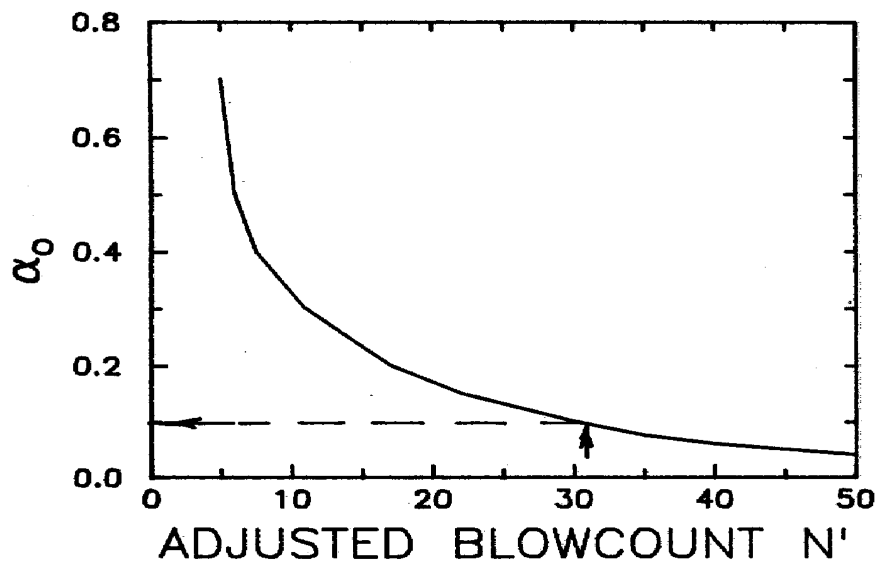
[172] Leonards, G. A. and Frost, J. D. 1988. "Settlement of Shallow Foundations on Granular Soils," Journal of geotechnical engineering, Vol 114, pp 791-809. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

[173] Alpan, I. 1964. "Estimating the Settlements of Foundations on Sands," Civil Engineering and Public Works Review, Vol 59, pp 1415-1418. Available from Morgan-Grampian Ltd., 30 Calderwood Street, Woolwich, London, SE18 GQH, England.

Figure 7-3 Chart to apply Alpan's procedure



a. ADJUSTED BLOWCOUNT FROM N AND σ_o'



b. PARAMETER α_o FROM ADJUSTED BLOWCOUNT

7.3.3.1.2. Parameter α_o

The adjusted blow count is entered in Figure 7-3b to determine α_o. α_o = 0.1 inch/tsf for adjusted N' = 31.

7.3.3.2. Schultze and Sherif Approximation

This procedure estimates settlement from the blow count of SPT results based on 48 field cases[174]

Equation 7-5: $$\rho_1 = \frac{fq\sqrt{B}}{N_{ave}^{0.87}\left(1+0.4\frac{D}{B}\right)}$$

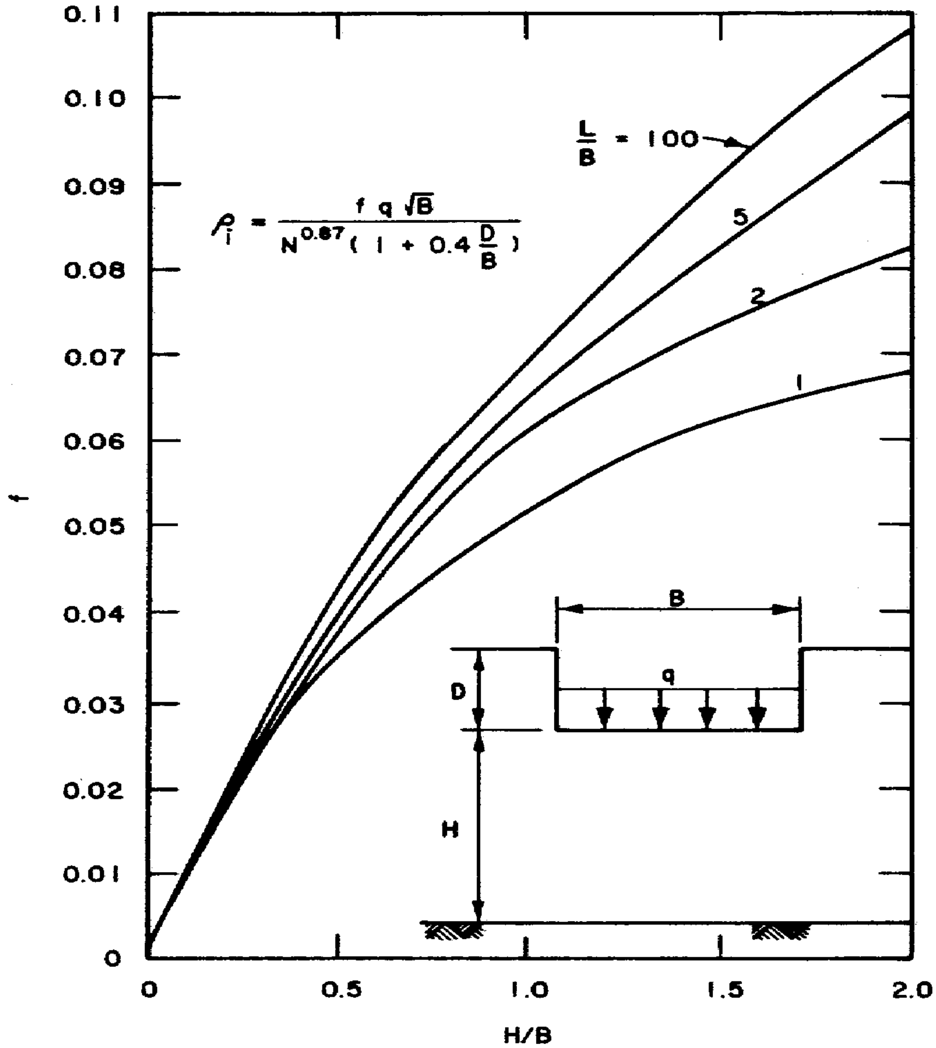
Where

- f = influence factor from elasticity methods for isotropic half space, Figure 7-4
- H = depth of stratum below footing to a rigid base, ft
- D = depth of embedment, ft
- N_{ave} = average blow count/ft in depth H

The depth to the rigid base H should be ≤ 2B. N_{ave} is based on measured blow counts adjusted to N_{60} by Equation 4-2.

[174] Schultze, E. and Sherif, G. 1973. "Prediction of Settlements From Evaluated Settlement Observations for Sand," Proceedings Eighth International Conference on Soil Mechanics and Foundation Engineering, Vol 1, pp 225-230. Available from USSR National Society for Soil Mechanics and Foundation Engineers, Gosstray USSR, Marx Prospect 12, Moscow K-9.

Figure 7-4 Settlement from the standard penetration test



7.3.3.3. Modified Terzaghi and Peck Approximation

This procedure is a modification of the original Terzaghi and Peck approach to consider overburden pressure and water table[175]

Equation 7-6: $\rho_i = \dfrac{q}{18 q_1}$

Where q_1 = soil pressure from Figure 7-5a using corrected blow count N' and the ratio of embedment depth D to footing width B, tsf. The corrected blow count N' is found from

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[175] Peck, R. B. and Bazarra, A. 1969. "Discussion of Settlement of Spread Footings on Sand," by D'Appolonia, et. al, Journal of the Soil Mechanics and Foundations Division, Vol 95, pp 905-909. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017, and Peck, R. B., Hanson, W. F., and Thornburn, T. H. 1974. Foundation Engineering, 2nd ed., pp 307-314. Available from John Wiley and Sons, 605 3rd Ave., New York, NY 10016.

Equation 7-7: $N' = (N_1)_{60} C_W$

Where

$(N_1)_{60}$ = average blow count per foot in the sand, corrected according to Equation 4-2.

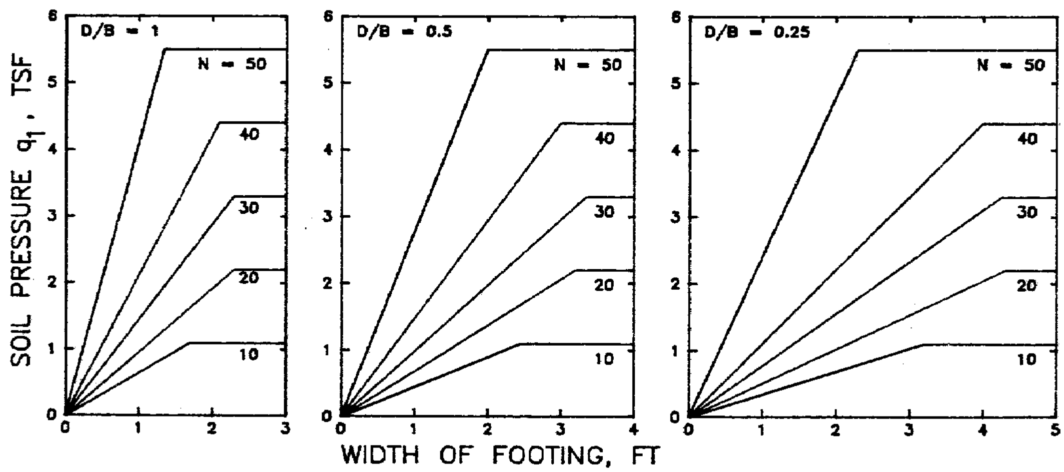
C_w = correction for water table depth, Equation 7-8.

Equation 7-6 calculates settlements 2/3 of the Terzaghi and Peck method as recommended by Peck and Bazarra. The correction water table correction C_w is given by

Equation 7-8: $C_W = 0.5 + 0.5 \frac{D_W}{D + B}$

where D_w = depth to groundwater level, ft. The correction factor C_w = 0.5 for a groundwater level at the ground surface. The correction factor is 1 if the sand is dry or if the groundwater level exceeds the depth D + B below the ground surface.

Figure 7-5 Charts for Modified Terzaghi and Peck Approximation[176]



EVALUATION OF SOIL PRESSURE q_1 FROM CORRECTED BLOWCOUNT N' AND EMBEDMENT DEPTH/FOOTING WIDTH RATIO D/B

7.3.3.4. Schmertmann Approximation

This procedure provides settlement compatible with field measurements in many different areas. The analysis assumes that the distribution of vertical strain is compatible with a linear elastic half space subjected to a uniform pressure

Equation 7-9: $\rho_i = C_1 C_t \Delta p \sum_{i=1}^{n} \frac{\Delta z_i}{E_{si}} I_{z1}$

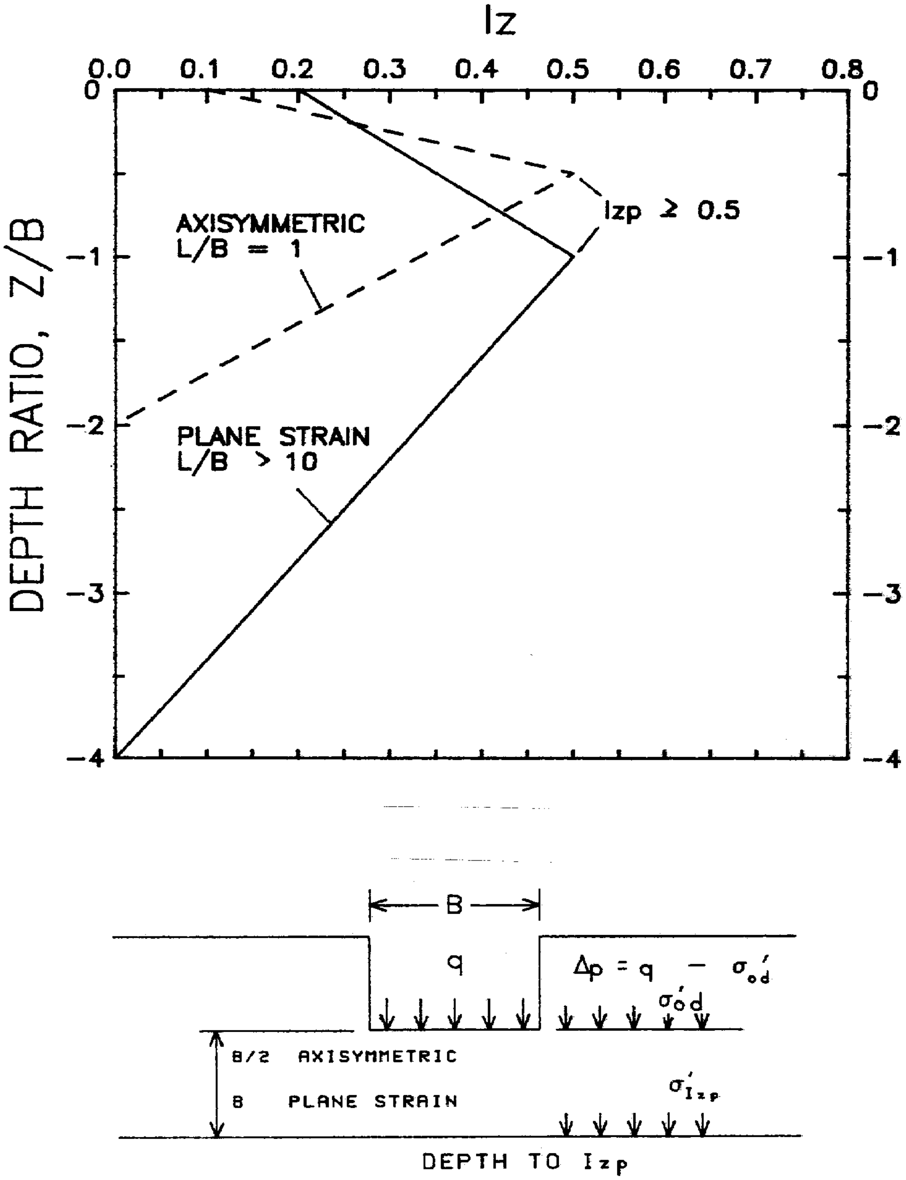
Where

[176] Reprinted by permission of John Wiley & Sons, Inc. from *Foundation Engineering*, 2nd Edition, Copyright © 1974 by R. B. Peck, W. E. Hanson, and T. H. Thornburn, pp 309, 312

- C_1 = correction to account for strain relief from embedment = $1 - 0.5\frac{\sigma'_{od}}{\Delta p} \geq 0.5$
- σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf
- Δp = net applied footing pressure = $q - \sigma'_{od}$, tsf
- C_t = correction for time dependent increase in settlement = $1 + 0.2\log_{10}\frac{t}{0.1}$
- t = time, years
- E_{si} = elastic modulus of soil layer i, tsf
- Δz_i = depth increment i, 0.2B, ft
- I_{zi} = influence factor of soil layer i, Figure 7-6

Settlement may be calculated with the assistance of the calculation sheet, Figure 7-7. The time-dependent increase in settlement is related with creep and secondary compression as observed in clays.

Figure 7-6 Recommended strain influence factors for Schmertmann's Approximation[177]



Z = DEPTH BELOW FOOTING BOTTOM, FT
B = FOOTING WIDTH, FT
Iz = DEPTH INFLUENCE FACTOR
Izp = PEAK DEPTH INFLUENCE FACTOR

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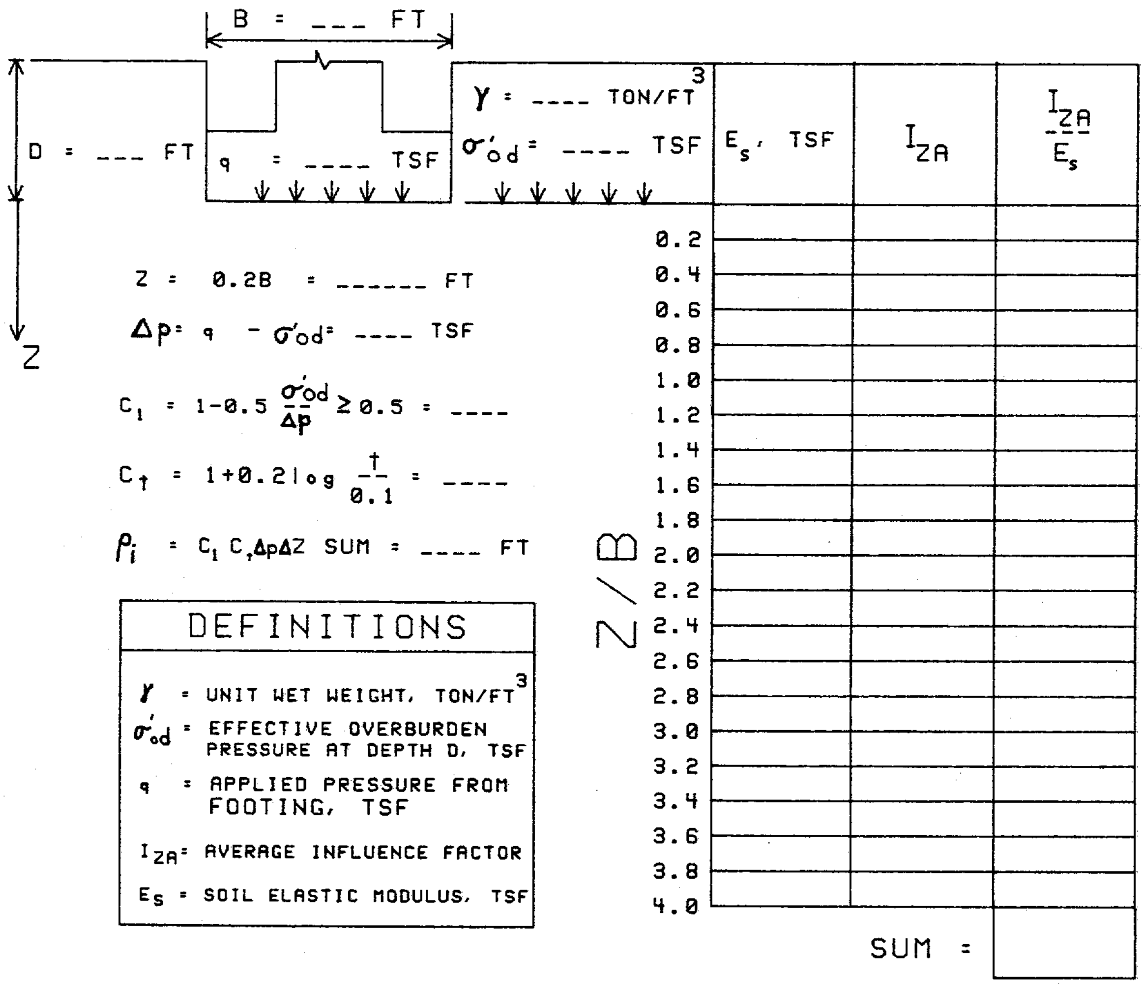
177 Reprinted with permission of the American Society of Civil Engineers from the Journal of the geotechnical engineering Division, Vol 104, 1978, "Improved Strain Influence Factor diagram", by J. M. Schmertmann, J. P. Hartman, and P. R. Brown, p 1134

Figure 7-7 Settlement calculation sheet for cohesionless soil using Schmertmann's method



7.3.3.4.1. Influence factor

The influence factor I_z is based on approximations of strain distributions for square or axisymmetric footings and for infinitely long or plane strain footings observed in cohesionless soil, which are similar to an elastic medium such as the Boussinesq distribution. The peak value of the influence factor I_{zp} in Figure 7-6 is[178]

[178] Schmertmann, J. H., Hartman, J. P., and Brown, P. R. 1978. "Improved Strain Influence Factor Diagrams," Journal of the geotechnical engineering Division, Vol 104, pp 1131-1135, Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Equation 7-10: $I_{zp} = 0.5 + 0.1\sqrt{\dfrac{\Delta p}{\sigma'_{Izp}}}$

Equation 7-11: $\sigma'_{Izp} = 0.5B\gamma' + D\gamma'$ **(Axisymmetric, L/B = 1)**

Equation 7-12: $\sigma'_{Izp} = B\gamma' + D\gamma'$ **(Plane Strain, L/B ≥ 10)**

Where

- σ'_{Izp} = effective overburden pressure at the depth of I_{zp}, tsf
- γ' = effective unit weight (wet soil unit weight γ less unit weight of water) in units of ton/cubic foot.
- D = excavated or embedded depth, ft

The parameter σ_{Izp} may be assumed to vary linearly between Equation 7-11 and Equation 7-12 for L/B between 1 and 10. I_z may be assumed to vary linearly between 0.1 and 0.2 on the Iz axis at the ground surface for L/B between 1 and 10 and Z/B may be assumed to vary linearly between 2 and 4 on the Z/B axis for L/B between 1 and 10.

7.3.3.4.2. Elastic modulus

Elastic modulus E_{si} may be estimated from results of the mechanical (Dutch Static) Cone Penetration Test (CPT)

Equation 7-13: $E_{s1} = 2.5q_c$ **(Axysymmetric Footings, L/B = 1)**

Equation 7-14: $E_{s1} = 3.5q_c$ **(Plane Strain Footings, L/B ≥ 10)**

where q_c is the cone tip bearing resistance in units of tsf. E_{si} may be assumed to vary linearly between Equation 7-13 and Equation 7-14 for L/B between 1 and 10. SPT data may also be converted to Dutch cone bearing capacity by the correlations in Table 7-6. The estimated average elastic modulus of each depth increment may be plotted in the E_s column of Figure 7-7.

Table 7-6 Correlations between Dutch Cone Tip Resistance q_c and Blow Count N from the SPT[179][180]

Soil	*q_c/N_**
Silts, sandy silts, slightly cohesive silt-sand	2
Clean, fine to medium sands and slightly silty sands	3.5
Coarse sands and sands with little gravel	5
Sandy gravel and gravel	6

7.3.3.4.3. Calculation of settlement

I_z/E_s is computed for each depth increment z/B and added to obtain SUM, Figure 7-7. Immediate settlement of the soil profile may then be calculated as shown on Figure 7-7. If a rigid base lies within z = 2B, then settlement may be calculated as shown down to the rigid base.

[179] Schmertmann, J. H. 1970. "Static Cone to Compute Static Settlement Over Sand," Journal of the Soil Mechanics and Foundations Division, Vol 96, pp 1011-1043. Available from American Society of Civil Engineers, 345

[180] Units of q_c are in tsf and N in blows/ft

7.3.3.5. Burland and Burbidge Approximation

This procedure based on 200 SPT case studies predicts settlements less than most of these methods[181].

1) Immediate settlement of sand and gravel deposits may be estimated by

$$\textbf{Equation 7-15: } \rho_i = f_s f_i \left[\left(\Delta P'_{ave} - \frac{2}{3}\sigma'_p \right) B^{0.7} I_c \right] \quad \Delta P'_{ave} > \sigma'_p$$

$$\textbf{Equation 7-16: } \rho_i = f_s f_i \Delta P'_{ave} \frac{I_c}{3} \quad \Delta P'_{ave} < \sigma'_p$$

Where

- f_s = shape correction factor, $[(1.25\ L/B)/(L/B + 0.25)]^2$
- f_l = layer thickness correction factor, $H/z_1\ (2 - H/z_1)$
- $\Delta P'_{ave}$ = average effective bearing pressure, $q_{oave} + \sigma'_{oave}$, tsf
- q_{oave} = average pressure in stratum from foundation load, tsf
- σ'_{oave} = average effective overburden pressure in stratum H, tsf
- σ'_p = maximum effective past pressure, tsf
- H = thickness of layer, ft
- z_1 = depth of influence of loaded area, ft
- I_c = compressibility influence factor, $\approx 0.23/(N^{1.4}_{ave})$ with coefficient of correlation 0.848
- N_{ave} = average SPT blow count over depth influenced by loaded area

a) The depth of influence z_1 is taken as the depth at which the settlement is 25% of the surface settlement. This depth in feet may be approximated by $1.35B^{0.75}$ where N_{ave} increases or is constant with depth. z_i is taken as 2B where N_{ave} shows a consistent decrease with depth.

b) N_{ave} is the arithmetic mean of the measured N values within the depth of influence z_1. N_{ave} is not corrected for effective overburden pressure, but instead considers compressibility using I_c. The arithmetic mean of the measured N_{ave} should be corrected to $15 + 0.5(N_{ave} - 15)$ when $N_{ave} > 15$ for very fine and silty sand below the water table and multiplied by 1.25 for gravel or sandy gravel.

c) The probable limits of accuracy of Equation 7-15 and Equation 7-16 are within upper and lower bound values of I_c given by

$$\textbf{Equation 7-17: } \frac{0.08}{N^{1.3}_{ave}} \le I_c \le \frac{1.34}{N^{1.67}_{ave}}$$

2) Settlement after time t at least 3 years following construction from creep and secondary compression effects may be estimated by

$$\textbf{Equation 7-18: } \rho_c = f_t \rho_1$$

Where

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[181] Burland, J. B. and Burbidge, M. C. 1985. "Settlement of Foundations on Sand and Gravel," Proceedings, Institution of Civil Engineers, Part 1, Vol 78, pp 1325-1381. Available from Thomas Telford Ltd., 1-7 Great George Street, Westminster, London, SW1P 3AA, England.

- $f_t = 1 + R_3 + R_t \log t/3$
- R_3 = time-dependent settlement ratio as a proportion of ρ_i during first 3 years following construction, ≈ 0.3
- R_t = time-dependent settlement ratio as a proportion of ρ_i for each log cycle of time after 3 years, ≈ 0.2

Values of R_3 and R_t are conservative based on 9 case records.

7.3.3.6. Dilatometer Approximation

The dilatometer consists of a stainless steel blade 96 mm wide and 15 mm thick with a sharp edge containing a stainless steel membrane centred and flush with one side of the blade. The blade is preferably pushed (or driven if necessary) into the soil. A pressure-vacuum system is used to inflate/deflate the membrane a maximum movement of 1.1 mm against the adjacent soil[182].

7.3.3.6.1. Calculation

This procedure predicts settlement from evaluation of one-dimensional vertical compression or constrained modulus E_d by the DMT

Equation 7-19: $$\rho_i = \frac{q_{oave} H}{E_d}$$

Where

- q_{oave} = average increase in stress caused by the applied load, tsf
- H = thickness of stratum at depth z where q_{oave} is applicable, ft
- E_d = constrained modulus, $R_D E_s$, tsf
- $R_D = (1 - \nu_s)/[(1 + \nu_s)(1 - 2\nu_s)]$, factor that varies from 1 to 3 relates E_d to Young's soil modulus E_s ν_s = Poisson's ratio 3-11

Refer to 3.3 for additional information on elastic parameters. The influence of prestress on settlement may be corrected using results of DMT and CPT tests after Schmertmann's approximation to reduce settlement overestimates.

7.3.3.6.2. Evaluation of elastic modulus

The dilatometer modulus of soil at the depth of the probe is evaluated as 34.7 times the difference in pressure between the deflated and inflated positions of the membrane. Young's elastic modulus has been found to vary from 0.4 to 10 times the dilatometer modulus[183]. A Young's elastic modulus equal to the dilatometer modulus may be assumed for many practical applications in sands.

7.3.3.6.3. Adjustment for other soil

The constrained modulus E_d may be adjusted for effective vertical stress σ'_o other than that of the DMT for overconsolidated soil and normally consolidated clay by

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182 Schmertmann, J. H. 1986. "Dilatometer to Compute Foundation Settlement," Use of Insitu Tests in geotechnical engineering, Geotechnical Special Publication No. 6, pp 303-321. Available from American Society of Civil Engineers, New York, NY 10017.

183 Lutenegger, A. J. 1988. "Current Status of the Marchetti Dilatometer Test," Penetration Testing 1988 ISOPT-1, Vol 1, pp 137-155, Orlando, FL, ed. J. DeRuiter. Available from A. A. Balkema, P.O. Box 1675, Rotterdam, The Netherlands.

Equation 7-20: $E_d = m\sigma'_o$

where m = [(1 + e)/C_c] ln 10

e = void ratio

C_c = compression index

The constrained modulus for normally consolidated silts and sand is

Equation 7-21: $E_d = m\sqrt{\sigma'_o}$

Where σ'_o is the effective vertical overburden pressure, tsf. These settlements include time-dependent settlements excluding secondary compression and creep. Total settlement of a heterogeneous soil with variable E_d may be estimated by summing increments of settlement using Equation 7-19 for layers of thickness H.

7.3.3.7. Recommendations

A minimum of three methods should be applied to estimate a range of settlement. Settlement estimates based on *in situ* test results are based on correlations obtained from past experience and observation and may not be reliable.

7.3.3.7.1. Evaluation from SPT Data

The Alpan (Equation 7-4), Schultze and Sherif (Equation 7-5), Modified Terzaghi and Peck (Equation 7-6) approximations should all be applied to estimate immediate settlement if blow count data from SPT are available. The Burland and Burbidge approximation (Equation 7-15 and Equation 7-16) should be applied if the maximum past pressure of the soil can be estimated; this approximation using Equation 7-17 may also be applied to estimate a range of settlement.

7.3.3.7.2. Evaluation from CPT Data

The Schmertmann approximation (Equation 7-9) should be used to estimate settlement if CPT data are available.

7.3.3.7.3. Evaluation from DMT Data

The Dilatometer approximation (Equation 7-19) should be used if data from this test are available. The range of settlement may be determined by assuming minimum and maximum values of the factor R_D of 1 and 3.

7.3.3.7.4. Evaluation from PMT Data

The pressuremeter unload-reload modulus from the corrected pressure versus volume change curve is a measure of twice the shear modulus, 3.3. The Young's elastic modulus may be evaluated from the shear modulus, Table 3-7, and settlement estimated from Equation 7-9. The constrained modulus may be evaluated from Young's elastic modulus, Table 3-7, and settlement estimated from Equation 7-19.

7.3.3.7.5. Long-Term Settlement

The Schmertmann and Burland and Burbidge approximations may be used to estimate long-term settlement in cohesionless soil from CPT and SPT data. The constrained modulus E_d may also be adjusted to consider consolidation from Equation 7-20 and Equation 7-21 and settlement estimated from Equation 7-19.

7.3.3.8. Application

A footing 10-ft square is to be constructed 3 ft below grade on medium dense ($\gamma = \gamma' = 0.06$ ton/ft^3) and moist sand with total stratum thickness of 13 ft (H = 10'). The water table is at least 5 ft below the base of the footing. The effective vertical overburden pressure at the bottom of the footing is $\sigma'_{od} = \gamma'\ z = (0.06)(3) = 0.18$ tsf. The bearing pressure of the footing on the sand q = 2 tsf. Field data indicate an average blow count in the sand $N_{ave} = 20$ blows/ft and the cone tip bearing resistance is about 70 tsf. The average elastic modulus determined from dilatometer and pressuremeter tests indicated $E_s = 175$ tsf. Refer to Figure 7-8 for a schematic description of this problem. Estimates of settlement of this footing at end of construction (EOC) and 10 years after construction are required.

(1) Results of the settlement computations comparing several of the above methods are shown in Table 3-2.

a. Figure 3-6 illustrates computation of settlement by Schmertmann's method.

b. Computation of settlement by the Burland and Burbidge and dilatometer approximations requires an estimate of the average effective bearing pressure $\Delta P'_{ave.}$ Assuming that the 2:1 stress distribution of Figure 7-1 is adequate, the average pressure from the foundation load is

$$q_{oave} = \frac{q + \Delta\sigma_z}{2}$$

where $\Delta\sigma_z$ is found

from Equation 7-2. Therefore, if B = L = H = 10 ft and Q = q B L, then

$$q_{oave} = 0.5\left[2.0 + \frac{2.0 + 10^2}{(10+10)^2}\right] = 1.25 \text{ tsf}$$

The average effective overburden pressure $\sigma'_{oave} = 0.06\ (3 + 13)/2.0$ or 0.48 tsf. The average effective bearing pressure $\Delta P'_{ave}$ is therefore 1.25 + 0.48 = 1.73 tsf. The soil is assumed normally consolidated; therefore, $\sigma'_p = \sigma'_{od} = 0.18$ tsf and Equation 7-15 is applicable. Factor $f_s = 1.0$, $H/z_I = 1.31$ and $f_l = 0.91$. $I_c = 0.23/(20)_{1.4} = 0.0035$.

Table 7-7 Estimation of Immediate Settlement for Example Application of Footing on Cohesionless Soil

7.3.3.8.1. Calculations

Method	*Equation*	*Calculations*
Alpan		B = 10', L/B = 1, m' = 1.0, N' = 65 blows/foot. N = 20 blows/foot and $\sigma'_o = 0.18$ tsf,
	Equation 7-4	$\rho_i = 1\ ((2)(10)/(1+10))^2(0.05/12)(2) = 0.027' = 0.33''$
Schultz and Sherif	Equation 7-5	H/B = 1, D/B = 0.3, f = 0.052, Figure 7-4 $\rho_i = ((0.052)(2)(10)^{0.5})/(20^{0.87}(1+(0.4)(0.3)) = 0.022' = 0.33''$
Modified Terzaghi and Peck	Equation 7-7	$C_n = 1.6$, Figure 7-5b $C_w = 1$ N' =(1)(1.6)(20) = 32 $q_1 = 3.5$ tsf, Figure 7-5a
	Equation 7-6	$\rho_i = (1/18)\ (2/3.5) = 0.031'$ or 0.38" (0.57" ignoring Peck and Bazzara 1/2 reduction)

	7-6	1/3 reduction)
Schmertmann		Refer to
	Equation 7-13	$q_c = 70$ tsf
	Equation 7-12	$E_{sl} = (2.5)(70) = 175$ tsf
	Equation 7-11	$\sigma'_{Izp} = (0.5)(10)(0.06) + (3)(0.6) = 0.48$ tsf $I_{zp} = 0.5 + (0.1)(1.82/0.48)^{0.5} = 0.695$

z, ft.	z/B	I_z	I_{za}
0	0.0	0.1	
2	0.2	0.34	0.22
4	0.4	0.58	0.62
6	0.6	0.65	0.61
8	0.8	0.56	0.51
10	1.0	0.46	

$$\rho_i \ @ \ \text{EOC} = (.95)(1)(1.82)(2)(0.0138) = 0.0477' = 0.57''$$

$$\rho_i \ @ \ 10 \text{ yr} = (0.0477)(1.4) = 0.0668' = 0.80''$$

Burland and Burbidge	Equation 7-15	End of Construction: $\rho_i = (1)(0.91)((1.73-(0.67)(0.18))(10)^{0.7}(0.0035)) = 0.028' = 0.34''$
	Equation 7-18	$\rho_t = (0.028)(1.4) = 0.039' = 0.47''$
	Equation 7-17	$I_{cmin} = 0.08/20^{1.4} = 0.0016$ $\rho_{imin} = (0.028)(0.0016)/(0.0035) = 0.013' = 0.15''$
	Equation 7-17	$I_{cmax} = 1.34/20^{1.67} = 0.009$ $\rho_{imax} = (0.028)(0.009)/(0.0035) = 0.073' = 0.87''$
Dilatometer		$q_{oave} = 1.25$ tsf, H = 10', $E_s = 175$ tsf, Poisson's ratio not needed.
	Equation 7-19	Minimum settlement: $R_D = 1$, $E_d = (3)(175) = 525$ tsf $\rho_{imin} = (1.25)(10)/525 = 0.024' = 0.29''$
	Equation 7-19	Maximum Settlement: $R_D = 1$, $E_d = (1)(175) = 175$ tsf $\rho_{imax} = (1.25)(10)/(175) = 0.071' = 0.86''$

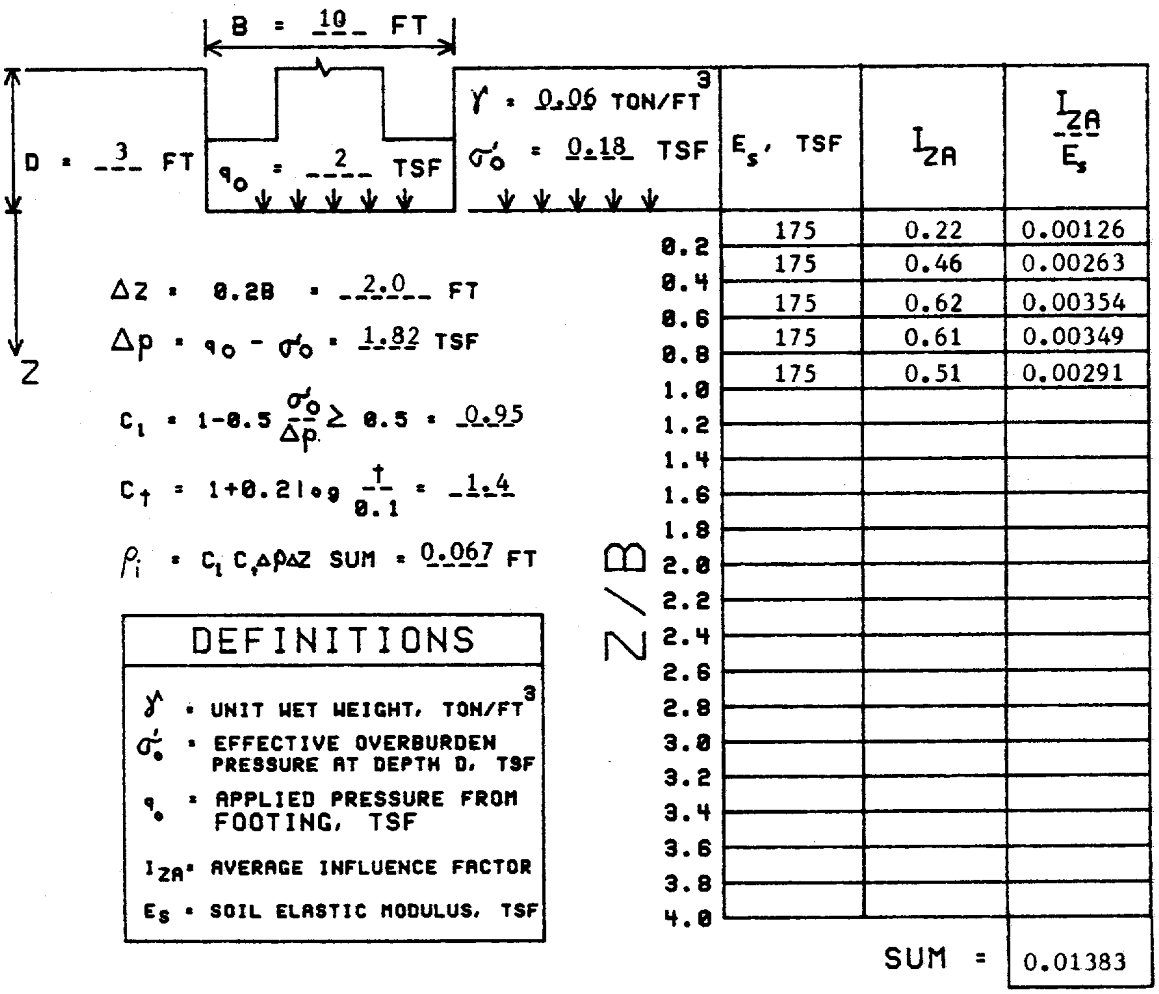
7.3.3.8.2. Comparison

Method	*Immediate Settlement, ft (in.)*
Alpan	0.027 (0.33)
Schultz and Sherif	0.022 (0.27)
Modified Terzaghi and Peck	0.031 (0.38)
Schmertmann	0.048 (0.57)

Burland and Burbidge	0.028 (0.34)
Dilatometer	0.024 - 0.071 (0.29-0.86)

(2) A comparison of results in Table 7-7b shows that the Alpan, Schultze and Sherif, Modified Terzaghi and Peck, and Burland and Burbidge methods provide consistent settlements of about 0.3 to 0.4 inch. The Schmertmann method is reasonably conservative with settlement of 0.57 inch. This settlement is the same as that from the Modified Terzaghi and Peck method ignoring the 1/3 reduction recommended by Peck and Bazarra. Longterm settlement is 0.5 (Burland and Burbidge) and 0.8 inch (Schmertmann) after 10 years. The expected range of settlement is 0.2 to 1.0 inch after the Burland and Burbidge method and 0.3 to 0.9 inch from the dilatometer. Settlement is not expected to exceed 1 inch.

Figure 7-8 Estimation of immediate settlement by Schmertmann's method



7.3.4. Immediate Settlement of Cohesive Soil for Static Loads

Static loads cause immediate and long-term consolidation settlements in cohesive or compressible soil. The stress in the soil caused by applied loads should be estimated (7.1.2.2) and compared with estimates of the maximum past pressure (7.1.2.1). If the stress in the soil exceeds the maximum past pressure, then primary consolidation and secondary compression settlement may be significant and should be evaluated by the methods in Sections III and IV. Immediate rebound or heave may occur in compressible soil at the bottom of excavations, but may not be a design or construction problem unless rebound causes the elevation of the basement or first floor to exceed specifications or impair performance.

7.3.4.1. Rebound in Excavations

Most rebound in excavations lying above compressible strata occurs from undrained elastic unloading strains in these strata. Additional long-term heave due to wetting of the soil following reduction in pore water pressure following removal of overburden in excavations is discussed in 7.5. Rebound of compressible soil in excavations may be approximated as linear elastic by[184]

Equation 7-22: $$S_{RE} = F_{RD} F_{RS} \frac{\gamma D}{E_s^*}$$

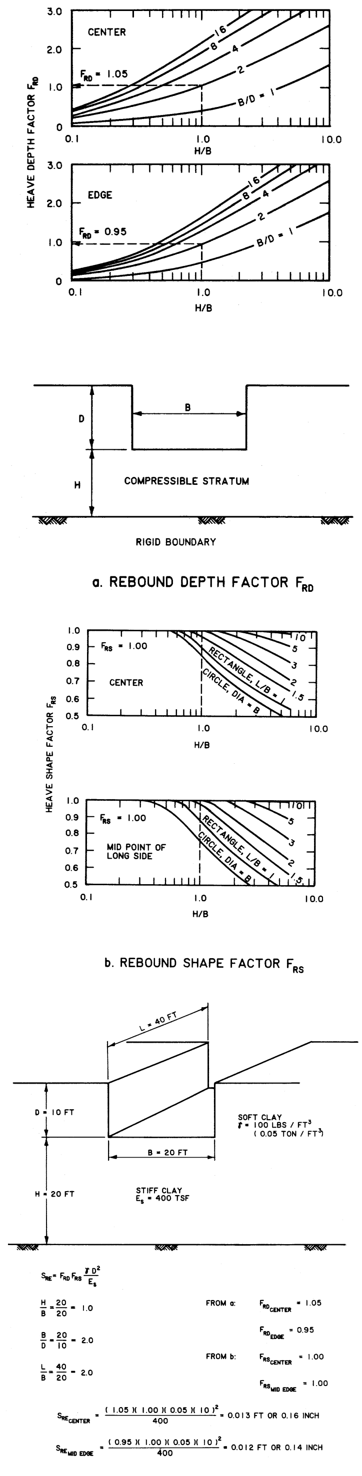
Where

- ∞ S_{RE} = undrained elastic rebound, ft
- ∞ F_{RD} = rebound depth factor, Figure 7-9a
- ∞ F_{RS} = rebound shape factor, Figure 7-9b
- ∞ γ = wet unit weight of excavated soil, tons/ft^3
- ∞ D = depth of excavation, ft
- ∞ E^*_s = equivalent elastic modulus of soil beneath the excavation, tsf.

The equivalent elastic modulus E^*_s may be estimated by methods described in 3.3. The compressible stratum of depth H is assumed to be supported on a rigid base such as unweathered clay shale, rock, dense sand or gravel. An example application is provided in Figure 7-9c.

[184] Baladi, G. 1968. "Distribution of Stresses and Displacements Within and Under Long, Elastic and Viscoelastic Embankments," Available from Purdue University, West LaFayette, IN 47907

Figure 7-9 Factors to calculate elastic rebound in excavations[185]



[185] Reprinted by permission of the author G. Y. Baladi from "Distribution of Stresses and Displacements Within and Under Long Elastic and Viscoelastic Embankments," Ph.D. Thesis, 1968, Purdue University

7.3.4.2. Immediate Settlement in Cohesive Soil

The immediate settlement of a structure on cohesive soil (see 5.2.7 for definition) consists of elastic distortion associated with a change in shape without volume change and, in unsaturated clay, settlement from a decrease in volume. The theory of elasticity is generally applicable to cohesive soil.

7.3.4.2.1. Improved Janbu Approximation

The average immediate settlement of a foundation on an elastic soil may be given by[186]

Equation 7-23: $$\rho_i = \mu_o \mu_i \frac{qB}{E_s^*}$$

Where

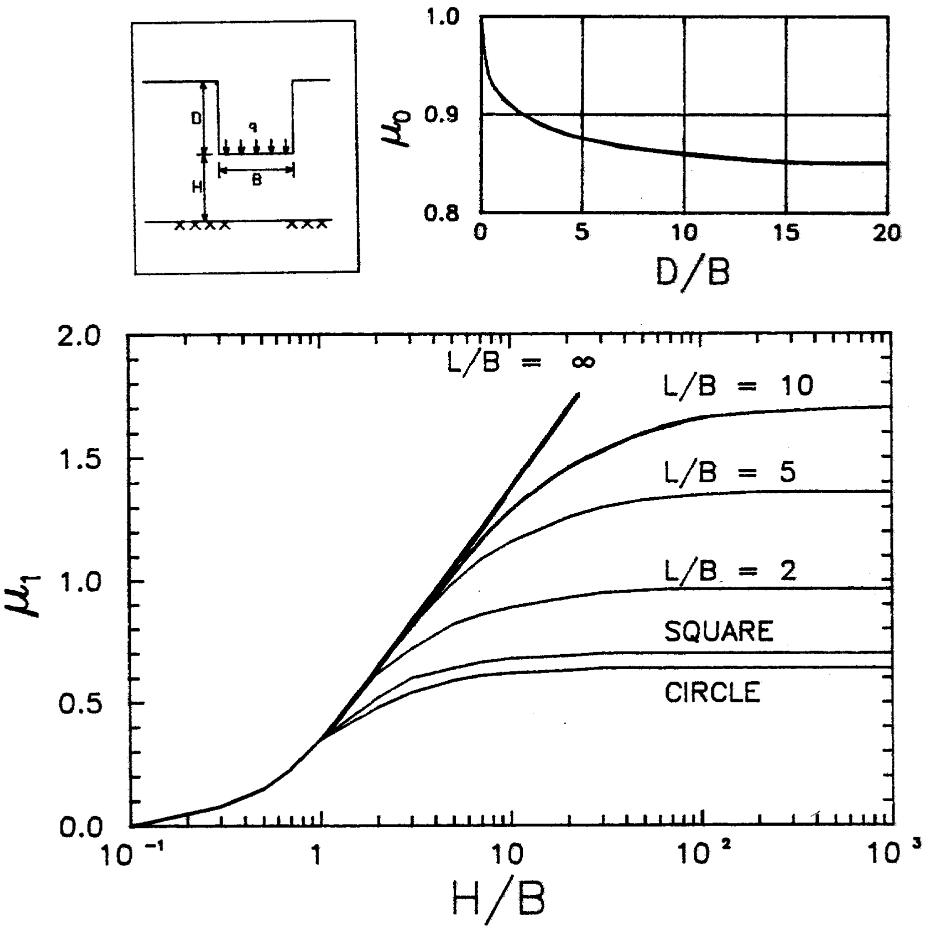
- μ_o = influence factor for depth D of foundation below ground surface, Figure 7-10
- μ_1 = influence factor for foundation shape, Figure 7-10
- E^*_s = equivalent Young's modulus of the soil, tsf

(1) A comparison of test calculations and results of finite element analysis have indicated errors from Equation 7-23 usually less than 10% and always less than 20% for H/B between 0.3 and 10, L/B between 1 and 5, and D/B between 0.3 and 3, Figure 7-10. Reasonable results are given in most cases when μ_o is set equal to unity. Poisson's ratio ν_s is taken as 0.5.

(2) E^*_s may be estimated by methods in 3.3.

[186] Christian, J. T. and Carrier III, W. D. 1978. "Janbu, Bjerrum and Kjaernsli's Chart Reinterpreted," Canadian Geotechnical Journal. Available from National Research Council of Canada, Resarch Journals, Ottawa, Ontario K1A OR6, Canada.

Figure 7-10 Chart for estimating immediate settlement in cohesive soil[187]



$$\rho_i = \mu_0 \cdot \mu_1 \cdot \frac{q \cdot B}{E_s^*}$$

L = LENGTH, FT
B = WIDTH, FT
E_s^* = EQUIVALENT YOUNG'S SOIL MODULUS, TSF
ν_s = POISSON'S RATIO, 0.5

7.3.4.2.2. Perloff Approximation

The immediate vertical settlement beneath the centre and edge of a mat or footing may be given by

Equation 7-24: $$\rho_i = IqB\left[\frac{1-\nu_s^2}{E_s}\right]\alpha$$

Where

∞ I = influence factor for infinitely deep and homogeneous soil, Table 7-8a

[187] Reprinted by permission of the National Research Council of Canada from Canadian Geotechnical Journal, Vol 15, 1978, "Janbu, Bjerrum, and Kjaernsli's Chart Reinterpreted", by J. T. Christian and W. D. Carrier III, p 127.

- E_s = elastic soil modulus, tsf
- ν_s = soil Poisson's ratio
- α = correction factor for subgrade soil, Table 7-8b

The influence factor I may be modified to account for heterogeneous or multilayered soil usually encountered in practice. If the upper soil is relatively compressible and underlain by stiff clay, shale, rock, or dense soil, then a finite layer of depth H supported on a rigid base may approximate the compressible soil stratum. The influence factor I is given in Figure 7-11 for settlement beneath the centre and midpoint of the edge of flexible foundations. If the subgrade soil supporting the foundation with modulus E_{s1} and thickness H is underlain by less rigid infinitely deep material with modulus E_{s2}, then settlement at the centre of a uniformly loaded circular area placed on the surface of the more rigid soil is corrected with the factor α, Table 7-8b.

Table 7-8 Factors for Estimating Immediate Settlement in Cohesive Soil[188]

a. Shape and Rigidity Factor I for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space[189]

Shape	*Centre – Centre*	*Corner – Corner*	*Middle Short Side*	*Middle Long Side*
Circle	1.00	0.64	0.64	0.64
Rigid Circle	0.79			
Square	1.12	0.56	0.76	0.76
Rigid Square	0.99			
Rectangle: Length/Width				
1.5	1.36	0.67	0.89	0.97
2	1.52	0.76	0.98	1.12
3	1.78	0.88	1.11	1.35
5	2.10	1.05	1.27	1.68
10	2.53	1.26	1.49	2.12
100	4.00	2.00	2.20	3.60
1000	5.47	2.75	2.94	5.03
10000	6.90	3.50	3.70	6.50

b. Correction Factor α at the Centre of a Circular Uniformly Loaded Area of Width B on an Elastic Layer of Modulus E_{s1} of Depth H Underlain by a Less Stiff Elastic Material of Modulus E_{s2} of Infinite Depth

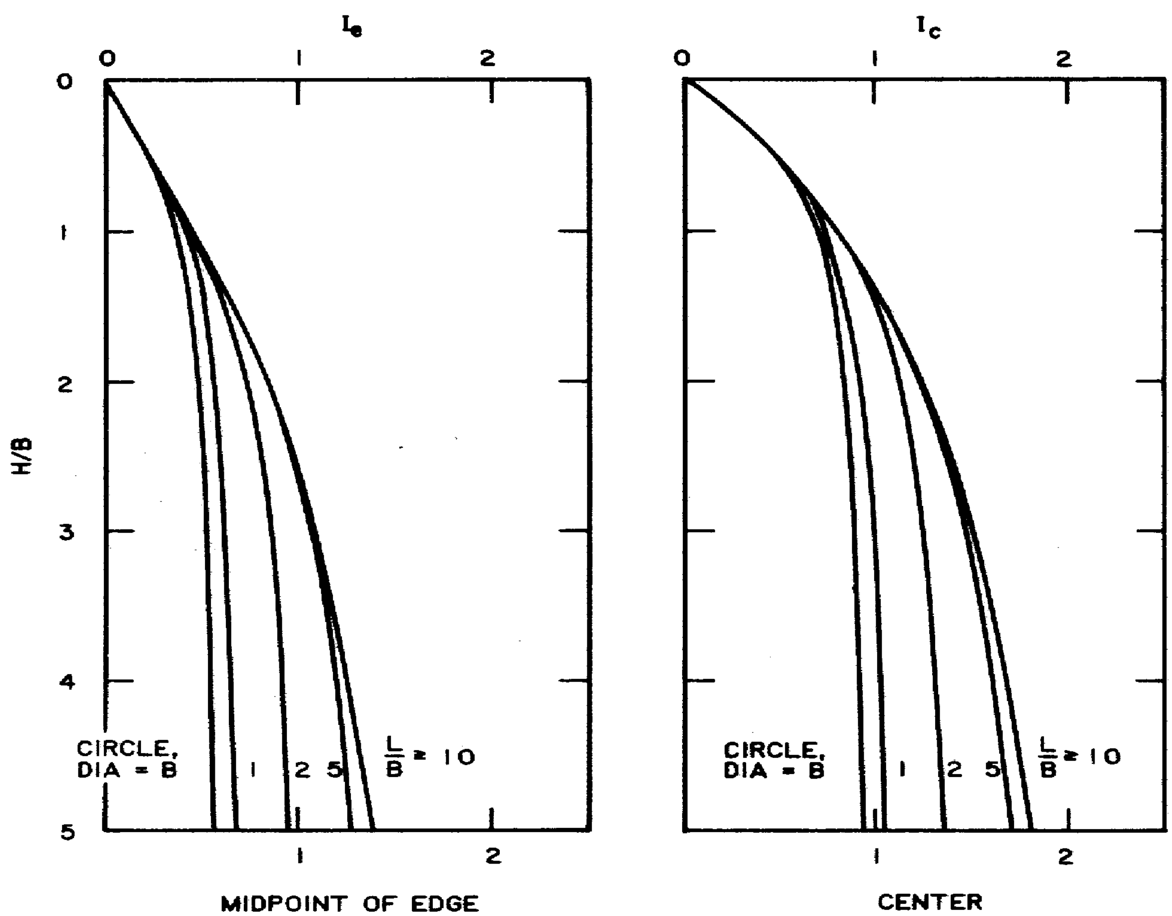
E_{s1}/E_{s2}

[188] Reprinted from D. M. Burmister 1965, "Influence Diagrams for Stresses and Displacements in a Two-Layer Pavement System for Airfields", Contract NBY 13009, Department of the Navy, Washington, D. C. (item 7)

[189] Perloff, W. H. 1975. "Pressure Distribution and Settlement," Chapter 4," Foundation Engineering Handbook, pp 148-196, H. F. Winterkorn and H. Y. Fang, ed. Available from Van Nostrand Reinhold Company, New York, NY 10020.

H/B	*1*	*2*	*5*	*10*	*100*
0	1.000	1.000	1.000	1.000	1.000
0.1	1.000	0.972	0.943	0.923	0.760
0.25	1.000	0.885	0.779	0.699	0.431
0.5	1.000	0.747	0.566	0.463	0.228
1.0	1.000	0.627	0.399	0.287	0.121
2.5	1.000	0.550	0.274	0.175	0.058
5	1.000	0.525	0.238	0.136	0.036
∞	1.000	0.500	0.200	0.100	0.010

Figure 7-11 Influence factor I for settlement of a completely flexible mat or footing of width B and length L on a finite elastic material of depth H supported on a rigid base[190]



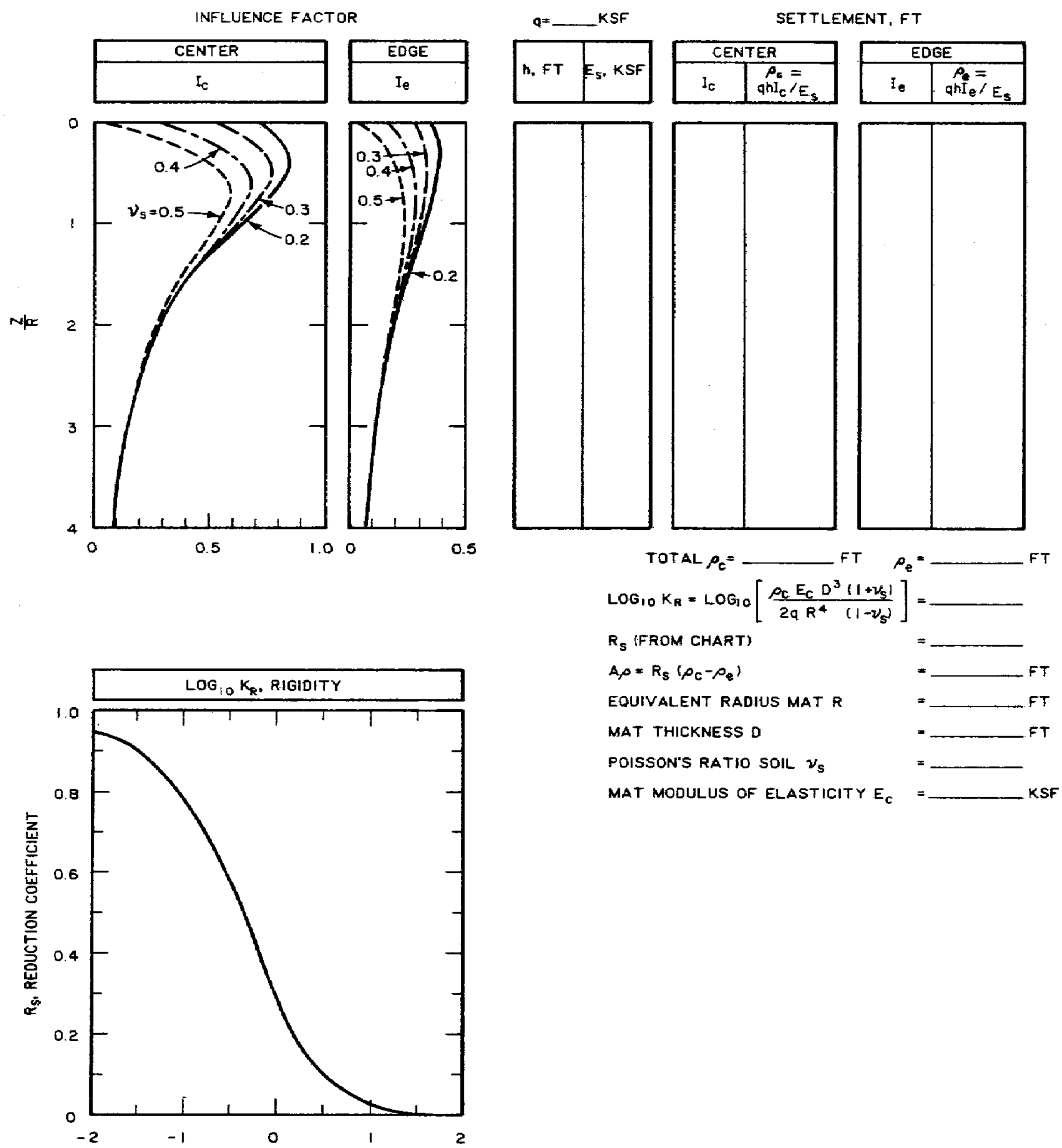
[190] Data taken with permission of McGraw-Hill Book Company from Tables 2-4 and 2-5, Foundations of Theoretical Soil Mechanics, by M. E. Harr, 1966, p 98, 99.

7.3.4.2.3. Kay and Cavagnaro Approximation

The immediate elastic settlement at the centre and edge of circular foundations and foundations with length to width ratios less than two may be evaluated for layered elastic soil by the graphical procedure, Figure 7-12[191]. The method considers the relative rigidity of the foundation relative to the soil and can evaluate the differential displacement between the centre and edge of the foundation.

191 Kay, J. N. and Cavagnaro, R. L. 1983. "Settlement of Raft Foundations," Journal of the geotechnical engineering Division, Vol 109, pp 1367-1382. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Figure 7-12 Computation of elastic settlement beneath a mat foundation



- z = depth beneath mat, ft;
- R = equivalent radius, ft.

7.3.4.3. Recommendations

7.3.4.3.1. Janbu Approximation

The Janbu approximation is recommended when an average computation of settlement is required for a wide range of depths, lengths, and widths of foundations supported on compressible soil of depth H.

7.3.4.3.2. *Perloff Approximation*

The Perloff approximation should be used when total and differential settlement is required beneath flexible foundations located at or near the surface of the soil; settlements may be evaluated at the centre, corner, and middle edges of both the short and long sides of the foundation.

7.3.4.3.3. *Kay and Cavagnaro Approximation*

The Kay and Cavagnaro approximation should be used when total and differential settlement is required beneath footings and mats of a given stiffness supported on compressible soil of variable elastic modulus; settlement may be evaluated at the centre and edge for a given foundation depth. A reasonable estimate of Poisson's ratio for cohesive soil is 0.4.

7.3.4.3.4. *Linear Modulus Increase*

The Gibson model described in 3.3 may be used if the elastic modulus may be assumed zero at the ground surface. A parametric analysis using the Kay and Cavagnaro graphical procedure for an elastic modulus increasing linearly with depth indicates that the centre settlement beneath a foundation may be calculated by

Equation 7-25: $$\rho_c = \frac{q}{k}\left[0.7 + (2.3 - 4v_s)\log_{10} n\right]$$

Where

- $n = kR/(E_o + kD_b)$
- k = constant relating the elastic modulus with depth; i.e., $E_o = E_s + kz$, ksf/ft
- R = equivalent radius of the mat or footing, LB/π
- E_o = elastic soil modulus at the ground surface, ksf
- D_b = depth of the mat base or stiffening beams beneath the ground surface, ft

Edge and corner settlement of a flexible mat or footing will be approximately 1/2 and 1/4 of the centre settlement, respectively. Differential movement of the mat or footing may be calculated from Figure 7-12.

7.3.4.4. Application

A footing 10 ft square, 1 ft thick with base 3 ft below ground surface, is to be constructed on cohesive soil. The pressure applied on the footing is q = 2 tsf (4 ksf). The equivalent elastic modulus of this clay, which is 10 ft deep beneath the footing, is 175 tsf (350 ksf) and Poisson's ratio is 0.4. Table 7-9 compares settlement computed by the improved Janbu and Perloff methods. Refer to Figure 7-13 for application of the Kay and Cavagnaro method.

Table 7-9 Estimation of Immediate Settlement for Example Application in Cohesive Soil

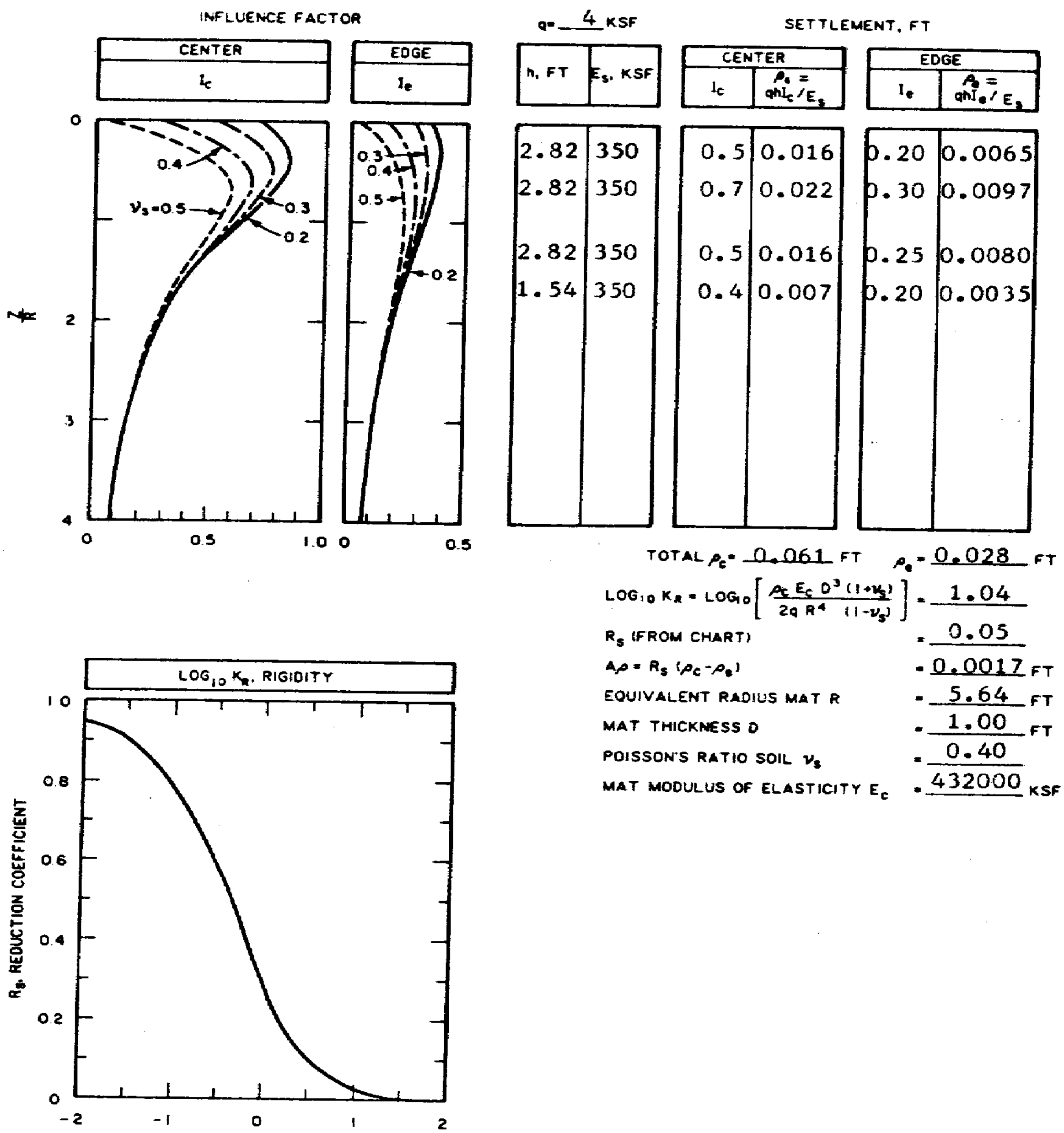
Method	*Equation*	*Immediate Settlement, ρ_i*
Janbu	Equation 7-23	D/B = 0.3, H/B = L/B = 1.0
		$\mu_o = 1.00$, $\mu_1 = 0.35$
		$\rho_i = (1.00)(0.35)(2.0)(10)/(175)$
		= 0.040 ft or 0.48 inch
Perloff	Equation 7-24	From Fig. 3-9, $I_e = 0.4$, $I_c = 0.7$

7-24

Edge: $\rho_i = (0.4)(2)(10)(1-0.16)/175 = 0.038' = 0.46''$

Centre: $\rho_i = (0.7)(2.0)(10)(1-0.16)/175 = 0.067' = 0.81''$

Figure 7-13 Estimation of immediate settlement for the example problem by the Kay and Cavagnaro method



a. Average settlement by the improved Janbu method is 0.48 inch.

b. The Kay and Cavagnaro method in Figure 3-11 calculates smaller edge settlement of 0.33 inch compared with 0.46 inch and smaller centre settlement of 0.73 inch compared with 0.81 inch calculated from the Perloff method. Actual differential settlement when considering stiffness of the footing is only about 0.02 inch, Figure 7-13; the footing is essentially rigid. Settlement will be less than 1 inch and expected to be about 0.5 inch.

7.3.5. Primary Consolidation Settlement

7.3.5.1. Description

Vertical pressure σ_{st} from foundation loads transmitted to a saturated compressible soil mass is initially carried by fluid or water in the pores because water is relatively incompressible compared with that of the soil structure. The pore water pressure u_{we} induced in the soil by the foundation loads is initially equal to the vertical pressure σ_{st} and it is defined as excess pore water pressure because this pressure exceeds that caused by the weight of water in the pores. Primary consolidation begins when water starts to drain from the pores. The excess pressure and its gradient decrease with time as water drains from the soil causing the load to be gradually carried by the soil skeleton. This load transfer is accompanied by a decrease in volume of the soil mass equal to the volume of water drained from the soil. Primary consolidation is complete when all excess pressure has dissipated so that $u_{we} = 0$ and the increase in effective vertical stress in the soil $\Delta\sigma' = \sigma_{st}$. Primary consolidation settlement is usually determined from results of one-dimensional (one dimensional) consolidometer tests. Refer to 7.7 for a description of one-dimensional consolidometer tests.

7.3.5.1.1. Normally Consolidated Soil

A normally consolidated soil is a soil which is subject to an *in situ* effective vertical overburden stress σ'_o equal to the preconsolidation stress σ'_p. Virgin consolidation settlement for applied stresses exceeding σ'_p can be significant in soft and compressible soil with a skeleton of low elastic modulus such as plastic CH and CL clays, silts, and organic MH and ML soils.

7.3.5.1.2. Overconsolidated Soil

An overconsolidated soil is a soil which is subject to an *in situ* effective overburden stress σ'_o less than σ'_p. Consolidation settlement will be limited to recompression from stresses applied to the soil up to σ'_p. Recompression settlement is usually much less than virgin consolidation settlement caused by applied stresses exceeding σ'_p.

7.3.5.2. Ultimate one dimensional Consolidation

The ultimate or long-term one-dimensional consolidation settlement is initially determined followed by adjustment for overconsolidation effects. Refer to Table 7-10 for the general procedure to determine ultimate settlement by primary consolidation.

Table 7-10 Procedure for Calculation of Ultimate Primary Consolidation Settlement of a Compressible Stratum

Step	*Description*
1	Evaluate the preconsolidation stress σ'_p from results of a one-dimensional consolidation test on undisturbed soil specimens using the Casagrande construction procedure, Table 7-11a, or by methods in 7.1.2.1. Refer to 7.7 for a description of one-dimensional consolidation tests.
2	Estimate the average initial effective overburden pressure σ'_o in each compressible stratum using soil unit weights, depth of overburden on the compressible stratum, and the known groundwater

	level or given initial pore water pressure in the stratum. Refer to Equation 5-1, $\sigma'_{oz} = \gamma z - u_w$. $\sigma'_o = (\sigma'_{oz1} + \sigma'_{oz2})/2$ where σ'_{oz1} = effective pressure at top of compressible stratum and σ'_{oz2} = effective pressure at bottom of compressible stratum.
3	Determine the soil initial void ratio e_o as part of the one dimensional consolidation test or by methods in 3.4.2.
4	Evaluate the compression index C_c from results of a one dimensional consolidation test using the slope of the field virgin consolidation line determined by the procedure in Table 7-11a as illustrated in Figure 7-14 and Figure 7-15, or preliminary estimates may be made from Table 7-12. Determine the recompression index C_r for an overconsolidated soil as illustrated in Figure 7-14 and Figure 7-15; preliminary estimates may be made from Figure 7-16.
5	Estimate the final applied effective pressure σ'_f where $\sigma'_f = \sigma'_o + \sigma_{st}$. σ_{st}, soil pressure caused by the structure, may be found from methods described in § 5.
6	Determine the change in void ratio Δe_j of stratum j for the pressure increment $\sigma'_f - \sigma'_o$ graphically from a data plot similar to Figure 7-14, from Equation 7-27 for a normally consolidated soil, or from Equation 7-29 for an overconsolidated soil.
7	Determine the ultimate one-dimensional consolidation settlement of stratum j with thickness H_j, from Equation 7-26.
8	Determine the total consolidation ρ_c of the entire profile of compressible soil from the sum of the settlement of each stratum, Equation 7-28.
9	Correct ρ_c for effect of overconsolidation and small departures from one-dimensional compression on the initial excess pore pressure using the Skempton and Bjerrum procedure, Equation 7-30 where λ is found from Figure 7-17. $\lambda = 1$ if B/H > 4 or if depth to the compressible stratum is > 2B. The equivalent dimension of the structure when corrected to the top of the compressible stratum B_{cor} is found by the approximate distribution $B_{cor} = (B'L')^{0.5}$ where B' = B + z and L' = L + z, B = foundation width, L = foundation length and z = depth to top of the compressible soil profile. Substitute B_{cor} for B in Figure 7-17. $\rho_{\lambda c}$ is the corrected consolidation settlement. This correction should not be applied to bonded clays.

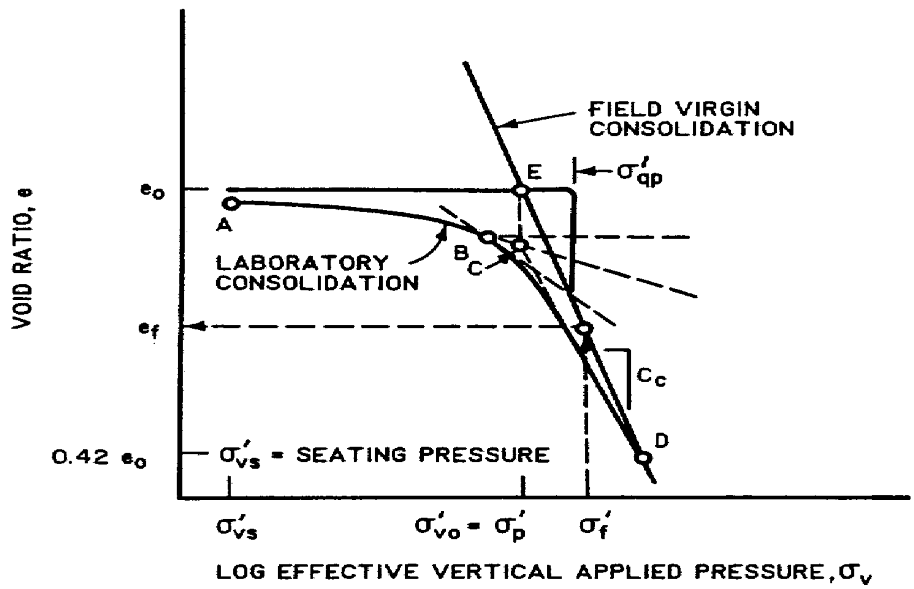
Table 7-11 Reconstruction of Virgin Field Consolidation[192]

Step	*Description*
	Normally Consolidated Soil (Figure 3-12a)
1	Plot point B at the point of maximum radius of curvature of the laboratory consolidation curve.
2	Plot point C by the Casagrande construction procedure: (1) Draw a horizontal line from B; (2) Draw a line tangent to the laboratory consolidation curve through B ; and (3) Draw the bisector between horizontal and tangent lines. Point C is the intersection of the straight portion of the laboratory curve with the bisector. Point C indicates the maximum past pressure σ'_p.
3	Plot point E at the intersection e_o and σ'_p. e_o is given as the initial void ratio prior to testing in the consolidometer and σ'_p is found from step 2.
4	Plot point D at the intersection of the laboratory virgin consolidation curve with void ratio e = 0.42e

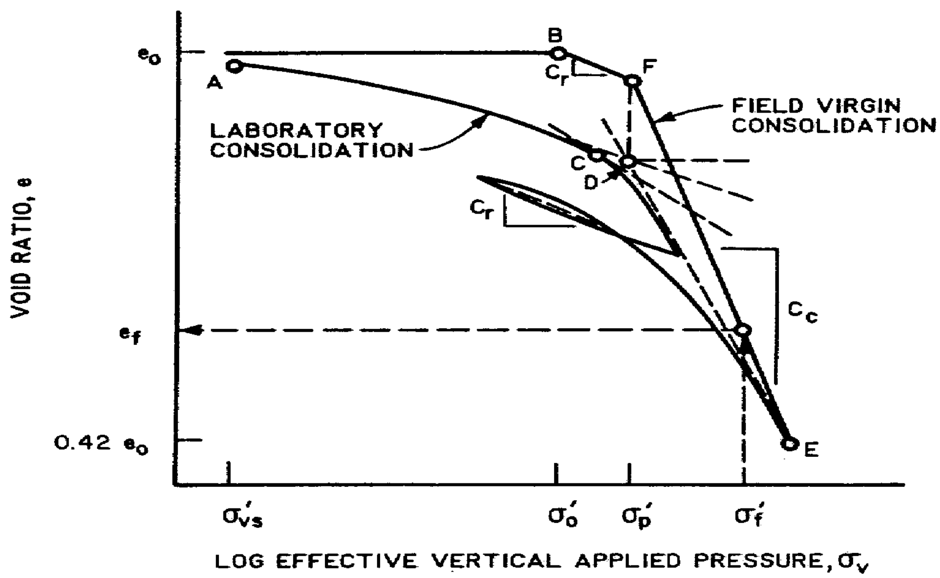
192 Schmertmann, J. H. 1955. "The Undisturbed Consolidation Behavior of Clay," Transactions, Vol 120, pp 1201-1233. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

	$0.42e_o$.
5	The field virgin consolidation curve is the straight line determined by points E and D.
	Overconsolidated Soil (Figure 3-12b)
1	Plot point B at the intersection of the given e_o and the initial estimated *in situ* effective overburden pressure σ'_o.
2	Draw a line through B parallel to the mean slope C_r of the rebound laboratory curve.
3	Plot point D using step 2 in Table 3-6a above for normally consolidated soil.
4	Plot point F by extending a vertical line through D up through the intersection of the line of slope C_r extending through B.
5	Plot point E at the intersection of the laboratory virgin consolidation curve with void ratio e = $0.42e_o$.
6	The field virgin consolidation curve is the straight line through points F and E.

Figure 7-14 Construction of field virgin consolidation relationships



a. NORMALLY CONSOLIDATED SOIL



b. OVERCONSOLIDATED SOIL

Figure 7-15 Example void ratio - logarithm pressure relationship

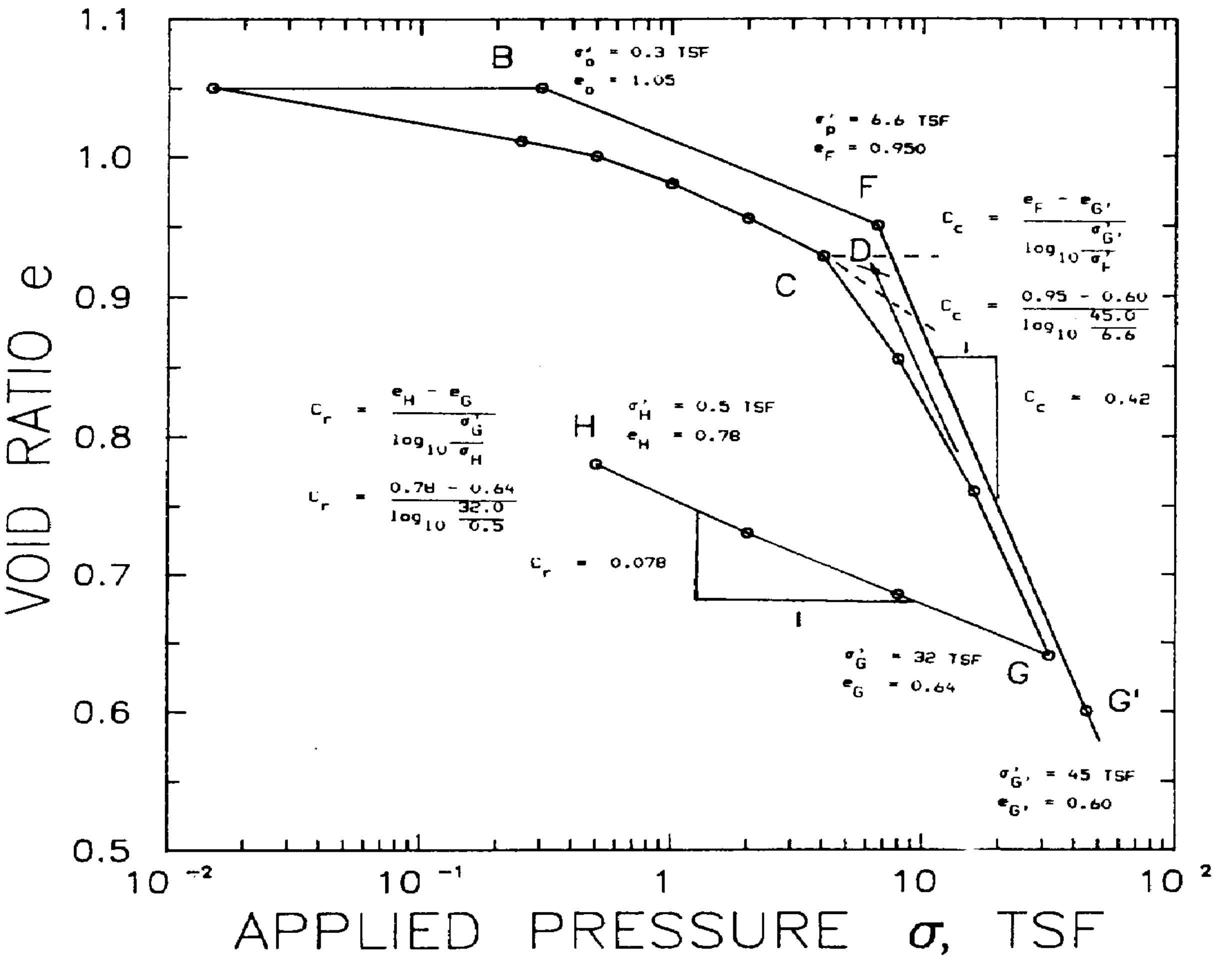


Table 7-12 Estimates of the Virgin Compression Index, C_c[193]

Soil	***C_c***
Inorganic soils with sensitivity less than 4	$C_c = 0.009(LL - 10)$
Organic soils, peat	$0.0115W_n$
Clays	$1.15(e_o - 0.35)$
	$0.012W_n$
	$0.01(LL - 13)$
Varved clays	$(1 + e_o)\,[0.1 + 0.006(W_n - 25)]$
Uniform silts	0.20
Uniform sand, load range of 2 to 8 ksf	
Loose	0.05 to 0.06

[193] Note: LL = liquid limit, percent, W_n = natural water content, percent, e_o = initial void ratio.

Dense	0.02 to 0.03

Figure 7-16 Approximate correlations for the swelling index of silts and clays

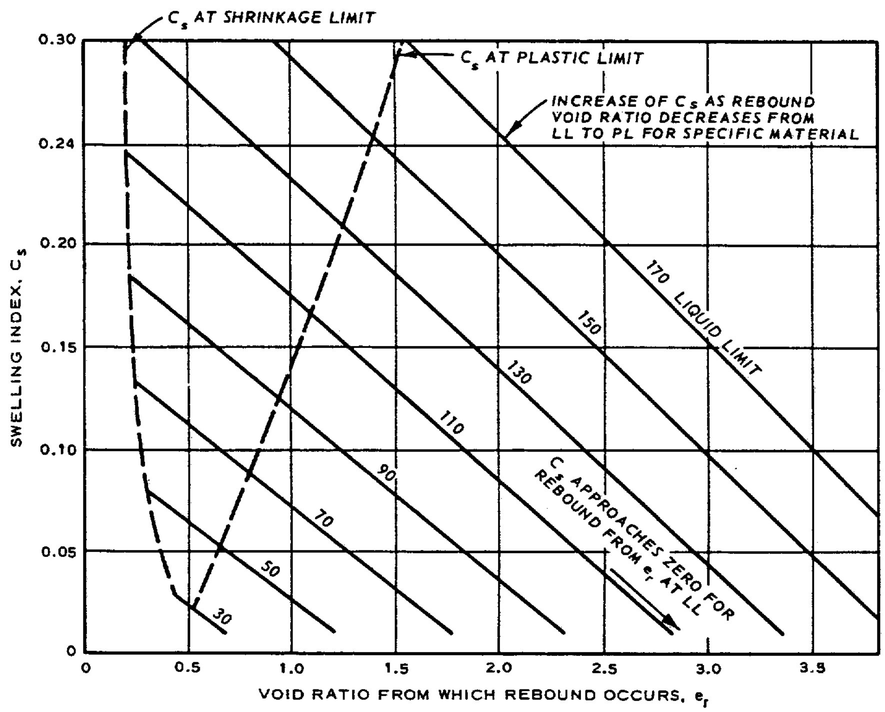
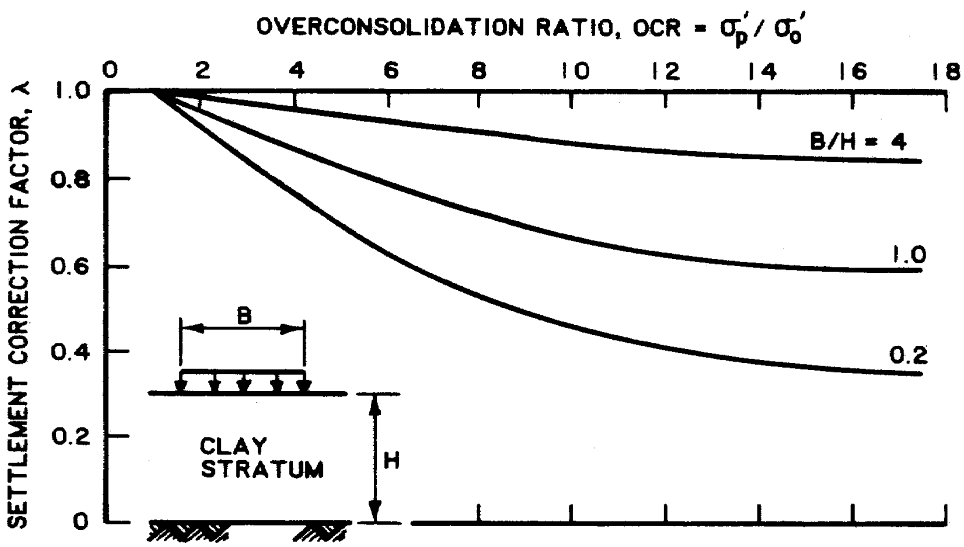


Figure 7-17 Settlement correction factor for overconsolidation effects[194]



[194] Reprinted by permission of the Transportation Research Board, National Research Council from Special Report 163, 1976, "Estimating Consolidation Settlement of Shallow Foundations On Overconcolidated Clay," by G.A. Leonards, p. 15.

7.3.5.2.1. Evaluation of Void Ratio-Pressure Relationship

Estimates of the ultimate consolidation settlement following complete dissipation of hydrostatic excess pressure requires determination of the relationship between the *in situ* void ratio and effective vertical stress in the soil. The loading history of a test specimen taken from an undisturbed and saturated soil sample, for example, may be characterized by a void ratio versus logarithm pressure diagram, Figure 7-14.

7.3.5.2.1.1. Correction of laboratory consolidation curve

Removal of an impervious soil sample from its field location will reduce the confining pressure, but tendency of the sample to expand is restricted by the decrease in pore water pressure. The void ratio will tend to remain constant at constant water content because the decrease in confinement is approximately balanced by the decrease in water pressure; therefore, the effective stress remains constant in theory after Equation 5-1 and the void ratio should not change. Classical consolidation assumes that elastic expansion is negligible and the effective stress is constant during release of the *in situ* confining pressure after the sample is taken from the field. Some sample disturbance occurs, however, so that the laboratory consolidation curve must be corrected as shown in Figure 7-14. Perfectly undisturbed soil should indicate a consolidation curve similar to line e_oED, Figure 7-14a, or line e_oBFE, Figure 7-14b. Soil disturbance increases the slope for stresses less than the preconsolidation stress illustrated by the laboratory consolidation curves in Figure 7-14. Pushing undisturbed samples into metal Shelby tubes and testing in the consolidometer without removing the horizontal restraint helps maintain the *in situ* horizontal confining pressure, reduces any potential volume change following removal from the field, and helps reduce the correction for sample disturbance.

7.3.5.2.1.2. Normally consolidated soil

A normally consolidated soil *in situ* will be at void ratio e_o and effective overburden pressure σ'_o equal to the preconsolidation stress σ'_p. e_o may be estimated as the initial void ratio prior to the test if the water content of the sample had not changed during storage and soil expansion is negligible. *In situ* settlement from applied loads is determined from the field virgin consolidation curve.

a. Reconstruction of the field virgin consolidation curve with slope C_c shown in Figure 7-14a may be estimated by the procedure in Table 7-11a. Determining the point of greatest curvature for evaluation of the preconsolidation stress requires care and judgment. Two points may be selected bounding the probable location of maximum curvature to determine a range of probable preconsolidation stress. Higher quality undisturbed specimens assist in reducing the probable range of σ'_p. If σ'_p is greater than σ'_o, then the soil is overconsolidated and the field virgin consolidation curve should be reconstructed by the procedure in Table 7-11b. The scale of the plot may have some influence on evaluation of the parameters.

b. Consolidation settlement may be estimated by

$$\textbf{Equation 7-26: } \rho_{cj} = \frac{\Delta e_j}{1 + e_{cj}} H_j$$

Where

- ρ_{cj} = consolidation settlement of stratum j, ft
- Δe_j = change in void ratio of stratum j, $e_{oj} - e_{fj}$
- e_{oj} = initial void ratio of stratum j at initial pressure σ'_{oj}
- e_{fj} = final void ratio of stratum j at final pressure σ'_{fj}

∞ H_j = height of stratum j, ft

The final void ratio may be found graphically using the final pressure σ'_f illustrated in Figure 7-14a. The change in void ratio may be calculated by

$$\textbf{Equation 7-27: } \Delta e_j = C_c \log_{10} \frac{\sigma'_{rj}}{\sigma'_{oj}}$$

Where C_c is the slope of the field virgin consolidation curve or compression index. Figure 7-15 illustrates evaluation of C from results of a one dimensional consolidation test. Table 7-12 illustrates some empirical correlations of C_c with natural water content, void ratio, and liquid limit.

c. Total consolidation settlement ρ_c of the entire profile of compressible soil may be determined from the sum of the settlement of each stratum

$$\textbf{Equation 7-28: } \rho_c = \sum_{j=1}^{n} \rho_{cj}$$

Where n is the total number of compressible strata. This settlement is considered to include much of the immediate elastic compression settlement ρ_i, Equation 7-3.

7.3.5.2.1.3. Apparent preconsolidation

A presumably normally consolidated soil may exhibit an apparent preconsolidation stress σ'_{qp}, Figure 7-14a. σ'_{qp} may be caused by several mechanisms; for example, the most common cause is secondary compression or the gradual reduction in void ratio (accompanied by an increase in attractive force between particles) at constant effective stress over a long time. Other causes of σ'_{qp} include a change in pore fluid, which causes attractive forces between particles to increase, or cementation due to precipitation of cementatious materials from flowing groundwater. This apparent preconsolidation is sensitive to strain and may not be detected because of sample disturbance. Existence of σ'_{qp} in the field can substantially reduce settlement for a given load and can be used to reduce the factor of safety or permit greater pressures to be placed on the foundation soil, if collapse will not be a problem. Refer to 7.5.3.1 to 7.5.3.4 for estimating potential collapse.

7.3.5.2.1.4. Overconsolidated soil

An overconsolidated soil will be at a void ratio e_o and effective vertical confining pressure σ'_o represented by point B, Figure 7-14b. At some time in the past, the soil was subject to the preconsolidation stress σ'_p, but this pressure was later reduced, perhaps by soil erosion or removal of glacial ice, to the existing overburden pressure σ'_o. The *in situ* settlement for an applied load will be the sum of recompression settlement between points B and F and any virgin consolidation from a final effective vertical applied pressure σ'_f exceeding the preconsolidation stress σ'_p. Reloading a specimen in the consolidometer will give the laboratory curve shown in Figure 7-14b.

a. Reconstruction of the field virgin consolidation curve with slope C_c may be estimated by the procedure in Table 7-11b. Refer to Table 7-12 for methods of estimating C_c.

b. The rebound loop in the laboratory curve is needed to develop the recompression line BF. Evaluation of the recompression index C_r is illustrated in Figure 7-15. The recompression index is equal to or slightly smaller than the swelling index C_s. Approximate correlations of the swelling index are shown in Figure 7-16.

c. Settlement ρ_{cj} of stratum j in inches may be estimated as the sum of recompression and virgin consolidation settlements. The final void ratio is found graphically from Figure 7-14b. The change in void ratio may be calculated by

$$\textbf{Equation 7-29: } \Delta e_j = C_r \log_{10} \frac{\sigma'_{pj}}{\sigma'_{oj}} + C_c \log_{10} \frac{\sigma'_{rj}}{\sigma'_{pj}}$$

where C_r is the average slope of the recompression line BF. If $\sigma'_{fj} < \sigma'_{pj}$, ignore the right-hand term of Equation 7-29 containing C_c and substitute σ'_{fj} for σ'_{pj} in the term containing C_r. Settlement of stratum j is found from Equation 7-26 and ultimate settlement ρ_c of compressible soil in the profile is found from Equation 7-28.

7.3.5.2.1.5. Underconsolidated soil

Occasionally, a compressible soil stratum may be found to have excess hydrostatic pore pressures such as when the stratum had not reached equilibrium pore water pressures under existing overburden pressures or the groundwater level had been lowered. The effective stress will increase as the pore pressures dissipate and cause recompression settlement until the effective stress equals the preconsolidation stress. Virgin consolidation settlement will continue to occur with increasing effective stress until all excess pore pressures are dissipated. If the initial effective stress is less than the preconsolidation stress σ'_p, then the ultimate settlement may be found as for an overconsolidated soil from Equation 7-29 and Equation 7-26. σ'_{oj} is the initial effective stress found from Equation 5-1, the initial total overburden pressure minus the initial total pore water pressure.

σ'_{fj} is the final effective stress found from the final total overburden pressure minus the equilibrium or final pore water pressure. If σ'_{oj} equals σ'_p, then the ultimate settlement may be found as for a normally consolidated soil from Equation 7-27 and Equation 7-26.

7.3.5.2.2. Adjustment for Overconsolidation Effects

The effects of overconsolidation and departure from one dimensional compression on the initial excess pore pressure may require correction to the calculated settlement and rate of settlement. The following semi-empirical procedures have been used to correct for these effects. Numerical methods of analysis offer a rational alternative approach to include 3-D affects, but these have not proved useful in practice.

7.3.5.2.2.1. Skempton and Bjerrum correction

The corrected consolidation settlement $\rho_{\lambda c}$ of a clay stratum is found by

$$\textbf{Equation 7-30: } \rho_{\lambda c} = \lambda \rho_c$$

where λ is the settlement correction factor, Figure 7-17. The equivalent dimension of the loaded area should be corrected to the top of the compressible stratum by the approximate stress distribution method as illustrated in step 9, Table 7-10, or 3.3. The corrected settlement is still assumed one dimensional, although overconsolidation effects are considered. $\lambda = 1$ if $B/H > 4$ or if depth to the compressible stratum is $> 2B$.

7.3.5.2.2.2. Stress path correction

This alternative approach attempts to simulate stress paths that occur in the field, as illustrated in Table 7-13. This procedure may require special laboratory tests using triaxial cells capable of undrained loading followed by consolidation. These tests have not usually been performed and are without standard operating guidelines. Approximations necessary to estimate suitable points in the soil profile for testing and estimates of stresses applied to soil elements at the selected points may introduce errors more significant than the Skempton and Bjerrum correction procedure.

Table 7-13 Summary of the Stress Path Procedure[195]

Step	*Procedure*
1	Select one or more points within the soil profile beneath the proposed structure.
2	Determine initial stresses and pore pressures at the selected points.
3	Estimate for each point the stress path for loading to be imposed by the structure. The stress path usually depends on undrained loading initially, followed by consolidation.
4	Perform laboratory tests which follow the estimated stress paths; duplicate initial stresses, measure strains from undrained loading, then consolidate to the final effective stress σ'_f and measure strains.
5	Use the strains measured to estimate settlement of the proposed structure.

7.3.5.3. Time Rate of Settlement

The solution for time rate of primary consolidation settlement is based on the Terzaghi one dimensional consolidation theory in which settlement as a function of time is given by

$$\textbf{Equation 7-31:}\ \rho_{ct} = \frac{U_r \rho_{ic}}{100}$$

Where

- ρ_{ct} = consolidation settlement at time t, ft
- U_t = degree of consolidation of the compressible stratum at time t, percent
- $\rho_{\lambda c}$ = ultimate consolidation settlement adjusted for overconsolidation effects, ft

Refer to Table 7-14 for the general procedure to determine time rate of settlement from primary consolidation.

Table 7-14 Time Rate of Settlement

Step	*Description*
1	Evaluate lower and upper bound values of the coefficient of consolidation, c_v, of each soil stratum in the profile for each consolidation load increment from deformation-time plots of data from one dimensional consolidometer tests. Plot c_v as a function of the logarithm of applied pressure. Refer to Table 7-16 and Figure 7-19 for methods of calculating c_v.
2	Select appropriate values of c_v from the c_v versus logarithm pressure plots using σ'_f found from step 5, Table 7-10. Preliminary estimates of c_v may be made from Figure 7-20.
3	Select minimum and maximum values of c_v and calculate the effective thickness H' of a multilayer soil profile using the procedures in Table 7-17 relative to one of the soil layers with a given c_{vi}. If the soil profile includes pervious incompressible seams, then evaluate T_v and U_t in steps 4 to 6 for each compressible layer and calculate U_t of the soil profile by step 7.

195 McCarthy, D.F. *Essentials of Soil Mechanics and Foundations*. Sixth Edition. Upper Saddle River, NJ: Prentice Hall, Inc., 2002.

4 Evaluate minimum and maximum time factors T_v of the compressible soil profile from Equation 7-32 for various times t using c_v from step 3 (or c_{vi} for multilayer soil). The equivalent compressible soil height H_e is 1/2 of the actual height (or 1/2 of the effective height H' of multiple soil layers) for double drainage from top and bottom surfaces of the compressible soil and equal to the height of the compressible soil for single drainage.

5 Select the case, Table 7-15 and Figure 7-18 that best represents the initial pore water pressure distribution. If none of the given pressure distributions fit the initial distribution, then approximate the initial distribution as the sum or difference of some combination of the given standard distributions in Table 7-15 as illustrated in Figure 7-21. Note the cases and relative areas of the standard pore water pressure distributions used to approximate the initial distribution.

6 Evaluate minimum and maximum values of the degree of consolidation U_t for given T_v from Table 7-15. If none of the four cases in Table 7-15 model the initial pore pressure distribution, then the overall degree of consolidation may be evaluated by dividing the pore pressure distribution into areas that may be simulated by the cases in Table 7-15 and using Equation 7-37. U_t may also be the degree of consolidation of a soil bounded by internal drainage layers (pervious soil). Omit step 7 if U_t is the degree of consolidation of the soil where pervious seams are not present.

7 Evaluate influence of internal drainage layers (pervious seams) on settlement by, Equation 7-37 where U_t is the degree of consolidation at time t and ρ_c is the ultimate consolidation settlement of the compressible soil profile. Subscripts 1, 2,..., n indicate each compressible layer between seams.

8 Determine the consolidation settlement as a function of time ρ_{ct}, where $\rho_{ct} = U_t \rho_{\lambda c}$, Equation 7-31.

7.3.5.3.1. Evaluation of the Degree of Consolidation

Solution of the Terzaghi consolidation theory to determine U_t is provided in Table 7-15 as a function of time factor T_v for four cases of different distributions of the initial excess pore water pressure. Figure 7-18 illustrates example distributions of the initial excess pore water pressure for single (drainage from one surface) and double (drainage from top and bottom surfaces) drainage.

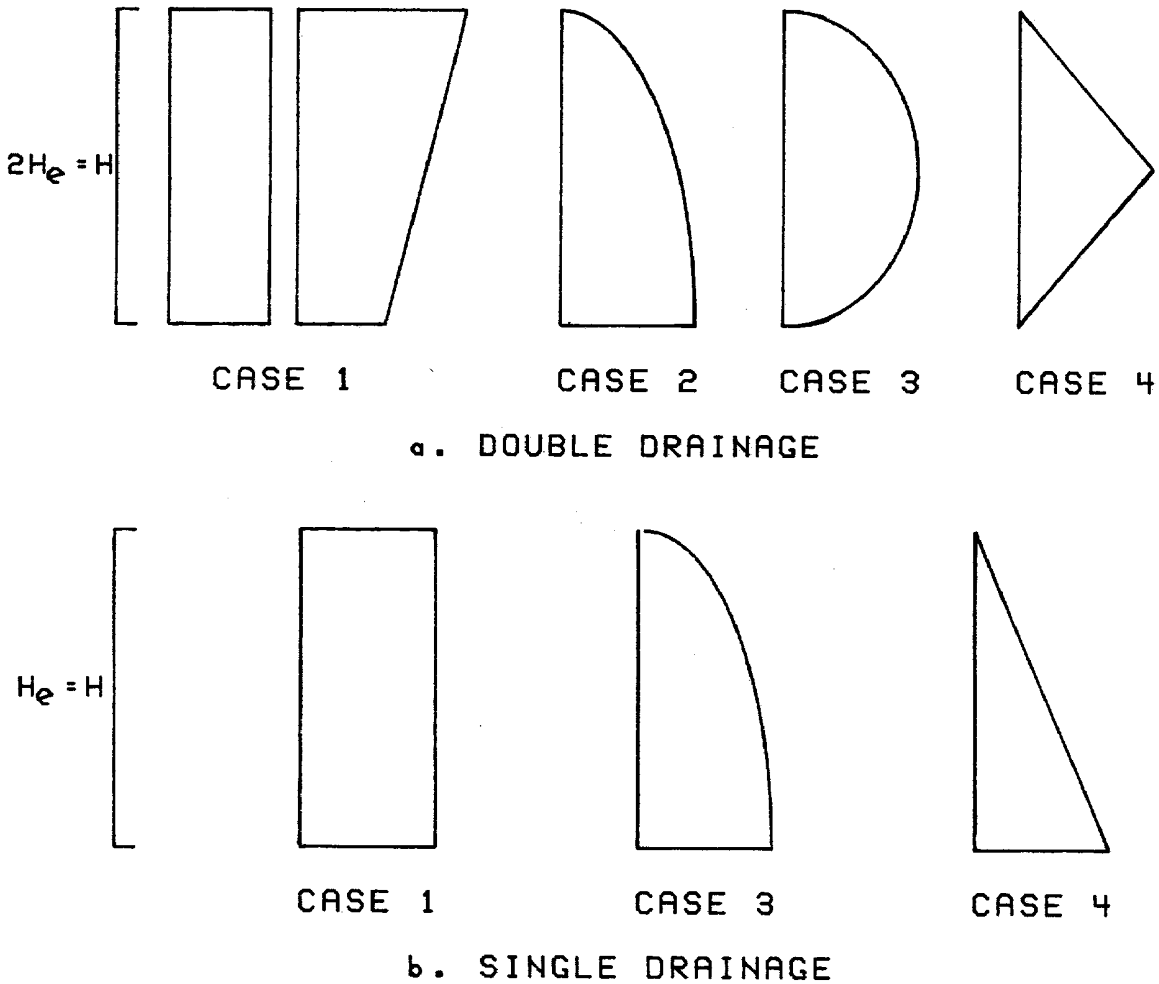
Table 7-15 Degree of Consolidation as a Function of Time Factor T_v

T_v	Average Degree of Consolidation, U_t Percent			
	Case 1[196]	Case 2	Case 3	Case 4
0.004	7.14	6.49	0.98	0.80
0.008	10.09	8.62	1.95	1.60
0.012	12.36	10.49	2.92	2.40
0.020	15.96	13.67	4.81	4.00
0.028	18.88	16.38	6.67	5.60
0.036	21.40	18.76	8.50	7.20
0.048	24.72	21.96	11.17	9.69

[196] See Figure 7-18.

0.060	27.64	24.81	13.76	11.99
0.072	30.28	27.43	16.28	14.36
0.083	32.51	29.67	18.52	16.51
0.100	35.68	32.88	21.87	19.77
0.125	39.89	36.54	26.54	24.42
0.150	43.70	41.12	30.93	28.86
0.175	47.18	44.73	35.07	33.06
0.200	50.41	48.09	38.95	37.04
0.250	56.22	54.17	46.03	44.32
0.300	61.32	59.50	52.30	50.78
0.350	65.82	64.21	57.83	56.49
0.400	69.79	68.36	62.73	61.54
0.500	76.40	76.28	70.88	69.95
0.600	81.56	80.69	77.25	76.52
0.800	88.74	88.21	86.11	85.66
1.000	93.13	92.80	91.52	91.25
1.500	98.00	97.90	97.53	97.45
2.000	99.42	99.39	99.28	99.26

Figure 7-18 Example distributions of excess pore water pressure for double and single drainage. H is the actual stratum thickness and H_e is the equivalent height



7.3.5.3.1.1. Time factor

The time factor is given by

$$\textbf{Equation 7-32: } T_v = \frac{C_v t}{H_e^2}$$

Where

- c_v = coefficient of consolidation of the stratum, ft/day
- H_e = equivalent height of the compressible stratum, ft

The equivalent thickness of a compressible stratum for single drainage (drainage from one boundary) is the actual height of the stratum. H_e is 1/2 of the actual height of the stratum for double drainage (drainage from top and bottom boundaries).

7.3.5.3.1.2. Coefficient of consolidation

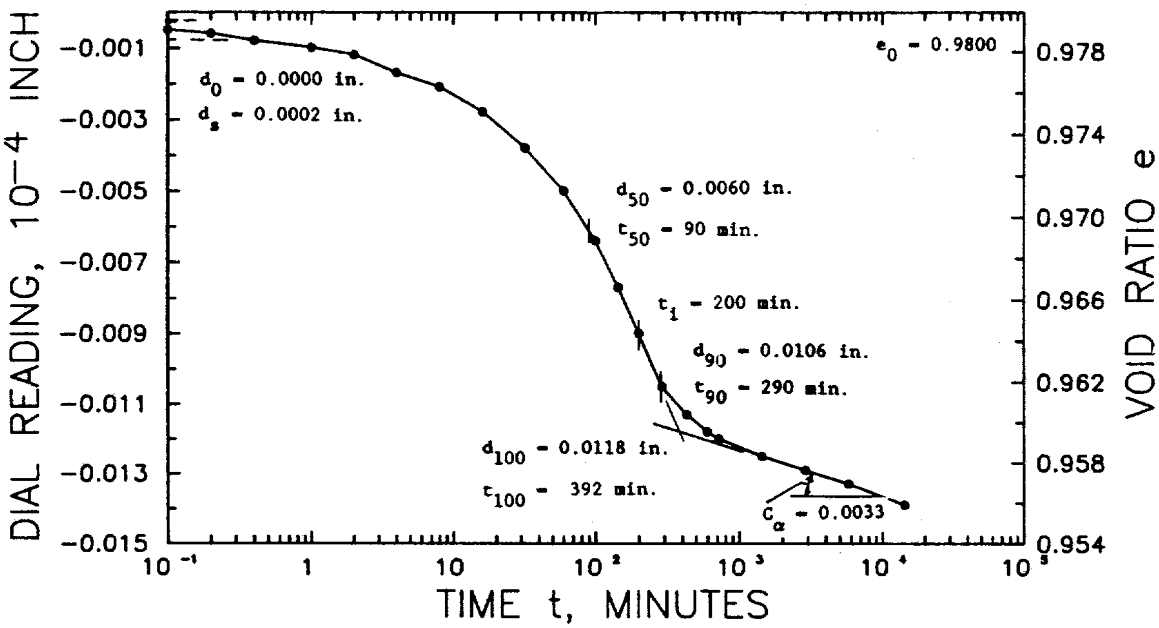
The coefficient of consolidation c_v may be found experimentally from conventional (step load) laboratory one dimensional consolidometer test results by four methods described in Table 7-16, Figure 7-19 and 7.7. Both Casagrande and Taylor methods, Table 7-16, are recommended and they may provide reasonable lower and upper bound values of the coefficient of consolidation. The Casagrande logarithm time method is usually easier to use with the less pervious cohesive soils; whereas, the Taylor square root of time method is easier to use with the more pervious cohesionless soils.

Table 7-16 Evaluation of the Coefficient of Consolidation c_v by the Step Load Test[197]

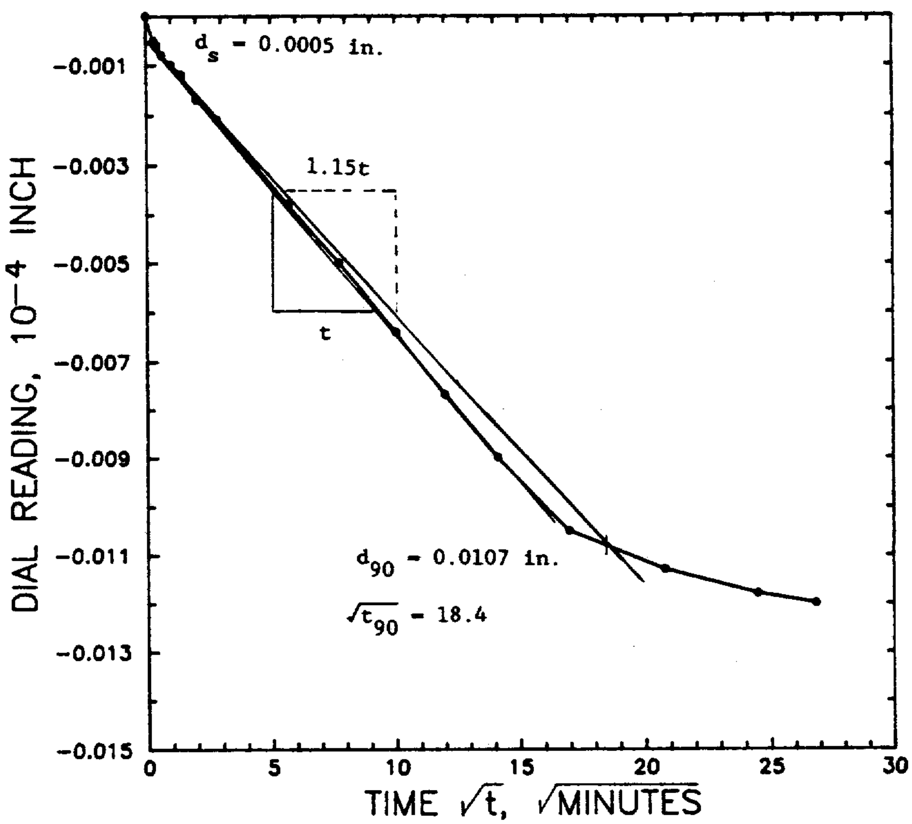
Method	Equation for c_v	Procedure	Example
Terzaghi	$\frac{0.848h_e^2}{t_{90}}$	1. Measure initial specimen height h_o and set initial dial reading d_o.	h_o = 1.148 inches
			d_o = 0.0000 inch
		2. Measure dial reading d as a function of time t and final specimen height h_f. Plot d versus $\log_{10}t$. Determine d_s, the corrected zero point ($d_o - d_s$ = initial compression), by measuring vertical distance between time of about 0.1 min. and a time that is 4 times this.	h_f = 1.140 inches; d_s = 0.0002 inch; Refer to Figure 3-17a
		3. Determine time t_{100} and compression d_{100} to 100 percent consolidation as the intersection of the tangent and asymptote of the consolidation curve. Then determine d_{90} and t_{90}.	t_{100} = 392 min. or 0.27 day; d_{100} = 0.0118 in.; $d_{90} = d_s + 0.9(d_{100} - d_s)$ = 0.0002 + 0.9(0.0118-0.0002) = 0.0002 + 0.0104 = 0.0106 inch.; t_{90} = 290 min. or 0.20 day from Figure 3-17a
		4. Determine equivalent thickness of drainage path, $h_e = (h_o + h_f)/2$ if single drainage; $h_e = (h_o + h_f)/4$ if double drainage	$h_e = (h_o + h_f)/4$ = 2.288/4 = 0.572 inch or 0.0477 ft
		5. Calculate c_v	$c_v = \frac{0.85 \cdot 0.0477^2}{0.20} = 0.010\ ft^2/day$
Casagrande	$\frac{0.197h_e^2}{t_{50}}$	Same as Terzaghi above except determine time to reach 50 percent of consolidation t_{50}	$d_{50} = d_s + 0.5(d_{100} - d_s)$ = 0.0002 + 0.5(0.0118-0.0002) = 0.0002 + 0.0058 = 0.0060 inch; t_{50} = 90 min. or 0.063 day; $c_v = \frac{0.197 \cdot 0.0477}{0.063} = 0.007\ ft^2/day$
Inflection Point	$\frac{0.405h_e^2}{t_i}$	Same as above except note time to reach inflection point of curve t_i	t_i = 200 min. or 0.14 day from Figure 3-17a; $c_v = \frac{0.405 \cdot 0.0477^2}{0.14} = 0.007\ ft^2/day$
Taylor	$\frac{0.848h_e^2}{t_{90}}$	1. Measure initial specimen height h_o and set initial dial reading d_o	h_o = 1.148 inches; d_o = 0.0000 inch
		2. Measure dial reading d as a function of time t and final specimen height h_f. Plot d versus $\sqrt{t}$ or square root of time.	h_f = 1.140; Refer to Figure 3-17b
		3. Extend straight line portion back to $\sqrt{t} = 0$ to obtain corrected initial reading d_o.	d_o = 0.0005 inch from Figure 3-17b
		4. Through d_o draw a straight line with inverse slope 1.15 times tangent and intersect laboratory curve to obtain t_{90}	$\sqrt{t_{90}}$ = 18.4 or t_{90} = 339 min. or 0.23 day; Refer to Figure 3-17b
		5. Determine h_e as above	h_e = 0.0477 ft
		6. Calculate c_v	$c_v = \frac{0.848 \cdot 0.0477^2}{0.23} = 0.008\ ft^2/day$

[197] Winterkorn, H. F. and Fang, Hsai-Yang 1975. "Soil Technology and Engineering Properties of Soils," Chapter 2, Foundation Engineering Handbook, H. F. Winterkorn and H-Y Fang, ed., pp 112-117. Available from Van Nostrand Reinhold Company, 135 West 50th Street, New York, NY 10020.

Figure 7-19 Example time plots from one dimensional consolidometer test, Δσ = 1 TSF



a. VOID RATIO-LOGARITHM TIME



b. DISPLACEMENT-SQUARE ROOT TIME

a) The Casagrande logarithm time method, Figure 7-19a, determines

Equation 7-33: $$c_v = \frac{0.197 h_o^2}{t_{50}}$$

Where

- c_v = coefficient of consolidation of stratum, ft^2/day
- h_e = equivalent specimen thickness, ft
- t_{50} = time at 50% of primary consolidation, days

The equivalent specimen thickness is the actual specimen height for single drainage and 1/2 of the specimen height for double drainage. This method usually provides a low value or slow rate of consolidation.

b) The Taylor square root of time method, Figure 7-19b, determines

Equation 7-34: $$c_v = \frac{0.848 h_o^2}{t_{90}}$$

This method usually calculates a faster rate of consolidation than the Casagrande method and may better simulate field conditions.

c) c_v should be plotted as a function of the applied consolidation pressure. An appropriate value of c_v can be selected based on the final effective pressure σ'_f of the soil for a specific case.

d) Figure 7-20 illustrates empirical correlations of the coefficient of consolidation with the liquid limit.

e) The procedure shown in Table 7-17 should be used to transform a compressible soil profile with variable coefficients of consolidation to a stratum of equivalent thickness H' and coefficient of consolidation c_v. T_v may be calculated from Equation 7-32 with H_e = H'. Refer to 7.3.5.3.3, "Internal Drainage Layers", to estimate U_t of a soil profile with pervious incompressible sand seams interspersed between compressible soils.

Figure 7-20 Correlations between Coefficient of Consolidation and Liquid Limit

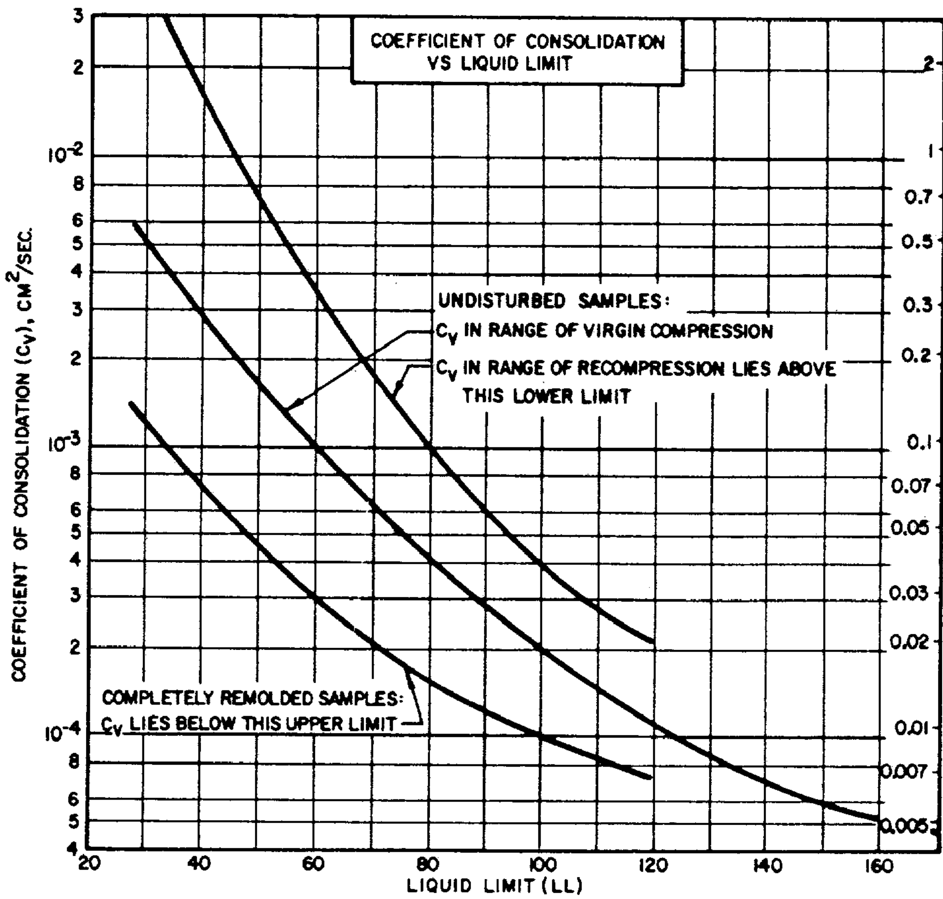


Table 7-17 Procedures to Evaluate the Effective Thickness and Average Degree of Consolidation for Multiple Soil Layers

For a soil system containing n layers with properties C_{vi}, (coefficient of consolidation) and H_i, (layer thickness), convert the system to one equivalent layer with equivalent properties, using the following procedure:

1. Select any layer i, with properties c_v, = c_{vi}, H = H_i.
2. Transform the thickness of every other layer to an equivalent thickness of a layer possessing the soil properties of layer i, as follows:

$$H_1' = H_1\sqrt{\frac{c_{vi}}{c_{v1}}}$$

Equation 7-35: $$H_2' = H_2\sqrt{\frac{c_{vi}}{c_{v2}}}$$

$$H_n' = H_n\sqrt{\frac{c_{vi}}{c_{vn}}}$$

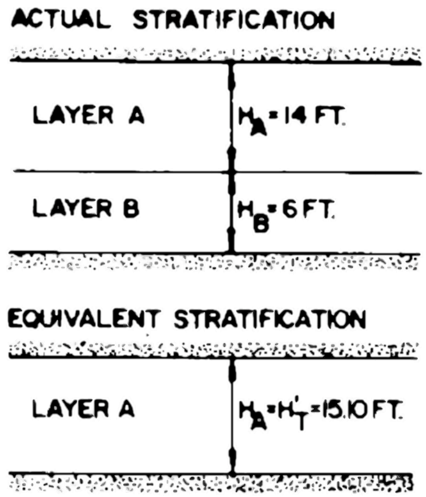
3. Calculate the total thickness of the equivalent layer:

Equation 7-36 $H'_T = H'_1 + H'_2 + \ldots + H'_i + \ldots + H'_n$

4. Treat the system as a single layer of thickness H'+T,, possessing a coefficient of consolidation c_v, = c_{vi}.
5. Determine values of percent consolidation (U) at various times (t) for total thickness (H'_T).

EXAMPLE OF COMPUTATION OF RATE OF CONSOLIDATION FOR A MULTI-LAYERED SYSTEM:

LAYERED SYSTEM:



KNOWN APPLICABLE SOIL PROPERTIES:

LAYER A:

$c_{vA} = 0.04$ FT.^{2}/DAY

LAYER B:

$c_{vB} = 1.20$ FT.^{2}/DAY

ASSUME: DOUBLE DRAINAGE

DETERMINATION OF EQUIVALENT LAYER THICKNESS:

1. ASSUME AN EQUIVALENT LAYER POSSESSING THE PROPERTIES OF SOIL A.
2. EQUIVALENT THICKNESS

$$H'_T = H_A + H'_B = H_A + H_B \left(\frac{c_{vA}}{c_{vB}}\right)^{1/2} = 14 + 6 \left(\frac{0.04}{1.20}\right)^{1/2} = 14 + 1.10$$

$$H'_T = 15.10 \text{ FT.}$$

3. DETERMINE $\bar{U}$ FROM FIGURE II, e.g. AT 1 = 0.25 YEARS, USING H = (15.10)/2 = 7.55 FT. (DRAINAGE PATH ASSUMING DOUBLE DRAINAGE) AND c_{vA} = 0.04 FT.^{2}/DAY, $\bar{U}$ = 29.1 %

7.3.5.3.2. Superposition of Excess Pore Pressure Distribution

An initial pore pressure distribution that is not modeled by any of the four cases in Table 7-15 and Figure 7-18 may sometimes be approximated by superposition of any of the four cases and the overall or weighted degree of consolidation found by

Equation 7-37: $U_t = \dfrac{U_{t1}A_1 + U_{t2}A_2 \ldots U_{ti}A_i}{A}$

Where

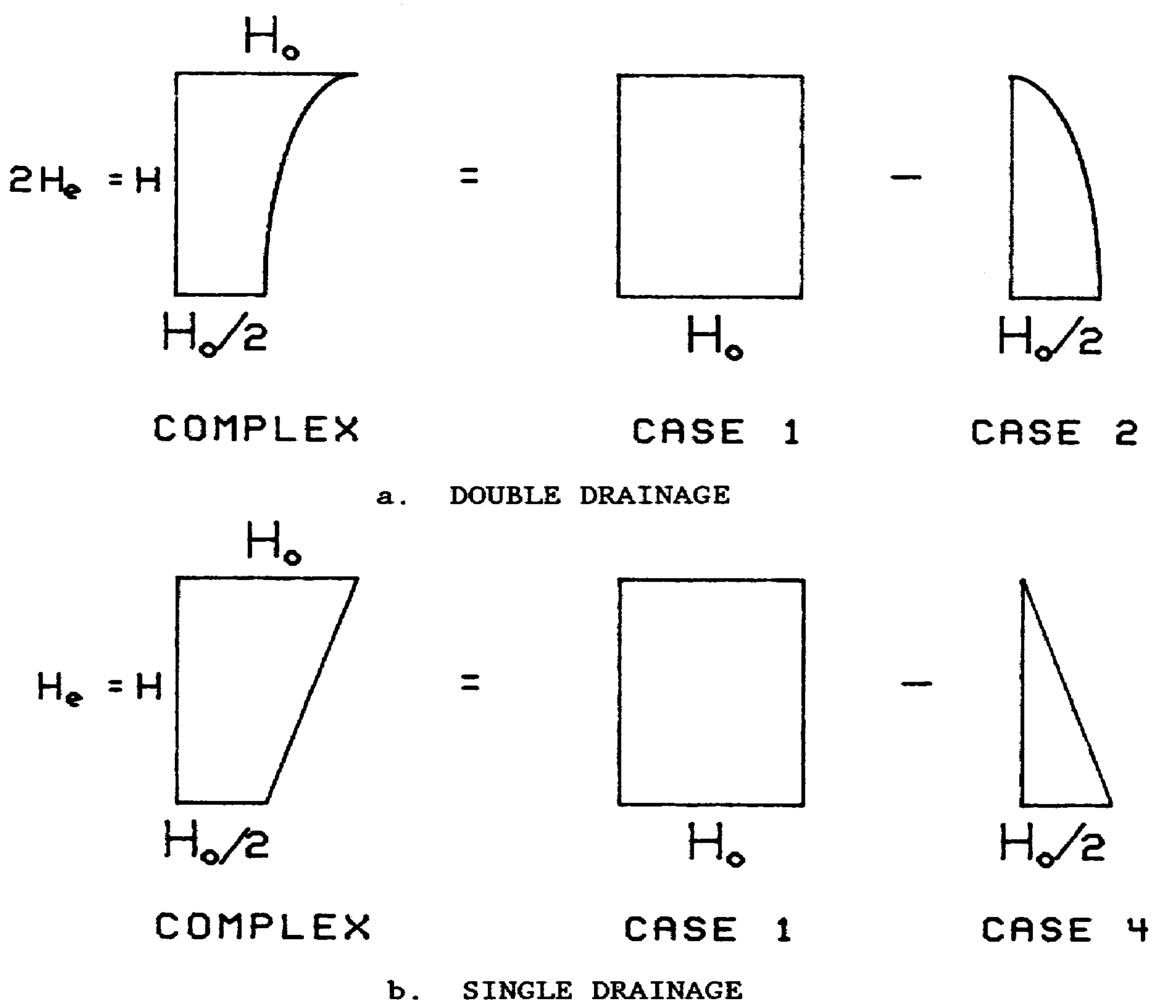
- U_{ti} = degree of consolidation of case i, i = 1 to 4
- A_i = area of pore pressure distribution of case I
- A = area of approximated pore pressure distribution

Subscripts 1, 2,..., i indicate each pore pressure distribution. Linearity of the differential equations describing consolidation permits this assumption.

7.3.5.3.2.1. Example excess pore water pressure distributions

Some example complex excess pore water pressure distributions are shown in Figure 7-21.

Figure 7-21 Example complex excess pore water pressure distributions



7.3.5.3.2.2. Application

For single drainage, a decreasing excess pore pressure distribution may be modeled as illustrated in Figure 7-21b. If $T_v = 0.2$, the degree of consolidation is 50.41 and 37.04% for cases 1 and 4, respectively, Table 7-15. The overall degree of consolidation from Equation 7-33 for the example in Figure 7-21b is

$$U_t = \frac{50.41 H_e H_o - (37.04)(0.5) H_e \dfrac{H_o}{2}}{H_e H_o - (0.5) H_e \dfrac{H_o}{2}}$$

$$U_t = \frac{(50.41)(1.0) - 37.04(0.25)}{1.0 - 0.25} = 54.87\%$$

The total area of the complex pore pressure distribution equals the area of case 1 less area of case 4, Figure 7-21b.

7.3.5.3.3. Internal Drainage Layers

Internal drainage layers of pervious soil within an otherwise low permeable clay stratum will influence the rate of settlement. This influence can be considered by summation of the degrees of consolidation of each compressible layer between the pervious seams by

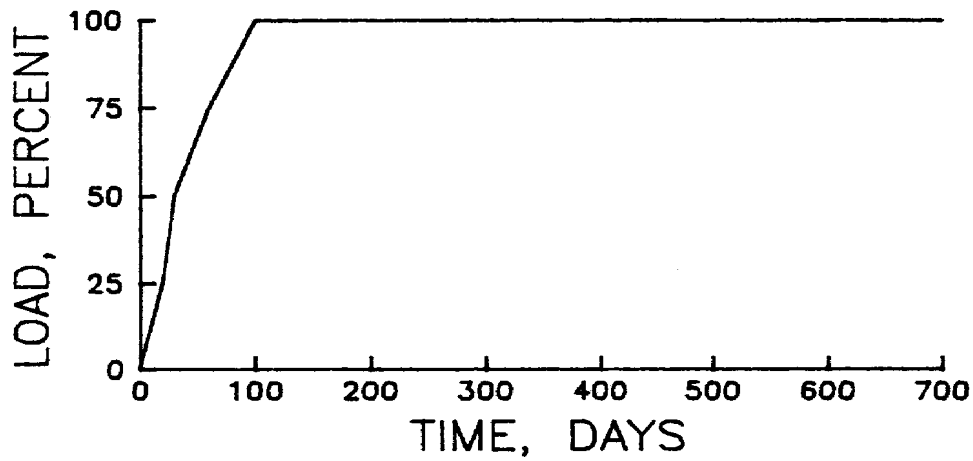
Equation 7-38: $$U_t = \frac{1}{\rho_c}\left(U_{t1}\rho_{c1} + U_{t2}\rho_{c2} + \ldots + U_{tn}\rho_{cn}\right)$$

where U_t is the degree of consolidation at time t and ρ_c is the ultimate consolidation settlement of the entire compressible stratum. The subscripts 1, 2,..., n indicate each compressible layer between pervious seams.

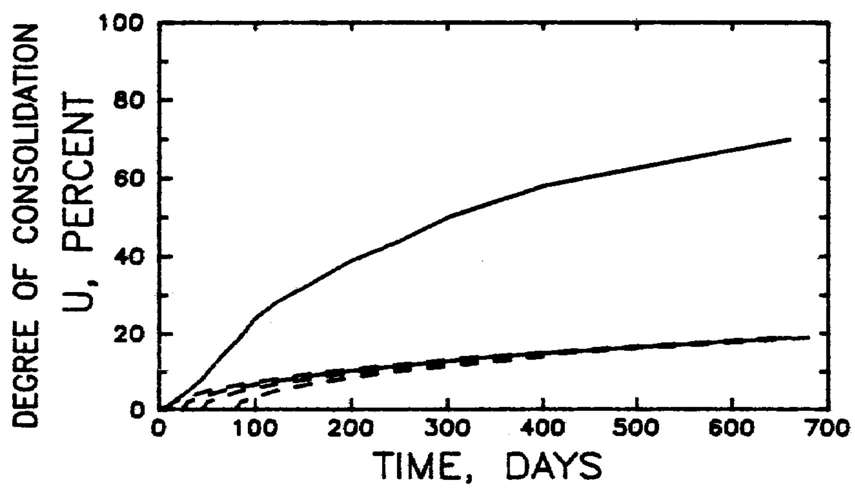
7.3.5.3.4. Time-Dependent Loading

The rate of load application to foundation soils is usually time-dependent. Estimates of the degree of consolidation of time-dependent loads may be made by dividing the total load into several equal and convenient increments such as the 25% increments illustrated in Figure 7-22. Each increment is assumed to be placed instantaneously at a time equal to the average of the starting and completion times of the placement of the load increment. The degree of consolidation U of the underlying compressible soil is evaluated for each of the equal load increments as a function of time and divided by the number of load increments to obtain a weighted U. Only one curve need be evaluated for the soil if the thickness of the compressible stratum and coefficient of consolidation are constant. The weighted U of each load increment may then be summed graphically as illustrated in Figure 7-22 to determine the degree of consolidation of the time-dependent loading. Figure 7-23 shows a nomograph for evaluating U for a uniform rate of load application.

Figure 7-22 Degree of consolidation for time-dependent loading

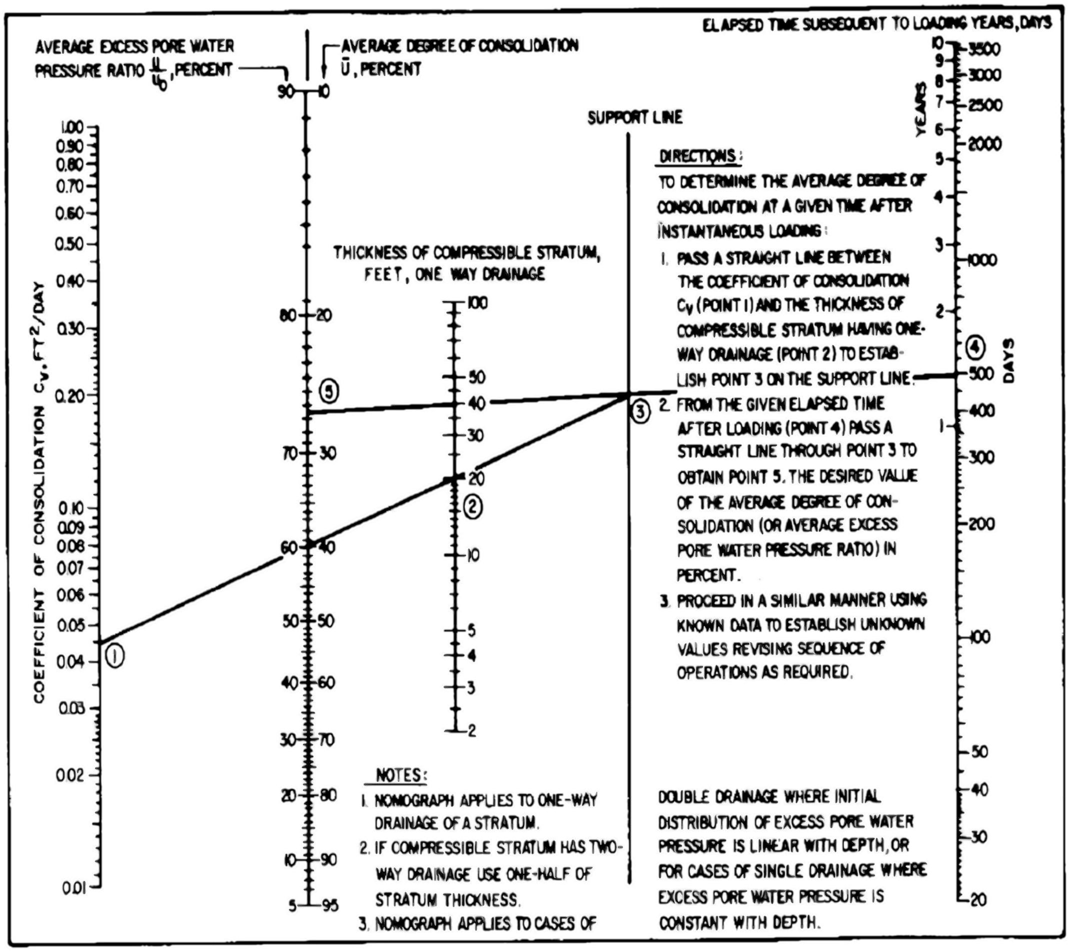


a. TIME-DEPENDENT LOADING



b. DEGREE OF CONSOLIDATION

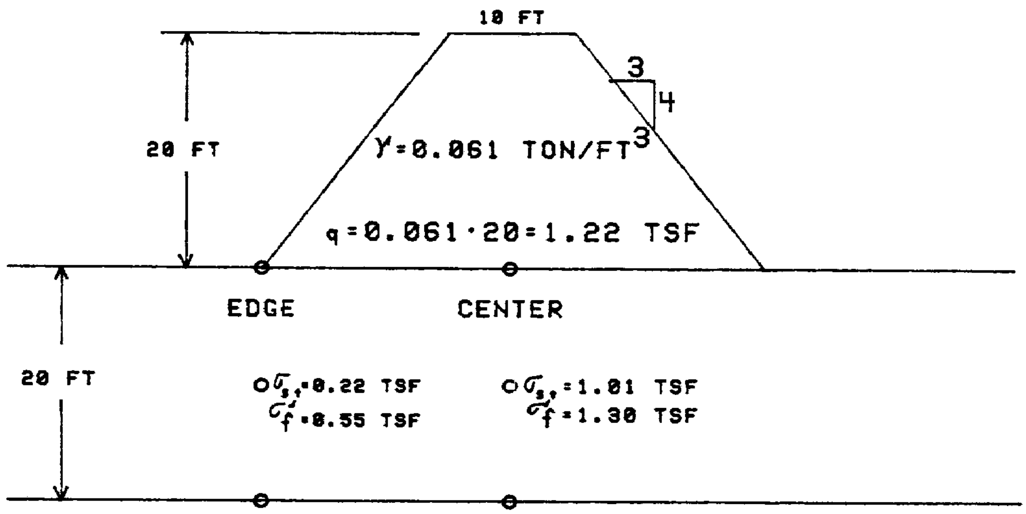
Figure 7-23 Nomograph for Consolidation with Vertical Drainage



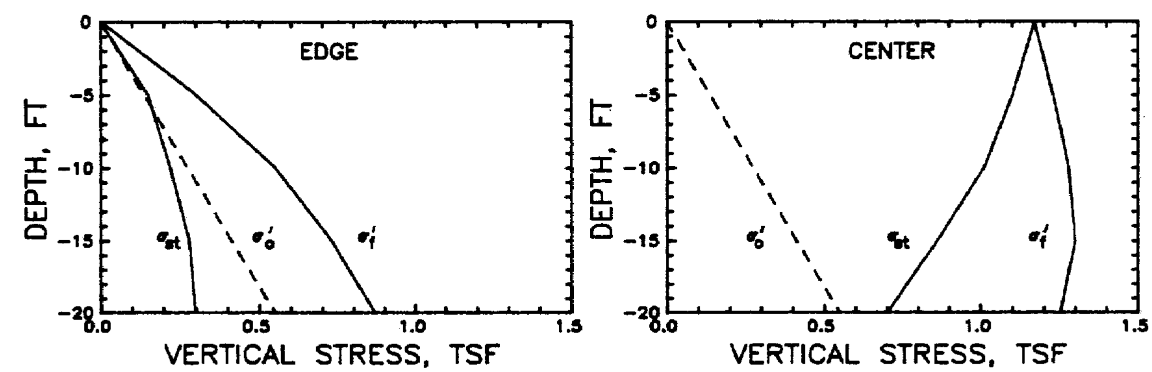
7.3.5.4. Example Application of Primary Consolidation

An embankment, Figure 7-24, is to be constructed on a compressible clay stratum 20 ft thick. The groundwater level is at the top of the compressible clay stratum. A consolidometer test was performed on an undisturbed specimen of the soil stratum after the standard load procedure described in 7.7. The specimen was taken from a depth of 10 ft and drainage was allowed on both top and bottom surfaces. A plot of the laboratory consolidation void ratio versus logarithm pressure relationship is shown in Figure 7-15.

Figure 7-24 Embankment for example application



a. ELEVATION DETAIL



b. VERTICAL STRESS DISTRIBUTION

7.3.5.4.1. Ultimate Primary Consolidation

The procedure described in Table 7-10 was applied to evaluate ultimate settlement beneath the edge and centre of the embankment by hand calculations. The solution is worked out in Table 7-18a.

7.3.5.4.2. Time Rate of Consolidation

The procedure described in Table 7-14 was applied to evaluate the rate of settlement beneath the edge and centre of the embankment by hand calculations assuming an instantaneous rate of loading. The solution is worked out in Table 7-18b.

Table 7-18 Evaluation of Consolidation Settlement by Hand Calculations for Example Application of Embankment, Figure 7-24

a. Total Settlment

Step	Description
1	The preconsolidation stress σ'_p after the Casagrande construction procedure, Table 3-6a, is 6.6 tsf shown in Figure 3-13 (neglecting a minimum and maximum range). Since $\sigma'_p > \sigma'_o$, the soil is overconsolidated with an OCR of about 22. The field virgin consolidation line is evaluated by the procedure in Table 3-6b.
2	The initial effective stress distribution σ'_o , is shown in Figure 3-21b. The wet unit weight γ is 0.061 ton/ft³ and γ_w is 0.031 ton/ft. σ'_o at 10 ft of depth is 0.3 tsf.
3	The initial void ratio of the specimen prior to consolidation is e_o = 1.05, Figure 3-13.
4	The virgin compression index $C_c = 0.42$ and the recompression index $C_r = 0.078$, Figure 3-13.
5	The pressure distribution applied by the embankment was calculated using the trapezoidal distribution, Table C-1b (Appendix C). At 10 ft below ground surface the vertical stress applied by the embankment at the edge is 0.22 tsf and at the center is 1.01 tsf. The final effective pressure σ'_f 10 ft below ground surface at the edge is 0.55 tsf and at the center 1.30 tsf. The estimated pressure distributions are shown in Figure 3-21b.
6	The change in void ratio Δe at the 10 ft depth may be found from Equation 3-23 where the right-hand part of the equation containing C_c is ignored because $\sigma'_f < \sigma'_p$, $\Delta e = C_r \bullet \log\frac{\sigma'_f}{\sigma'_o}$ *Edge*: $\Delta e_e = 0.078 \bullet \log_{10}\frac{0.55}{0.30} = 0.020$ *Center*: $\Delta e_c = 0.078 \bullet \log_{10}\frac{1.30}{0.30} = 0.050$
7, 8	Settlement of the stratum from Equation 3-20 is *Edge*: $\rho_{ce} = \frac{0.020}{1.00 + 1.05} \bullet 20 = 0.195\ ft\ or\ 2.34\ inches$ *Center*: $\rho_{cc} = \frac{0.050}{1.00 + 1.05} \bullet 20 = 0.488\ ft\ or\ 5.85\ inches$ Improved reliability may be obtained by testing additional specimens at different depths within the compressible stratum, calculating settlements within smaller depth increments, and adding the calculated settlements, Equation 3-22.

Step	Description
9	Settlement may be corrected for overconsolidation effects after the Skempton and Bjerrum procedure, Equation 3-24. λ is about 0.8 from Figure 3-15 for an overconsolidation ratio > 18. *Edge*: $\rho_{3ce} = 0.8 \bullet 2.34 = 1.87$ *inches* *Center*: $\rho_{3cc} = 0.8 \bullet 5.85 = 4.68$ *inches*

b. Time Rate of Settlement

Step	Description
1,2	Minimum and maximum estimates of the coefficient of consolidation may be made using the methods in Table 3-11 from a plot of the deformation as a function of time data, Figure 3-17. These data indicate c_v values from 0.007 ft²/day to 0.010 ft²/day, Table 3-11. The range of applied consolidation pressures is from 1 to 2 tsf using double drainage during the consolidation test.
3	The time factors for the range of c_v from 0.007 to 0.010 ft²/day is, Equation 3-26, $\frac{0.007t}{10^2} < T_v < \frac{0.010t}{10^2} = 0.00007t < T_v < 0.00010t$ where the time t is in days. The compressible stratum is assumed to drain on both top and bottom surfaces; therefore, the equivalent height H_e is 10 ft.
4	The excess pore water pressure distribution given by σ_{st} in Figure 3-21 appears to be similar to case 2 at the edge and case 1 at the center, Figure 3-16a. The average degree of consolidation in percent after 1, 10, and 50 years using Table 3-10 is

Time		T_v		U_t, Percent			
Years	Days	Min	Max	Edge (Case 2)		Center (Case 1)	
				Min	Max	Min	Max
1	364	0.025	0.036	14.7	18.8	17.1	21.4
10	3640	0.255	0.364	54.7	65.4	56.7	67.0
50	18200	1.274	3.640	95.6	98.9	95.8	98.9

Step	Description
5	Cases 1 and 2 of Figure 3-16a are considered representative of the initial excess pore water pressure distributions so that superposition of the cases in Table 3-9, step 5 and 6, is not necessary.

Step	Description
6	Consolidation settlement as a function of time t is, Equation 3-24

Time, Years	Settlement ρ_{ct}, inches			
	Edge		Center	
	Min	Max	Min	Max
1	0.27	0.35	0.80	1.00
10	1.02	1.22	2.65	3.14
50	1.79	1.85	4.48	4.63
∞	1.87		4.68	

7.3.5.5. Accuracy of Settlement Predictions

Experience shows that predictions of settlement are reasonable and within 50% of actual settlements for many soil types. Time rates of settlement based on laboratory tests and empirical correlations may not be representative of the field because time rates are influenced by *in situ* fissures, existence of high permeable sand or low permeable bentonite seams, impervious boundaries, and nonuniform soil parameters as well as the rate of construction.

7.3.5.5.1. Preconsolidation Stress

Soil disturbance of laboratory samples used for one-dimensional consolidation tests decreases the apparent preconsolidation stress.

7.3.5.5.2. Virgin Compression Index

Soil disturbance decreases the compression index.

7.3.5.5.3. Swelling and Recompression Indices

Soil disturbance increases the swelling and recompression indices.

7.3.5.5.4. Coefficient of Consolidation

Soil disturbance decreases the coefficient of consolidation for virgin compression and recompression, Figure 7-20, in the vicinity of initial overburden and preconsolidation stresses. The value of c_v decreases abruptly at the preconsolidation stress for good undisturbed samples.

7.3.5.5.5. Field Test Embankment

A field test embankment may be constructed for significant projects to estimate field values of soil parameters such as C_c and c_v. Installation of elevation markers, inclinometers, and piezometers allow the measurement of settlement, lateral movement, and pore pressures as a function of time. These field soil parameters may subsequently be applied to full-scale structures.

7.3.6. Secondary Compression and Creep

7.3.6.1. Description

Secondary compression and creep are time-dependent deformations that appear to occur at essentially constant effective stress with negligible change in pore water pressure. Secondary compression and creep may be a dispersion process in the soil structure causing particle movement and may be associated with electrochemical reactions and flocculation. Although creep is caused by the same mechanism as secondary compression, they differ in the geometry of confinement. Creep is associated with deformation without volume and pore water pressure changes in soil subject to shear; whereas, secondary compression is associated with volume reduction without significant pore water pressure changes.

7.3.6.1.1. Model

Secondary compression and creep may be modelled by empirical or semi-empirical visco-elastic processes in which hardening (strengthening) or softening (weakening) of the soil occurs. Hardening is dominant at low stress levels; whereas, weakening is dominant at high stress levels. Deformation in soil subject to a constant applied stress can be understood to consist of three stages. The first stage is characterized by a change in rate of deformation that decreases to zero. The second or steady state stage occurs at a constant rate of deformation. A third stage may also occur at sufficiently large loads in which the rate of deformation increases ending in failure because of weakening in the soil. Soil subject to secondary compression in which the volume decreases as during a one dimensional consolidometer test may gain strength or harden with time leading to deformation that eventually ceases, and, therefore, the second (steady state) and third (failure states) may never occur.

7.3.6.1.2. Relative Influence

Secondary compression and creep are minor relative to settlement caused by elastic deformation and primary consolidation in many practical applications. Secondary compression may contribute significantly to settlement where soft soil exists, particularly soft clay, silt, and soil containing organic matter such as peat or Muskeg or where a deep compressible stratum is subject to small pressure increments relative to the magnitude of the effective consolidation pressure.

7.3.6.2. Calculation of Secondary Compression

Settlement from secondary compression ρ_s has been observed from many laboratory and field measurements to be approximately a straight line on a semi-logarithmic plot with time, Figure 7-19a, following completion of primary consolidation. The decrease in void ratio from secondary compression is

Equation 7-39: $$\Delta e_{st} = C_\alpha \log \frac{t}{t_{100}}$$

Where

- Δe_{st} = change in void ratio from secondary compression at time t
- C_α = coefficient of secondary compression
- t = time at which secondary compression settlement is to be calculated, days
- t_{100} = time corresponding to 100% of primary consolidation, days

Secondary compression settlement is calculated from Equation 7-26 similar to primary consolidation settlement.

7.3.6.2.1. Coefficient of Secondary Compression

C_α is the slope of the void ratio-logarithm time plot for time exceeding that required for 100% of primary consolidation, t_{100}. t_{100} is arbitrarily determined as the intersection of the tangent to the curve at the point of inflection with the tangent to the straight line portion representing a secondary time effect, Figure 7-19a.

7.3.6.2.2. Estimation of C_α

A unique value of C_α/C_c has been observed, Table 7-19, for a variety of different types of soils. The ratio C_α/C_c is constant and the range varies between 0.025 and 0.100 for all soils. High values of C_α/C_c relate to organic soils. C_α will in general increase with time if the effective consolidation pressure σ' is less than a critical pressure or the preconsolidation stress σ'_p. For σ' greater than σ'_p, C_α will decrease with time; however, C_α will remain constant with time within the range of effective pressure $\sigma' > \sigma'_p$ if C_c also remains constant (e.g., the slope of the e-log σ curve is constant for $\sigma' > \sigma'_p$). A first approximation of the secondary compression index C_α is $0.0001W_n$ for $10 < W_n < 3000$ where W_n is the natural water content in percent.

Table 7-19 Coefficient of Secondary Compression C_α [198]

Soil	C_α/C_c
Clay	0.025 - 0.085
Silt	0.030 - 0.075
Peat	0.030 - 0.085
Muskeg	0.090 - 0.100
Inorganic	0.025 - 0.060

7.3.6.2.3. Accuracy

Soil disturbance decreases the coefficient of secondary compression in the range of virgin compression. Evaluation of settlement caused by secondary compression has often not been reliable.

7.3.6.2.4. Example Problem

The coefficient of secondary compression was determined to be 0.0033 and time t_{100} is 392 minutes or 0.27 day, Figure 7-19a, for this example problem. The change in void ratio after time t = 10 years or 3640 days is, Equation 7-39, The settlement from Equation 7-39 for an initial void ratio e_{100} = 0.96 is for a stratum of 20-ft thickness.

7.4. Evaluation of Settlement for Dynamic and Transient Loads

7.4.1. General

Dynamic and transient forces cause particle rearrangements and can cause considerable settlement, particularly in cohesionless soils, when the particles move into more compact positions. A large portion of dynamic live forces applied to foundation soil is from traffic on pavements. Dynamic forces from a rolling wheel depressing a pavement cause a multidirectional combination of cyclic shear and compression strains that precludes presentation of an appropriate settlement analysis in this section. This section provides guidance for analysis of settlement from earthquakes and repeated loads.

7.4.1.1. Amount of Settlement

The amount of settlement depends on the initial density of the soil, thickness of the soil stratum, and the maximum shear strain developed in the soil. Cohesionless soils with relative densities D_r greater than about 75% should not develop significant settlement; however, intense dynamic loading can cause some settlement of 1 to 2% of the stratum thickness even in dense sands.

7.4.1.2. Cause of Differential Settlement

A major cause of differential settlement is the compaction of loose sands during dynamic loading. Vibrations caused by machinery often cause differential settlement that may require remedial repairs or limitations on machine operations.

[198] Mesri, G. and Godlewski, P. 1977. "Time- and Stress-Compressibility Interrelationships," Journal of the geotechnical engineering Division, Vol 103, pp 417-430. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

7.4.1.3. Time Effects

Time required for settlement from shaking can vary from immediately to almost a day. Settlement in dry sands occurs immediately during shaking under constant effective vertical stress. Shaking of saturated sands induces excess pore water pressures that lead to settlement when the pore pressures dissipate.

7.4.1.4. Accuracy

Errors associated with settlement predictions from dynamic loads will exceed those for static loads and can be 50% or more. These first order approximations should be checked with available experience.

7.4.1.5. Minimizing Settlement

Dynamic settlement may be insignificant if the sum of dynamic and static bearing stresses remain less than one half of the allowable bearing capacity. Settlements that might occur under sustained dynamic loadings may be minimized by precompaction of the soil using dynamic methods. Dynamic compaction subjects the soil to severe dynamic loads that reduces the influence of any later shaking on settlement. Refer to 7.6 for dynamic compaction methods of minimizing settlement.

7.4.2. Settlement from Earthquakes

Earthquakes primarily cause shear stress, shear strain, and shear motion from deep within the earth that propagates up toward the ground surface. This shear can cause settlement initially in deep soil layers followed by settlement in more shallow layers. Settlement caused by ground shaking during earthquakes is often nonuniformly distributed and can cause differential movement in structures leading to major damage. Settlement can occur from compaction in moist or dry cohesionless soil and from dissipation of excess hydrostatic pore pressure induced in saturated soil by earthquake ground motions. Ground motions are multidirectional; however, measurements are generally made in two horizontal and one vertical acceleration components that propagate upward from underlying rock. The vertical component of acceleration is often considered to account for less than 25% of the settlement, but this percentage may be exceeded. Soil affected by ground motion and subsequent settlement may extend to considerable depth depending on the source of motion.

7.4.2.1. Tentative Simplified Procedure for Sand

A tentative simplified procedure to estimate settlement from the shaking forces of earthquakes on saturated sands that are at initial liquefaction and on dry sands is given in Table 7-20. Input data for this procedure include the blow count N from SPT data as a function of depth, effective and total overburden pressures σ'_o and σ_o, and an estimate of the maximum horizontal acceleration of the ground surface from earthquake records[199].

[199] Regulation Guide 1.6, Nuclear Regulatory Commission, items 33 and 34; Office, Chief of Engineer policy for Corps of Engineer specifications for ground motions is provided by the Earthquake Engineering and Geosciences Division, Geotechnical Laboratory, USAE Waterways Experiment Station

Table 7-20 A Suggested Tentative Procedure for Computation of Earthquake Settlement in Sand[200]

a. Saturated Sand that Reaches Initial Liquefaction

Step	Description
1	Determine the blow count N from SPT tests as a function of depth and divide the profile into discrete layers of sand with each layer containing sand with similar blow count.
2	Determine correction factor C_{ER} as follows where C_{ER} = estimated rod energy in percent/60

Country	Hammer	Hammer Release	C_{ER}
Japan	Donut	Free-Fall	1.3
	Donut	Rope and Pulley with special throw release	1.12*
USA	Safety	Rope and Pulley	1.00*
	Donut	Rope and Pulley	0.75
Europe	Donut	Free-Fall	1.00*
China	Donut	Free-Fall	1.00*
	Donut	Rope and Pulley	0.83

*Prevalent method in USA today

Step	Description
3	Estimate the total and effective overburden pressure σ_o and σ_o' in tsf units from known or estimated soil unit weights and pore water pressures of each layer.
4	Estimate the relative density D_r in percent from results of SPT data using Figure 4-1, improved correlations for overconsolidated soil (item 50), or the expression

$$D_r = 21 \bullet \frac{N_J}{\sigma_o' + 0.7} \qquad (4\text{-}1a)$$

where N_J is the blowcount by Japanese standards and σ_o' is the effective overburden pressure. D_r for normally consolidated material may be estimated by (item 42)

$$D_r = 11.7 + 0.76 \bullet [222N + 1600 - 736\sigma_o' - 50c_u^2]^{1/2} \qquad (4\text{-}1b)$$

where

σ_o' = effective overburden pressure, tsf
c_u = uniformity coefficient, D_{60}/D_{10}
D_{60} = grain diameter at which 60 percent of soil weight is finer
D_{10} = grain diameter at which 10 percent of soil weight is finer

[200] Tokimatsu, K. and Seed, H. B. 1987. "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of geotechnical engineering, Vol 113, pp 861-878. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017. Information referenced as “Figure 4-1” is actually Figure 4-10. “Figure 4-2” is Figure 4-11. “Figure 4-3” is Figure 7-25. ”Figure 4-4” is Figure 7-26. “Figure 4-5” is Figure 7-27. “Figure 4-6” is Figure 7-28.

Step	Description
5	Determine the correction factor C_N from Figure 4-2 using σ'_o and D_r .
6	Calculate normalized blowcount

$$(N_1)_{60} = C_{ER} \cdot C_N \cdot N \qquad (4\text{-}2)$$

where $(N_1)_{60}$ = SPT blowcount normalized to an effective energy delivered to the drill rod at 60 percent of theoretical free-fall energy.

7 Calculate the cyclic shear stress ratio causing initial liquefaction to occur for the given earthquake of magnitude M

$$\left[\frac{\tau_{av}}{\sigma'_o}\right]_M = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma'_o} \cdot r_d \qquad (4\text{-}3)$$

where

τ_{av} = average cyclic shear stress induced by earthquake shaking, tsf
σ'_o = effective overburden pressure, tsf
σ_o = total overburden pressure, tsf
a_{max} = maximum horizontal acceleration of the ground surface in units of g from earthquake records of magnitude M (Regulation Guide 1.60, Nuclear Regulatory Commision; refer to Earthquake Engineering and Geosciences Division, Geotechnical Laboratory, USAE Waterways Experiment Station for Corps of Engineers)
g = acceleration of gravity, 32.2 ft/sec/sec
r_d = stress reduction factor; 1.0 at the ground surface decreasing to 0.9 at depth 30 ft below ground surface

8 Convert $(\tau_{av}/\sigma'_o)_M$ to an equivalent earthquake of magnitude M = 7.5 by

$$\left[\frac{\tau_{av}}{\sigma'_o}\right]_{7.5} = \left[\frac{\tau_{av}}{\sigma'_o}\right]_M \cdot \frac{1}{r_m} \qquad (4\text{-}4)$$

where the scaling factor r_m is

Step	Description

Magnitude of Earthquake M	*No Representative Cycles at $0.65\tau_{max}$	r_m
8.5	26	0.89
7.5	15	1.00
6.75	10	1.13
6.00	5	1.32
5.25	2 - 3	1.50

* Representative of the number of equivalent stress cycles caused by the earthquake where τ_{max} = maximum cycle stress

9 Evaluate volumetric strain ε_c in percent after initial liquefaction from Figure 4-3 using calculated values of $(N_1)_{60}$ of step 6 and $(\tau_{av}/\sigma_o')_{7.5}$ of step 8.

10 Evaluate earthquake settlement ρ_e after initial liquefaction in inches from

$$\rho_e = \sum_{j=1}^{n} \frac{\epsilon_c}{100} \cdot h_j \qquad (4\text{-}5)$$

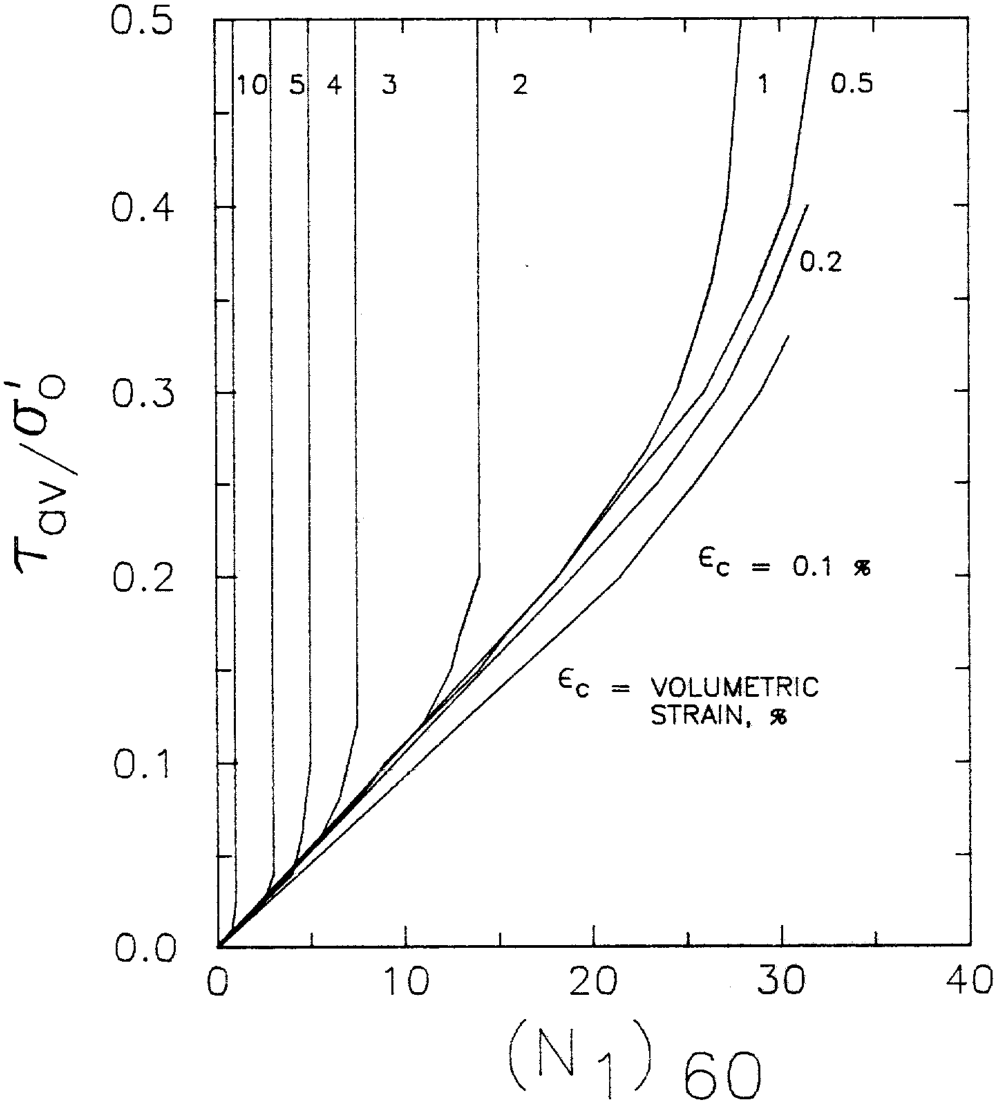
where h_j = thickness of each stratum j in inches

b. Dry Sand

Step	Description
1-6	Repeat steps 1 through 6 in Table 4-1a above to evaluate D_r and $(N_1)_{60}$.
7	Evaluate mean effective pressure σ_m' of each stratum in tsf (e.g., $\sigma_m' = \frac{(1 + 2K_o)}{3} \cdot \sigma_o' = 0.65\sigma_o'$ if the coefficient of lateral earth pressure K_o = 0.47) . σ_m' is considered the total mean pressure in dry sand.
8	Calculate $$G_{max} = 10 \cdot [(N_1)_{60}]^{1/3} \cdot [\sigma_m']^{1/2} \qquad (4\text{-}6)$$ where G_{max} = maximum shear modulus, tsf

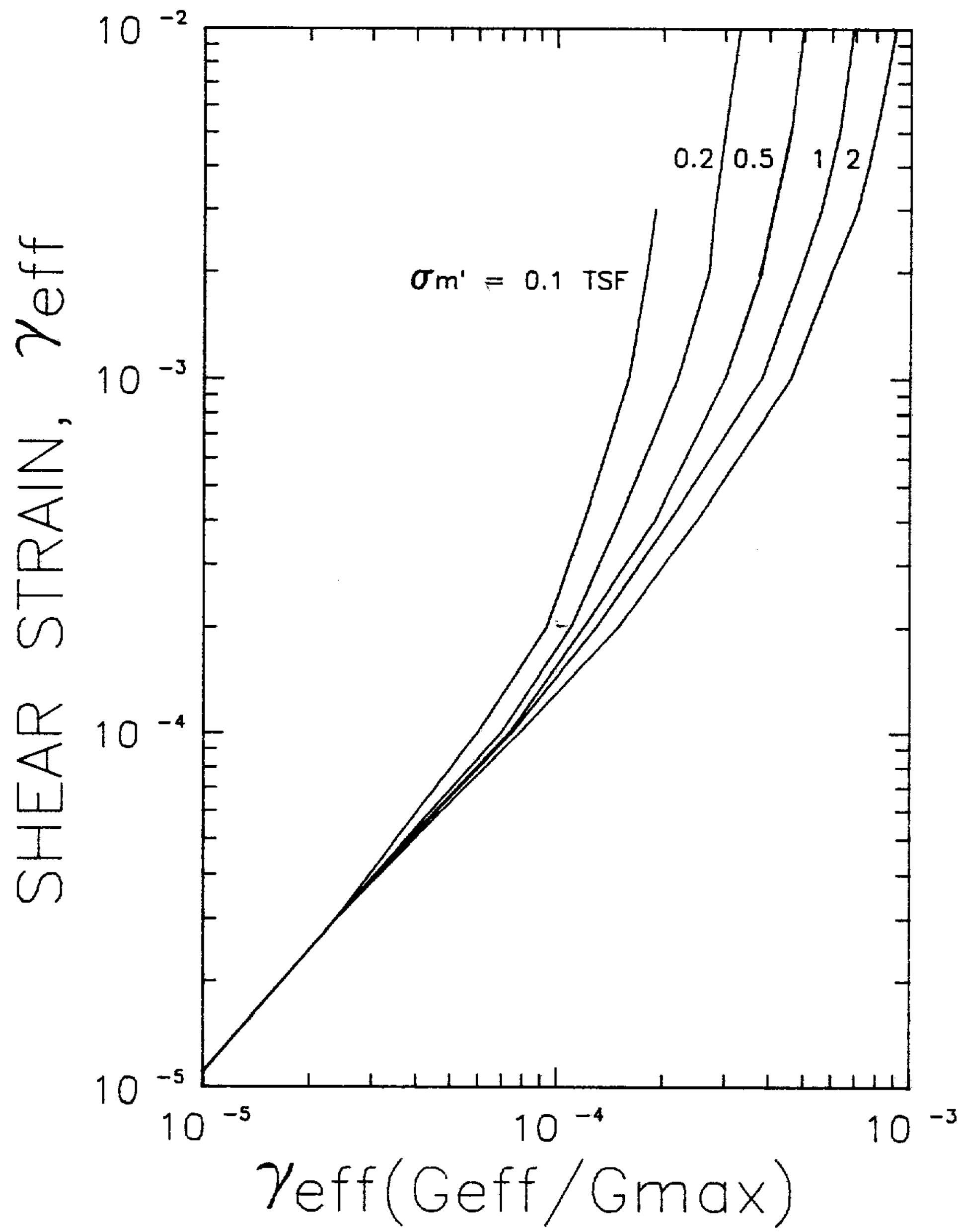
Step	Description
9	Evaluate using G_{max} from step 8 $$\gamma_{eff} \cdot \frac{G_{eff}}{G_{max}} = \frac{0.65 \cdot a_{max} \cdot \sigma_o \cdot r_d}{g \cdot G_{max}} \quad (4\text{-}7)$$ where γ_{eff} = effective cyclic shear strain induced by an earthquake G_{eff} = effective shear modulus at earthquake induced shear stress, tsf a_{max} = maximum horizontal acceleration at the ground surface in units of g σ_o = total overburden pressure, tsf g = acceleration of gravity, 32 ft/sec/sec r_d = stress reduction factor; 1.0 at the ground surface decreasing to 0.9 at a depth 30 ft below ground surface
10	Evaluate γ_{eff} from Figure 4-4 using $\gamma_{eff} \cdot G_{eff}/G_{max}$ from step 9 and σ'_m from step 7; multiply by 100 to convert to percent.
11	Use γ_{eff} and evaluate volumetric strain in percent $\varepsilon_{c,7.5}$ from Figure 4-5 using D_r or $(N_1)_{60}$ for an $M = 7.5$ magnitude earthquake.
12	Evaluate volumetric strain ratio from Figure 4-6 for the given magnitude of earthquake M and multiply this ratio by $\varepsilon_{c,7.5}$ to calculate $\varepsilon_{c,M}$.
13	Multiply $\varepsilon_{c,M}$ by 2 to consider the multidirection effect of earthquake shaking on settlement and evaluate total earthquake induced settlement of each stratum j of thickness h_j for n strata by $$\rho_e = \sum_{j=1}^{n} \frac{2\varepsilon_{c,M}}{100} \cdot h_j \quad (4\text{-}8)$$

Figure 7-25 Proposed relationship between cyclic stress ratio $(N_1)_{60}$ and volumetric strain ε_c for saturated clean sands for M = 7.5 earthquake[201]



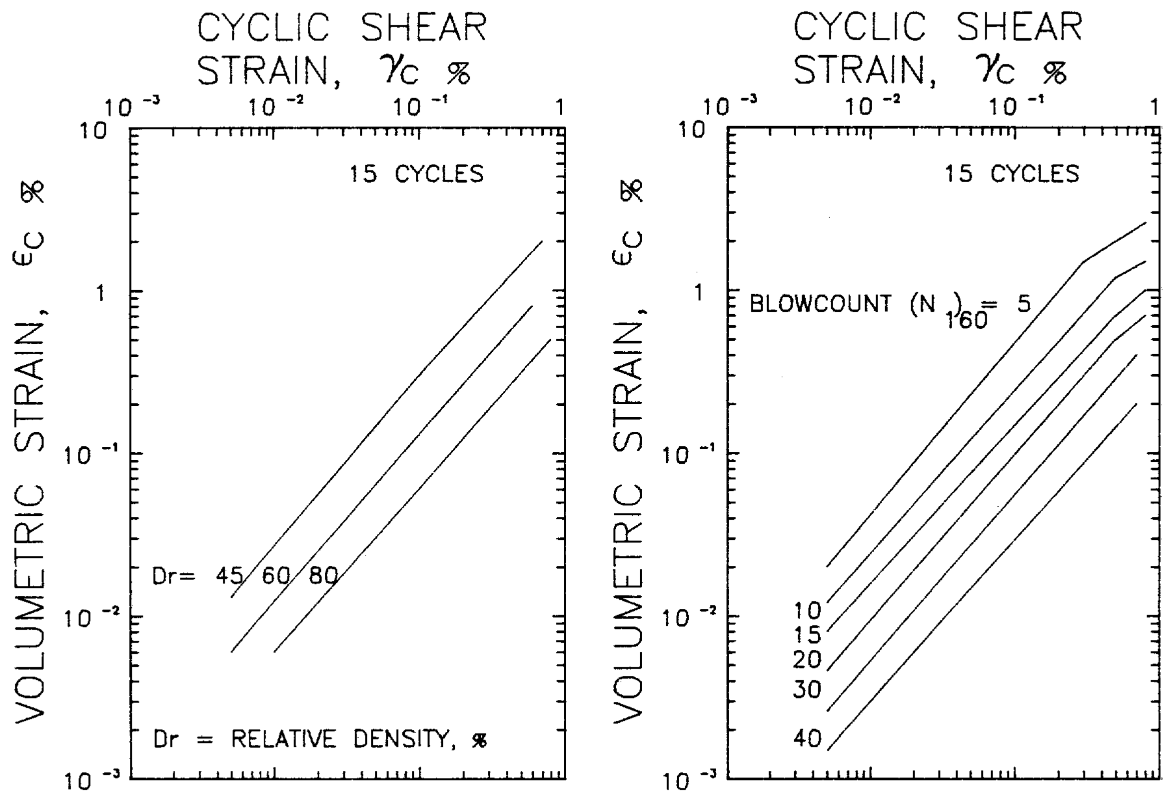
[201] Reprinted by permission of the American Society of Civil Engineers from Journal of geotechnical engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 866.

Figure 7-26 Plot for determination of induced strain in sand deposits[202]



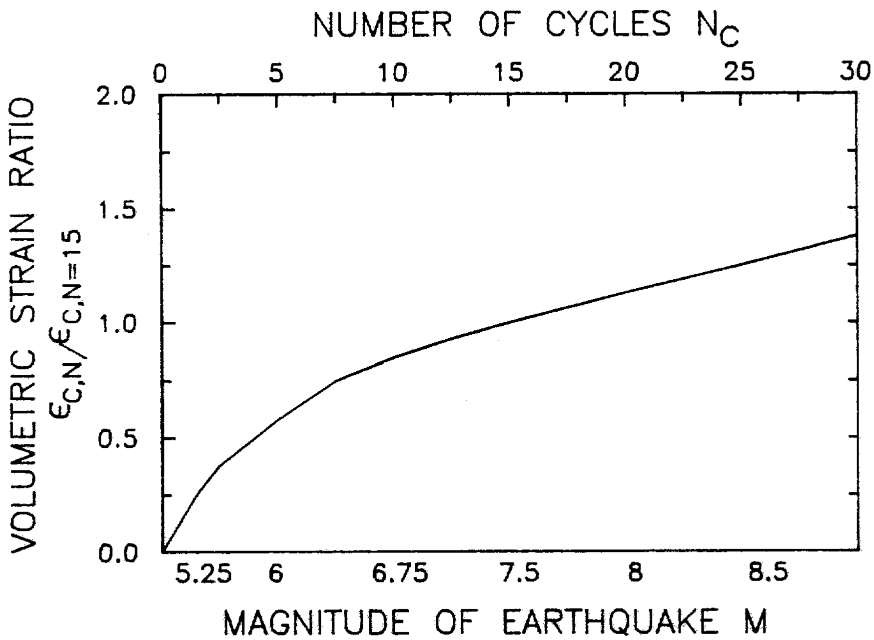
[202] Reprinted by permission of the American Society of Civil Engineers from Journal of geotechnical engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 873.

Figure 7-27 Relationships between volumetric strain ε_c and cyclic shear strain γ_c for dry sand and earthquake magnitude M = 7.5[203]



[203] Reprinted by permission of the American Society of Civil Engineers from Journal of geotechnical engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 874.

Figure 7-28 Relationship between volumetric strain ratio and number of cycles (earthquake magnitude) for dry sands[204]



7.4.2.1.1. Application

The procedure is applied to the Tokachioki earthquake in Table 7-21.

[204] Reprinted by permission of the American Society of Civil Engineers from Journal of geotechnical engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 874.

Table 7-21 Example Applications of Simplified Procedure to Estimate Earthquake Settlement

a. Saturated Sand at Initial Liquefaction Condition

Layer	Thickness, ft	N	C_{ER}	σ_o', psf	C_N	$(N_1)_{60}$	$\frac{\tau_{av}}{\sigma_o}$	ε_c, percent	ρ_e in.
1	4.0	1.0	0.82*	240					
2	3.3	0.5	0.82*	575	1.7	0.7	0.155	10.0	4.0
3	3.3	0.5	0.82*	764	1.57	0.6	0.185	10.0	4.0
4	3.3	0.5	1.09	954	1.44	0.8	0.200	10.0	4.0
5	3.3	2.0	1.09	1144	1.34	2.9	0.210	5.5	2.2
6	3.3	5.0	1.09	1334	1.24	6.8	0.215	3.2	1.3
7	3.3	23.0	1.21	1523	1.16	32.0	0.220	0.0	0.0
8	3.3	33.0	1.21	1713	1.09	44.0	0.225	0.0	
9	3.3	28.0	1.21	1903	1.03	35.0	0.225	0.0	
10	3.3	33.0	1.21	2093	0.97	39.0	0.225	0.0	

Note: *Corrected by 0.75 — Total Settlement = 15.5
Water Table = 4 ft — Observed Maximum Settlement ≈ 20
Estimated Maximum Acceleration a_{max} = 0.2 g

b. Dry Sand

Layer	Thickness, ft	σ_o', psf	G_{max}, ksf	$\gamma_{eff}\frac{G_{eff}}{G_{max}}$	γ_{eff}	$\varepsilon_{c,7.5}$, percent	$\varepsilon_{c,6.6}$ percent	ρ_e, in.
1	5	240	520	0.00013	0.0005	0.14	0.11	0.13
2	5	715	900	0.00023	0.0008	0.23	0.18	0.22
3	10	1425	1270	0.00032	0.0012	0.35	0.28	0.67
4	10	2375	1630	0.00040	0.0014	0.40	0.32	0.77
5	10	3325	1930	0.00045	0.0015	0.45	0.36	0.86
6	10	4275	2190	0.00046	0.0013	0.38	0.30	0.71

Note: Total estimated settlement = 2.7 inches — Total Settlement = 3.37

D_r = 45 percent — $(N_1)_{60}$ = 9
a_{max} = 0.45 g

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7.4.2.1.2. Validation

This tentative procedure has not been fully validated.

The example problems in Table 7-21 are based on estimated field behaviours and not on measured data against which to validate a settlement analysis.

7.4.3. Settlement from Repeated Loads and Creep

Structures subject to repeated vertical loads experience a long-term settlement from the compression of cumulative cyclic loads and secondary compression or creep. Operating machinery, pile driving, blasting, wave or wind actions are common causes of this type of dynamic loading. Methods of estimating secondary compression are provided in 7.3.

7.4.3.1. Compaction Settlement from Machine Vibrations

Vibration tends to densify loose nonplastic soils, causing settlement. The greatest effect occurs in loose, coarse-grained sands and gravels. These materials must be stabilized by compaction or other means to support spread foundations for vibrating equipment. Shock or vibrations near a foundation on loose, saturated nonplastic silt, or silty fine sands, may produce a quick condition and partial loss of bearing capacity. In these cases, bearing intensities should be less than those normally used for static loads. For severe vibration conditions, reduce the bearing pressures to one-half allowable static values.

In most applications, a relative density of 70% to 75% in the foundation soil is satisfactory to preclude significant compaction settlement beneath the vibratory equipment. However, for heavy machinery, higher relative densities may be required. The following procedure may be used to estimate the compaction settlement under operating machinery.

The critical acceleration of machine foundations, $(a)_{crit}$, above which compaction is likely to occur, may be estimated based on[205]:

Equation 7-40: $$(a)_{crit} = -\frac{\ln\left[1-\frac{(D_r)_0}{100}\right]}{\beta}$$

Where:

- $(a)_{crit}$ = critical acceleration expressed in g's
- $(D_r)_o$ = initial (*in situ*) relative density at zero acceleration expressed in percent.
- β = coefficient of vibratory compaction, a parameter depending on moisture content; varies from about 0.8 for dry sand down to 0.2 for low moisture contents (about 5%). It increases to a maximum value of about 0.88 at about 18% moisture content. Thereafter, it decreases.

When densification occurs because of vibrations there will be an increase in relative density D_r, and for a sand layer with a thickness H, the settlement would be ΔH. The strain ΔH/H can be expressed in terms of D_r as:

Equation 7-41: $$\frac{\Delta H}{H} = 0.0025\gamma_{do}\left(\frac{\Delta D_r\%}{100}\right)$$

Where γ_{do} = the initial dry density of the sand layer (lb/cu. ft)

The above equation is based on the range of maximum and minimum dry densities for sands. The change in relative density D_r due to vibration is defined as:

Equation 7-42: $$\Delta D_r = (D_r)_i - (D_r)_o$$

Where:

[205] Barkan, D. D., *Dynamics of Bases of Foundation*, McGraw Hill Book Company, Inc., 1962.

- $(D_r)_o$ = initial in-situ relative density which may be estimated from the standard penetration resistance
- $(D_r)_f$ = final relative density, which may be conservatively estimated based on:

$$\textbf{Equation 7-43: } (D_r)_f = 100\left\{-e^{-\beta\left[(a_i)_{crit}+a_i\right]}\right\},\ a_i > (a_i)_{crit}$$

Or

$$\textbf{Equation 7-44: } (D_r)_f = (D_r)_o,\ a_i < (a_i)_{crit}$$

Where:

a_i = acceleration expressed in g's.

In the above equation $(a_i)_{crit}$ and (a_i) are the critical acceleration and acceleration produced by equipment in each layer i. The acceleration a_i produced by equipment may be approximated using the following:

$$\textbf{Equation 7-45: } a_i = a_o\sqrt{\frac{r_o}{d}},\ d > r$$

$$\textbf{Equation 7-46: } a_i = a_o,\ d < r$$

Where:

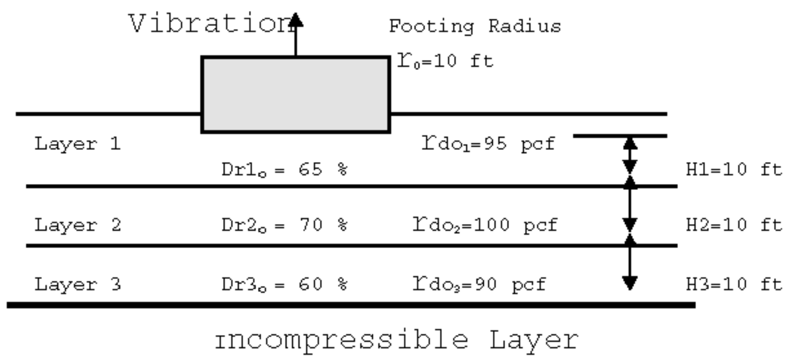
- a_o = acceleration of vibration in g's at foundation level
- d = distance from base of foundation to mid point of soil layer
- r_o = equivalent radius of foundation

If maximum displacement, A_{max}, and frequency of vibration, ω rad/sec), are known at base of foundation then:

$$\textbf{Equation 7-47: } a_o = \omega^2 A_{\max}$$

An example illustrating the use of the above principles is shown in Figure 10-10.

Figure 7-29 Example Calculation for Vibration Induced Compaction Settlement Under Operating Machinery



GIVEN: Soil profile as shown:

Footing with radius $r_o := 10\,\text{ft}$ subjected to a vibratory load causing a peak dynamic displacement

$A_{max} := 0.007 \cdot \text{in}$

Operating frequency $f := 2500\,\text{min}^{-1}$ (rev/min). Moisture content of soil is 16%. Use $\beta := 0.88$

$\omega := f \cdot 2 \cdot \pi$ $\quad \omega = 26$ (rad/sec) $\qquad a_o := \dfrac{\omega^2 \cdot A_{max}}{32.2\dfrac{\text{ft}}{\text{sec}^2}}$ $\qquad a_o = 1.2$ g

LAYER 1

Depth to mid layer $\quad d := 5 \cdot \text{ft} \quad d < r_o \qquad$ Therefore use $\quad a_i := a_o$

Critical Acceleration $\quad a_{crit} := \dfrac{-\ln\left(1 - \dfrac{Dr1_o}{100}\right)}{\beta} \qquad a_{crit} = 1.1$ g $\qquad a_i > a_{crit}$

$D_{rf} := 100\left[1 - \exp\left[(-\beta) \cdot \left(a_{crit} + a_i\right)\right]\right] \qquad \Delta Dr := D_{rf} - Dr1_o \qquad \Delta H := \left(0.0025\dfrac{\text{ft}^3}{\text{lb}} \cdot \dfrac{\Delta Dr}{100} \cdot \gamma_{do1}\right) \cdot H_1$

$D_{rf} = 88.26$ % $\qquad \Delta Dr = 23.2$ % $\qquad \Delta H = 6.63$

LAYER 2

$d_{midlayer} := 15\,\text{ft}$

$a_i := a_o \cdot \sqrt{\dfrac{r_o}{d_{midlayer}}} \qquad a_{crit} := \dfrac{-\ln\left(1 - \dfrac{Dr2_o}{100}\right)}{\beta}$

$a_i = 0.785$ g $\qquad a_{crit} = 1.368$ g

$a_i < a_{crit}$

$D_{rf} := Dr2_o$

No significant compaction settlement is likely.

LAYER 3

$d := 25\,\text{ft}$

$a_i := a_o \cdot \sqrt{\dfrac{r_o}{d}} \qquad a_{crit} := \dfrac{-\ln\left(1 - \dfrac{Dr3_o}{100}\right)}{\beta}$

$a_i = 0.785$ g $\qquad a_{crit} = 1.368$ g

$a_i < a_{crit}$

No significant compaction settlement is likely.

Anticipated Compaction Settlement = 6.6 in. Increase relative density of top layer to 70 percent or greater.

7.4.3.2. Settlement Calculated from Laboratory Cyclic Strain Tests

Drained cyclic triaxial tests may be performed on pervious soil to evaluate the cyclic settlement through a cyclic strain resistance r_ε [206].

7.4.3.2.1. Test procedure

The soil should be consolidated to simulate the *in situ* stress state of effective horizontal and vertical pressures. The soil is subsequently subject to three different cyclic stress levels to evaluate r_ε.

a) The soil should be consolidated so that a plot of one-half of the deviator stress versus the effective horizontal confining pressure provides a slope indicative of a realistic effective coefficient of lateral earth pressure. The slope s of this curve required to obtain a given coefficient of lateral earth pressure K_o is

Equation 7-48: $$s = 0.5\left(\frac{1}{K_o} - 1\right)$$

For example, the slope s should be 0.7 if K_o is 0.42. The soil should be consolidated to an effective horizontal confining pressure simulating the *in situ* soil.

b) Additional vertical dynamic loads should be applied so that the soil specimen is subject to three different cyclic stress levels of 200 to 300 cycles per stress level. The effective lateral confining pressure is maintained constant.

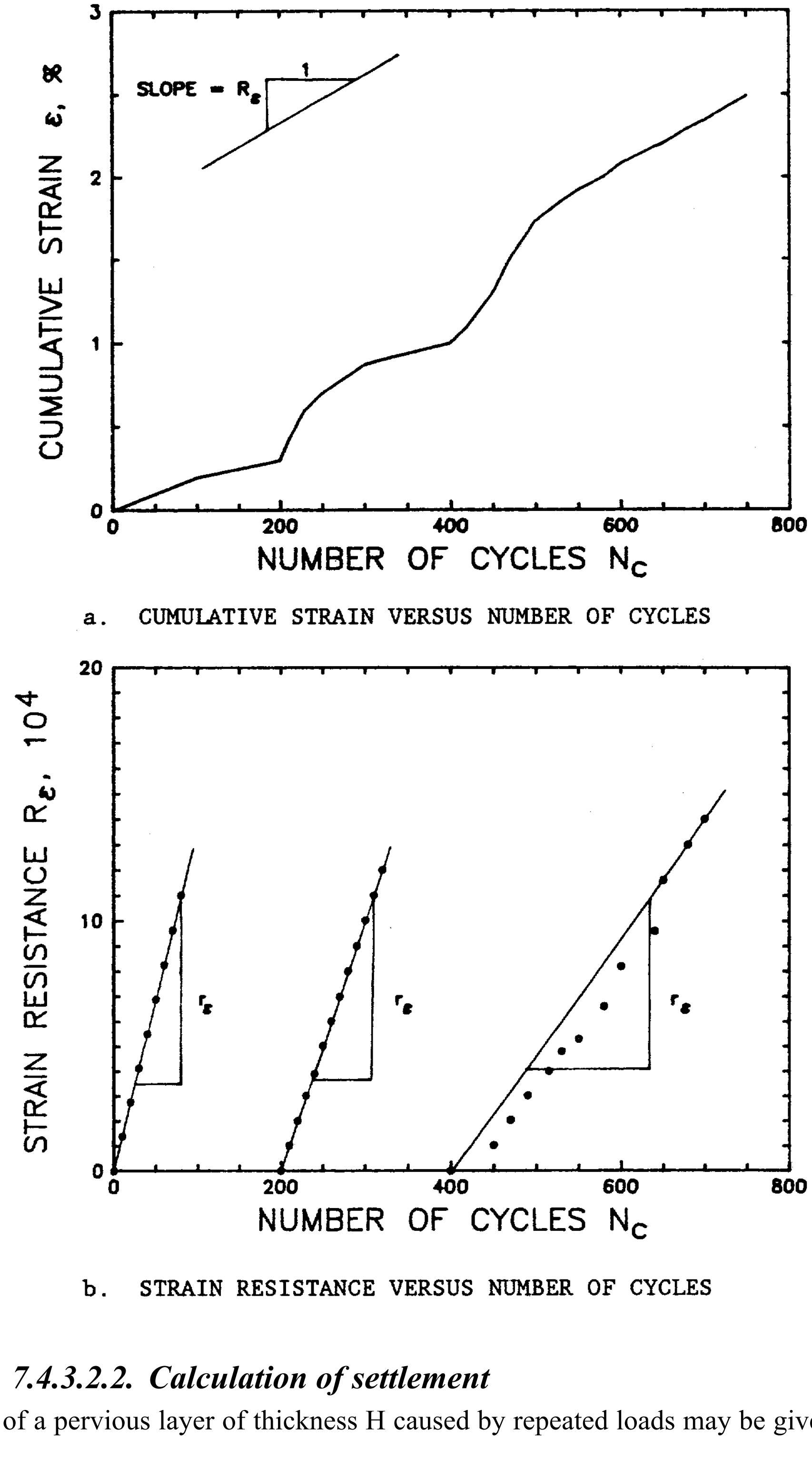
c) The cumulative strain as a function of the number of cycles N at each stress level should be plotted as shown in Figure 7-30a. The slope of the curves in Figure 7-30a is the strain resistance $R_\varepsilon = dN/d\varepsilon$.

d) The strain resistance should be plotted versus the number of cycles as shown in Figure 7-30b for each stress level. A straight line should subsequently be plotted through these data points for each stress level. The slope of this line is the cyclic strain resistance r_ε.

e) The cyclic strain resistance decreases with increasing stress levels and approaches zero when the shear strength is fully mobilized. The cyclic strain resistance may increase with increasing depth because the percentage of mobilized shear strength may decrease with increasing depth.

[206] Janbu, N. and Senneset, K. 1981. "Settlement Due to Drained, Cyclic Loads," Proceedings Tenth International Conference on Soil Mechanics and Foundation Engineering, Vol 1, pp 165-170, Stockholm, Sweden. Available from A. A. Balkema, P.O. Box 1675, Rotterdam, The Netherlands.

Figure 7-30 Example of Strain and Strain Resistance as a Function of Cycles N_c for Different Stress Levels



a. CUMULATIVE STRAIN VERSUS NUMBER OF CYCLES

b. STRAIN RESISTANCE VERSUS NUMBER OF CYCLES

7.4.3.2.2. Calculation of settlement

The settlement of a pervious layer of thickness H caused by repeated loads may be given for this drained soil by

Equation 7-49: $\rho_c = \frac{H}{r_z} \ln N$

Where

- ρ_r = settlement of the layer from repeated load, ft
- H = thickness of stratum, ft
- r_ε = cyclic strain resistance of stratum from laboratory tests
- N = number of cycles of repeated load

The appropriate value of r_ε to select from the laboratory test results depends on the maximum anticipated stress level in the soil caused by the repeated loads. For example, the maximum anticipated stress in the soil level might be calculated from the exciting force. The exciting force may be calculated from guidance provided elsewhere in this book.

7.4.3.2.3. Alternative settlement calculation

An alternative method of evaluating effects of repeated loads on settlement of clayey soil from laboratory cyclic triaxial tests is to apply the creep strain rate formulation

Equation 7-50: $\varepsilon_t = \varepsilon_{t1} + \frac{e^\alpha}{1-\lambda_d}\left[t^{1-\lambda_d} - t_1^{1-\lambda_d}\right]$

If λ_d = 1, then

Equation 7-51: $\varepsilon_1 = \varepsilon_{t1} + e^\alpha \ln \frac{t}{t_1}$

Where

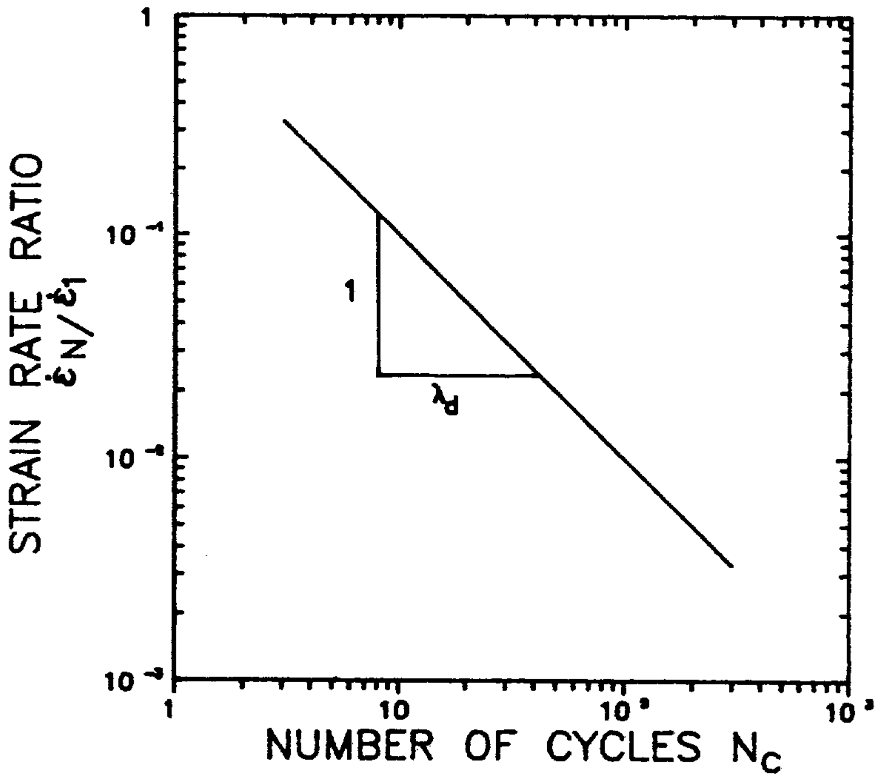
- ε = strain at time t
- ε_{t1} = strain at time t_1 or after one cycle
- e = base e or 2.7182818
- $\alpha = C\,\sigma_{rd} - B$
- B, C = constants from Table 7-22
- σ_{rd} = repeated deviator stress, tsf
- λ_d = decay constant found from slope of logarithmic strain rate $\varepsilon_N/\varepsilon_1$ versus logarithm number of cycles N_c, Figure 7-31.

a) Settlement may be found by substituting ε_t of Equation 7-51 for ε_c of Equation 4-5, Table 7-20a. Evaluation of ε_t from Equation 7-50 or Equation 7-51 is appropriate for repeated loads with frequencies between 0.1 Hz and 10 Hz, a typical range for traffic loads; however, settlement may be underestimated because traffic loads are more complex than compressive vertical loads. Repeated loads with various periods and rest intervals between repeated loads do not appear to cause significant change in strain.

Table 7-22 Constants B and C to Evaluate Creep Constant α as a Function of Overconsolidation Ratio OCR[207]

OCR	*C*	*B*
4	3.5	9.5
10	2.8	9.2
20	3.7	9.5

Figure 7-31 Example decay constant



b) An application of Equations 4-18 to London clay where $\lambda_d = 1$, $\varepsilon_{t1} = 0.0$ at $t_1 = 1$ second, $\sigma_{rd} = 1$ tsf, and OCR = 4 is

$$\varepsilon_1 = \varepsilon_{t1} + e^{\alpha} \ln\frac{t}{t_1} = e^{3.5\ 1.0-9.5} \ln t$$

$$= \frac{\ln t}{e^6} = \ln\frac{t}{403.4} = 0.0025 \ln t$$

After 10 seconds the strain ε_{10} is 0.0058. Settlement is the strain times thickness of the stratum.

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[207] Hyde, A. F. and Brown, S. F. 1976. "The Plastic Deformation of a Silty Clay Under Creep and Repeated Loading," Geotechnique, Vol 26, pp 173-184, The Institution of Civil Engineers. Available from Thomas Telford Ltd., 1-7 Great George Street, Westminster, London, SW1P 3AA, England.

7.5. Applications with Unstable Foundation Soil

7.5.1. Unstable Soils

Many types of soils change volume from causes different from elastic deformation, consolidation, and secondary compression. These volume changes cause excessive total and differential movements of overlying structures and embankments in addition to load-induced settlement of the soil.

Such unstable conditions include the heaving of expansive clays and collapse of silty sands, sandy silts, and clayey sands from alteration of the natural water content.

7.5.1.1. Effects of Excessive Movements

Excessive total and, especially, differential movements have caused damages to numerous structures that have not been adequately designed to accommodate the soil volume changes. Types of damage include impaired functional usefulness of the structure, external and interior cracked walls, and jammed and misaligned doors and windows. Important factors that lead to damages are the failure to recognize the presence of unstable soil and to make reasonable estimates of the magnitude of maximum heave or settlement/collapse. Adequate engineering solutions such as special foundation designs and soil stabilization techniques exist to accommodate the anticipated soil movement. A thorough field investigation is necessary to properly assess the potential movement of the soil. A qualitative estimate of potential vertical movement of proposed new construction may sometimes be made by examination of the performance of existing structures adjacent to the new construction.

7.5.1.2. Influence of Time on Movement

The time when heave or settlement/collapse occurs cannot be easily predicted because the location and time when water becomes available to the foundation soil cannot readily be foreseen.

Heave or settlement can occur almost immediately in relatively pervious foundation soil, particularly in local areas subject to poor surface drainage and in soil adjacent to leaking water lines. More often, heave or settlement will occur over months or years depending on the availability of moisture. Soil movement may be insignificant for many years following construction permitting adequate performance until some change occurs in field conditions to disrupt the moisture regime. A prediction of when heave or settlement occurs is usually of little engineering significance. Important engineering problems include reliable determination of the magnitude of potential heave or settlement and development of ways to minimize this potential for movement and potential distress of the structure.

7.5.2. Heaving Soil

7.5.2.1. General

Expansive or swelling soils are found in many areas throughout the United States and the entire world. These soils change volume within the active zone for heave from changes in soil moisture.

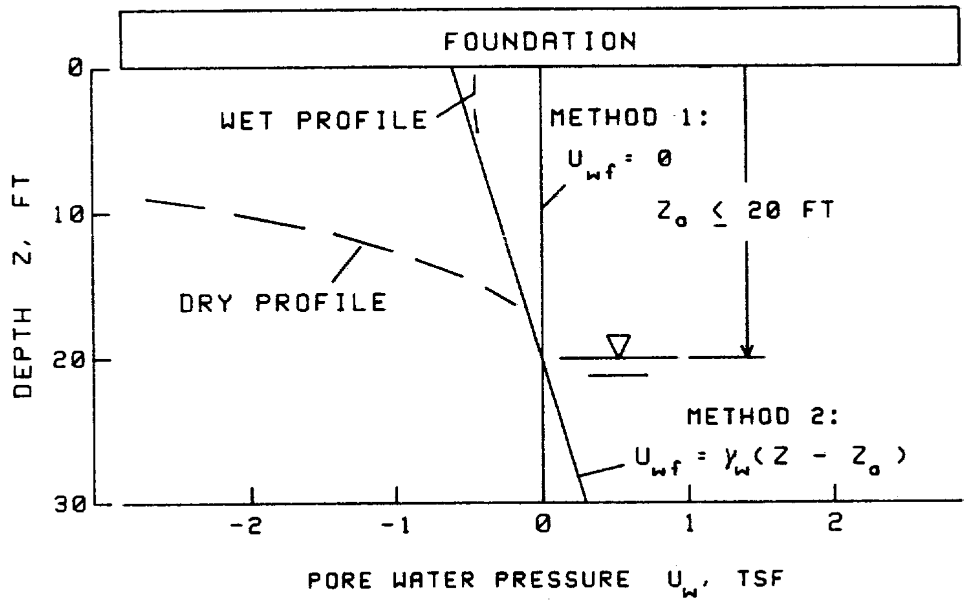
7.5.2.1.1. Soils Susceptible to Heave

These soils consist of plastic clays and clay shales that often contain colloidal clay minerals such as the montmorillonites or smectite. They include marls, clayey siltstone and sandstone, and saprolites. Some soils, especially dry residual clayey soil, may heave on wetting under low applied pressure, but collapse at higher pressure. Other clayey soil may initially collapse on wetting, but heave over long periods as water slowly wets the less pervious clay particles. Desiccation can cause expansive soil to shrink.

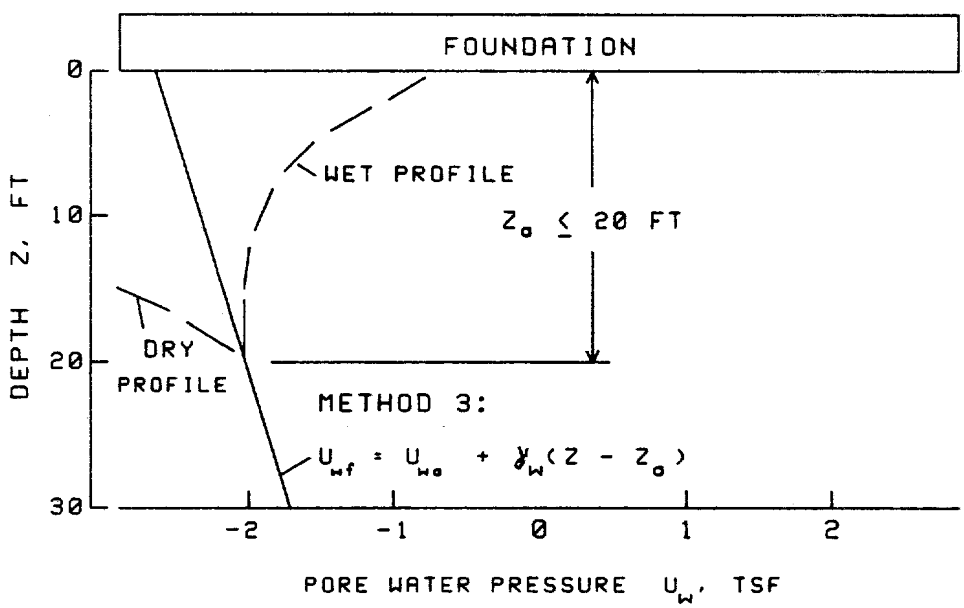
7.5.2.1.2. Depth of Active Zone

The depth of the active zone Z_a illustrated in Figure 7-32 is defined as the least soil depth above which changes in water content, and soil heave may occur because of change in environmental conditions following construction. The water content distribution should not change with time below Z_a. Experience indicates Z_a may be approximated following guidelines in Table 7-23.

Figure 7-32 Anticipated equilibrium pore water pressure profiles



a. SHALLOW GROUNDWATER LEVEL



b. DEEP GROUNDWATER LEVEL

Table 7-23 Guidelines for Estimating Depth of the Active Zone Z_a

Relative To	Guideline	
Water table	Z_a will extend to depths of shallow groundwater levels ≤ 20 ft	
Swell pressure	Z_a will be located within depths where $\sigma_{sj} - \sigma_{fj} \geq 0$, σ_{sj} = average swell pressure of stratum j and σ_{fj} = total average vertical overburden pressure after construction in stratum j	
Fissures	Z_a will be within the depth of the natural fissure system caused by seasonal swell/shrinkage	
Climate		Z_a, ft
	Humid	10
	Semi-arid	15
	Arid	20

7.5.2.1.3. Equilibrium Pore Water Pressure Profile

The pore water pressure beneath the centre of the foundation is anticipated to reach an equilibrium distribution; whereas, the pore water pressure profile beneath the perimeter will cycle between dry and wet extremes depending on the availability of water and the climate. Placement of a foundation on the soil may eliminate or reduce evaporation of moisture from the ground surface and eliminate transpiration of moisture from previously existing vegetation. Figure 7-32 illustrates three methods described below for estimating the equilibrium pore water pressure profile u_{wf} in units of tsf. If undisturbed soil specimens are taken from the field near the end of the dry season, then the maximum potential heave may be estimated from results of swell tests performed on these specimens.

7.5.2.1.3.1. Saturated profile (Method 1, Figure 7-32)

The equilibrium pore water pressure in the saturated profile within depth Z_a is

Equation 7-52: $u_{wf} = 0$

This profile is considered realistic for most practical cases including houses or buildings exposed to watering of perimeter vegetation and possible leaking of underground water and sewer lines. Water may also condense or collect in permeable soil beneath foundation slabs and penetrate into underlying expansive soil unless drained away or protected by a moisture barrier. This profile should be used if other information on the equilibrium pore water pressure profile is not available.

7.5.2.1.3.2. Hydrostatic with shallow water table (Method 2, Figure 7-32)

The equilibrium pore water pressure in this profile is zero at the groundwater level and decreases linearly with increasing distance above the groundwater level in proportion to the unit weight of water

Equation 7-53: $u_{wf} = \gamma_w (z - Z_a)$

Where

- γ_w = unit weight of water, 0.031 ton/ft^3
- z = depth below the foundation, ft

This profile is considered realistic beneath highways and pavements where surface water is drained from the pavement and where underground sources of water such as leaking pipes or drains do not exist. This assumption leads to smaller estimates of anticipated heave than Method 1.

7.5.2.1.3.3. Hydrostatic without shallow water table (Method 3, Figure 7-32)

The pore water pressure of this profile is similar to Method 2, but includes a value of the negative pore water pressure u_{wa} at depth Z_a.

Equation 7-54: $$u_{wf} = u_{wa} + \gamma_w (z - Z_a)$$

u_{wa} may be evaluated by methodology described in TM 5-818-7.

7.5.2.2. Identification

In addition to the laboratory tests described in 7.8.3, soils susceptible to swelling can be most easily identified by simple classification tests such as Atterberg limits and natural water content. Two equations that have provided reasonable estimates of free swell are[208]

Equation 7-55: $$\log S_f = 0.0367LL + 0.0833W_n + 0.458$$

and[209]

Equation 7-56: $$S_f = 2.27 + 0.131LL - 0.27W_n$$

Where

- S_f = free swell, percent
- LL = liquid limit, percent
- W_n = natural water content, percent

The percent swell under confinement can be estimated from the free swell by[210]

Equation 7-57: $$S = S_f \left(1 - 0.72\sqrt{\sigma_f}\right)$$

Where

- S = swell under confinement, percent
- σ_f = vertical confining pressure, tsf

These identification procedures were developed by correlations of classification test results with results of one-dimensional swell tests performed in consolidometers on undisturbed and compacted soil specimens.

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[208] Johnson, L. D. and Snethen, D. R. 1979. "Prediction of Potential Heave of Swelling Soils," *Geotechnical Testing Journal*, Vol 1, pp 117-124 and Vijayvergiya, V. N. and Ghazzaly, O. I. 1973, "Prediction of Swelling Potential for Natural Clays," Proceedings of the Third International Conference on Expansive Clay Soils, Vol 1. Available from Jerusalem Academic Press, Jerusalem.

[209] O'Neill, M. W. and Ghazzaly, O. I. 1977. "Swell Potential Related to Building Performance," Journal of geotechnical engineering Division, Vol 103, pp 1363-1379. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

[210] Gogoll, F. H. 1970. "Foundations in Swelling Clay Beneath a Granular Blanket," Proceedings Symposium on Soils and Earth Structures in Arid Climates, Adelaide, Australia, pp 42-48. Available from Institution of Engineers, Miadna Pty. Ltd., P.O. Box 588, Crons Nest, N5W 2065, Australia.

Soils with liquid limit less than 35% and plasticity index less than 12% have relatively low potential for swell and may not require swell testing.

7.5.2.3. Potential Vertical Heave

Useful estimates of the anticipated heave based on results from consolidometer swell tests can often be made.

7.5.2.3.1. Selection of Suitable Test Method

Suitable standard test methods for evaluating the potential for one dimensional heave or settlement of cohesive soils are fully described in 7.7.10. A brief review of three one-dimensional consolidometer tests useful for measuring potential swell or settlement using a standard consolidometer is provided below.

7.5.2.3.1.1. Free swell

After a seating pressure (e.g., 0.01 tsf applied by the weight of the top porous stone and load plate) is applied to the specimen in a consolidometer, the specimen is inundated with water and allowed to swell vertically until primary swell is complete. The specimen is loaded following primary swell until its initial void ratio/height is obtained. The total pressure required to reduce the specimen height to the original height prior to inundation is defined as the swell pressure σ_s.

7.5.2.3.1.2. Swell overburden

After a vertical pressure exceeding the seating pressure is applied to the specimen in a consolidometer, the specimen is inundated with water. The specimen may swell, swell then contract, contract, or contract then swell. The vertical pressure is often equivalent to the *in situ* overburden pressure and may include structural loads depending on the purpose of the test.

7.5.2.3.1.3. Constant volume

After a seating pressure and additional vertical pressure, often equivalent to the *in situ* overburden pressure, is applied to the specimen in a consolidometer, the specimen is inundated with water. Additional vertical pressure is applied as needed or removed to maintain a constant height of the specimen. A consolidation test is subsequently performed. The total pressure required to maintain a constant height of the specimen is the measured swell pressure. This measured swell pressure is corrected to compensate for sample disturbance by using the results of the subsequent consolidation test. A suitable correction procedure is similar to that for estimating the maximum past pressure.

7.5.2.3.2. Calculation from Void Ratio

The anticipated heave is

Equation 7-58: $$S_{\max} = \sum_{j=1}^{n} S_{\max j} = \sum_{j=1}^{n} \frac{e_{fj} - e_{oj}}{1 + e_{oj}} H_j$$

Where

- S_{max} = maximum potential vertical heave, ft
- n = number of strata within the depth of heaving soil
- S_{maxj} = heave of soil in stratum j, ft
- H_j = thickness of stratum j, ft
- e_{fj} = final void ratio of stratum j

- e_{oj} = initial void ratio of stratum j

The initial void ratio, which depends on a number of factors such as the maximum past pressure, type of soil, and environmental conditions, may be measured by standard consolidometer test procedures described in 7.7.10. The final void ratio depends on changes in soil confinement pressure and water content following construction of the structure; it may be anticipated from reasonable estimates of the equilibrium pore water pressure u_{wf}, depth of active zone Z_a, and edge effects by rewriting Equation 7-58 in terms of swell pressure shown in Equation 7-59 below.

7.5.2.3.3. Calculation from Swell Pressure

The anticipated heave in terms of swell pressure is

Equation 7-59: $$S_{max} = \sum_{j=1}^{n} \frac{C_{sj}}{1+e_{oj}} \log_{10}\left(\frac{\sigma_{sj}}{\sigma'_{fj}}\right) H_j$$

Where

- C_{sj} = swell index of stratum j
- σ_{sj} = swell pressure of stratum j, tsf
- σ'_{fj} = final or equilibrium average effective vertical pressure of stratum j, $\sigma_{fj} - u_{wfj}$, tsf
- σ_{fj} = final average total vertical pressure of stratum j, tsf
- u_{wfj} = average equilibrium pore water pressure in stratum j, tsf

The number of strata n required in the calculation is that observed within the depth of the active zone for heave.

7.5.2.3.3.1. Swell index

The swell or rebound index of soil in each stratum may be determined from results of consolidometer tests as described in 7.7.10. Preliminary estimates of the swell index may be made from Figure 7-16.

a) The swell index C_s measured from a swell overburden test (Swell Test described in 7.7.10 or Method B described in ASTM D 4546) may be less than that measured from a constant volume test (Swell Pressure Test described in 7.7.10 or Method C described in ASTM D 4546). The larger values of C_s are often more appropriate for analysis of potential heave and design.

b) A simplified first approximation of C_s developed from Corps of Engineer project sites through Central Texas is $C_s \approx 0.03 + 0.002(LL-30)$.

7.5.2.3.3.2. Swell pressure

The swell pressure of soil in each stratum may be found from results of consolidometer swell tests on undisturbed specimens as described in 7.7.10. Preliminary estimates of swell pressure may be made from[211]

Equation 7-60: $$\log_{10}\sigma_s = 2.1423 + 0.0208LL + 0.01065\gamma_d - 0.0269W_n$$

where σ_s = swell pressure, tsf γ_d = dry density, lbs/ft^3

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[211] Komornik, A. and David, D. 1969. "Prediction of Swelling Pressure of Clays," *Journal of the Soil Mechanics and Foundations Division*, Vol 95, pp 209-225. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

An alternative equation[212] is

$$\textbf{Equation 7-61: } \sigma_s = 0.028 PI^{1.12}\left[\frac{C}{W_n}\right]^2 + 0.273$$

Where

- PI = plasticity index, percent
- C = clay content, percent less than 2 microns

7.5.2.3.3.3. Final effective vertical pressure

The final total pressure σ_f may be estimated from the sum of the increase in soil stresses from the structural loads calculated and the initial overburden pressure σ_o. The final effective pressure σ'_f is σ_f less the assumed equilibrium pore water pressure profile u_{wf}, Figure 7-32.

7.5.2.4. Potential Differential Heave

Differential heave results from edge effects beneath a finite covered area, drainage patterns, lateral variations in thickness of the expansive foundation soil, and effects of occupancy. The shape, geometry, and loads of the structure also promote differential movement. Examples of the effect of occupancy include broken or leaking underground water lines and irrigation of vegetation adjacent to the structure. Other causes of differential heave include differences in distribution of loads and footing sizes.

7.5.2.4.1. Predictability of Variables

Reliable estimates of the anticipated differential heave and location of differential heave are not possible because of uncertainty in such factors as future availability of moisture, horizontal variations in soil parameters, areas of soil wetting, and effects of future occupancy.

7.5.2.4.2. Magnitude of Differential Heave

The difference in potential heave between locations beneath a foundation can vary from zero to the maximum potential vertical heave. Differential heave is often the anticipated total heave for structures on isolated spot footings or drilled shafts because soil beneath some footings or portions of slab foundations may experience no wetting and no movement. Refer to 7.2 for details on effect of differential movement on performance of the foundation.

1) A reasonable estimate of the maximum differential movement or differential potential heave ΔS_{max} is the sum of the maximum calculated settlement ρ_{max} of soil beneath a nonwetted point of the foundation and the maximum potential heave S_{max} following wetting of soil beneath some adjacent point of the foundation separated by the distance. If all of the soil heaves, then ΔS_{max} is the difference between S_{max} and S_{min} between adjacent points where S_{min} is the minimum heave.
2) The location of S_{max} may be beneath the most lightly loaded portion of the foundation such as beneath the centre of the slab.
3) The location of ρ_{max} may be beneath columns and consist only of immediate elastic settlement ρ_i in soil where wetting does not occur or will be S_{min} if wetting does occur in expansive soil.

[212] Nayak, N. V. and Christensen, R. W. 1971. "Swelling Characteristics of Compacted Expansive Soil," Clays and Clay Minerals, Vol 19, pp 251-261. Available from Allen Press, Inc., 1041 New Hampshire Street, Box 368, Lawrence, KS 66044.

4) The deflection ratio is $\Delta S_{max}/L$ where L may be the distance between stiffening beams.

7.5.2.5. Application

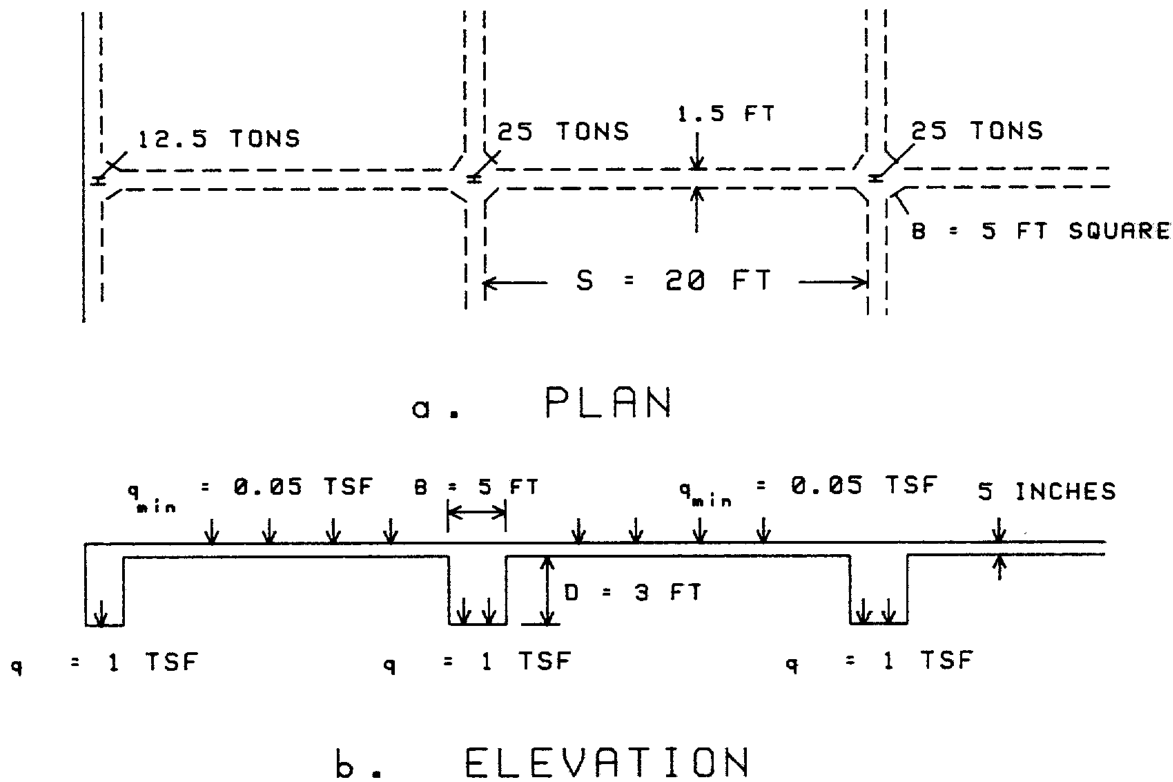
A stiffened ribbed mat is to be constructed on an expansive soil. The soil parameters illustrated in Table 7-24 were determined on specimens of an undisturbed soil sample taken 10 ft beneath the mat. Additional tests at other depths will improve reliability of these calculations.

Table 7-24 Soil Parameters for Example Estimation of Anticipated Heave

Parameter	*Value*
Elastic modulus E_s, tsf	200
Swell Pressure σ_s, tsf	1.0
Compression index C_c	0.25
Swell index C_s	0.10
Initial void ratio e_o	0.800
Unit wet soil weight γ, ton/ft^3	0.06
Active zone for heave Z_a, ft	20

The active zone for heave is estimated to extend 20 ft below ground surface or 20 ft below the base of the mat and 17 ft below the base of the columns. The maximum anticipated heave S_{max} and differential heave ΔS_{max} are to be estimated beneath portions of the mat. Stiffening beams are 3 ft deep with 20-ft spacing in both directions, Figure 7-33. Column loads of 25 tons interior and 12.5 tons perimeter lead to an applied pressure on the column footings q = 1.0 tsf. Minimum pressure q_{min} beneath the 5”-thick-flat slab is approximately 0.05 tsf. The heave calculations assume a zero stiffness mat.

Figure 7-33 Plan and elevation of stiffened mat in expansive soil



7.5.2.5.1. Calculation of Potential Heave

(1) Maximum potential heave S_{max}. The maximum heave is anticipated beneath unloaded portions of the mat. The potential heave is estimated assuming the equilibrium pore water pressure $u_{wf} = 0$ or the soil is saturated; therefore, the final effective pressure $\sigma'_f = \sigma_f$ or the final total pressure.

a. Table 7-25a illustrates the estimation of anticipated heave S_{max} beneath lightly loaded portions of the mat using Equation 5-4b, increment thickness ΔH = 2 ft, and results of a single consolidometer swell test.

b. Table 7-25a and Figure 7-34a show that S_{max} = 0.3 ft or 3.6 inches and that heave is not expected below 16 ft of depth where the swell pressure approximately equals the total vertical pressure σ_f.

c. Most heave occurs at depths less than 5 ft below the flat portion of the mat. Replacing the top 4 ft of expansive soil with nonexpansive backfill will reduce S_{max} to 0.115 ft or 1.4 inches, Table 7-25a and Figure 7-34a.

(2) Minimum potential heave S_{min}. The minimum potential heave on wetting of the soil to a saturated profile (Method 1, Figure 7-32) is expected beneath the most heavily loaded portions of the mat or beneath the columns. Table 7-25b and Figure 7-34b show that the minimum heave S_{min} calculated after Equation 7-59 substituting S_{min} for S_{max} is 0.092 ft beneath the column or S_{min} is 0.092 ft or 1.1 inches beneath the column. Heave is not expected below 13 ft beneath the columns.

Table 7-25 Heave Calculations for Example Application

a. Beneath Slab

Depth z, ft	*Overburden Pressure $\sigma_o = \gamma z$, tsf*	*Total Pressure $\sigma_f = \sigma'_f$, tsf*	*S_{maxj}, ΔH*	*S_{maxj}, ft*	*S_{max}, ft*
0	0.00	0.05	0.072		0.303
				0.115	
2	0.12	0.17	0.043		0.188
				0.073	
4	0.24	0.29	0.030		0.115
				0.052	
6	0.36	0.41	0.022		0.063
				0.037	
8	0.48	0.53	0.015		0.026
				0.025	
10	0.60	0.65	0.010		0.001
				0.016	
12	0.72	0.77	0.006		-0.015
				0.009	
14	0.84	0.89	0.003		-0.024
				0.003	
16	0.96	1.01	0.000		-0.027
				-0.007	
18	1.08	1.13	-0.007		-0.020
				-0.020	
20	1.20	1.25	-0.013		0.000

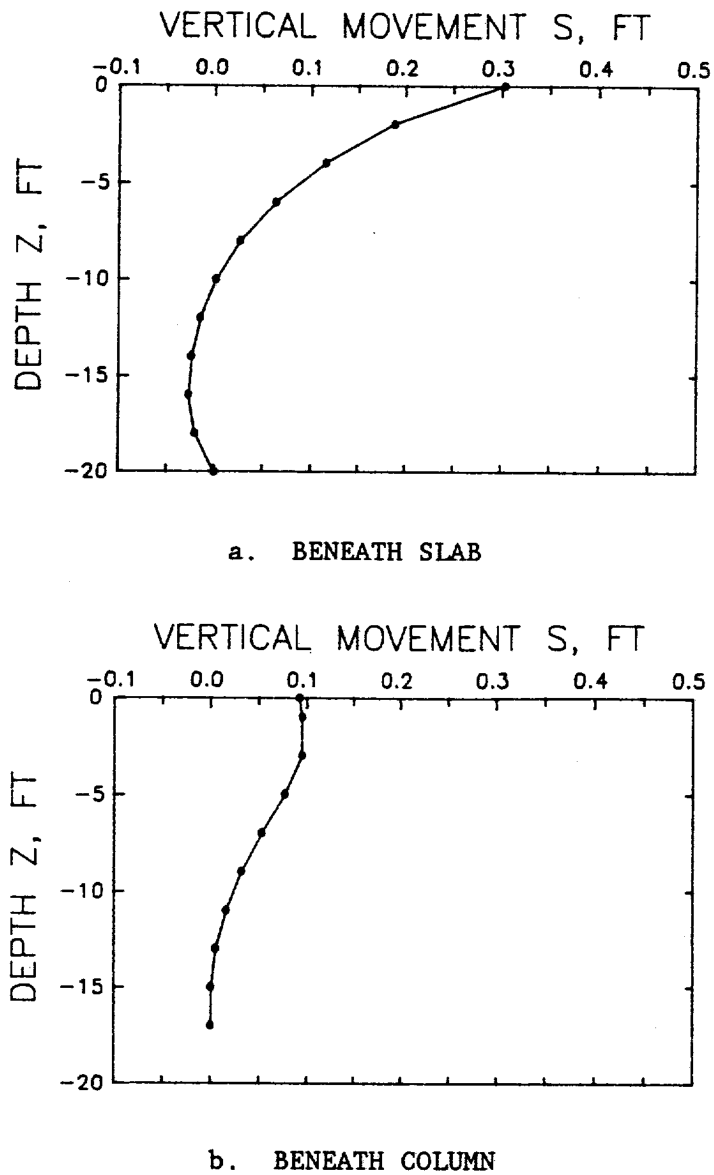
b. Beneath Columns

Depth z, ft	*z/B*	*Overburden Pressure $\sigma_o = \gamma z$, tsf*	*Column Pressure[213] $\Delta\sigma_z$, tsf*	*Total Pressure $\sigma_f = \sigma'_f$, tsf*	*$S_{minj}/\Delta H$*	*S_{minj}, ft*	*S_{min}, ft*
0	0.0	0.00	1.00	1.00	0.000		0.092
						-0.003	
1	0.2	0.06	0.96	1.02	-0.006		0.095

[213] Increase in pressure beneath columns calculated from Boussinesq assumptions.

						0.000	
3	0.6	0.18	0.61	0.79	0.006		0.095
						0.018	
5	1.0	0.30	0.30	0.60	0.012		0.077
						0.024	
7	1.4	0.42	0.20	0.62	0.012		0.053
						0.021	
9	1.8	0.54	0.16	0.70	0.009		0.032
						0.016	
11	2.2	0.66	0.09	0.75	0.007		0.016
						0.011	
13	2.6	0.78	0.07	0.85	0.004		0.005
						0.005	
15	3.0	0.90	0.05	0.95	0.001		0.000
						0.000	
17	3.4	1.02	0.04	1.06	-0.001		0.000

Figure 7-34 Calculated heave profile beneath mat foundation



7.5.2.5.2. Maximum Differential Heave ΔS_{max}

(1) ΔS_{max} is the sum of S_{max} and the immediate settlement ρ_i if soil wetting is nonuniform. The maximum immediate settlement ρ_i is anticipated to occur as elastic settlement beneath the loaded columns if soil wetting does not occur in this area. A common cause of nonuniform wetting is leaking underground water lines. From the improved Janbu approximation, Equation 7-23 and Figure 7-10, with reference to Figure 7-33

$$\mu_o = 0.92, \ \frac{D}{B} = 1.0$$

$$\mu_1 = 0.7\ , \ \frac{L}{B} = 1.0 \text{ and } \frac{H}{B} > 10$$

$$\rho_1 = \mu_0 \mu_1 \left[\frac{qB}{E_s} \right] = 0.92 \quad 0.7 \quad \frac{1.0 \quad 5.0}{200}$$

The maximum differential heave $\Delta S_{max} = S_{max} + \rho_i = 3.6 + 0.2 = 3.8$ inches or 0.317 ft. The deflection ratio Δ/L is $\Delta S_{max}/L = 0.317/20$ or 1/64 where L is 20 ft, the stiffening beam spacing. This deflection ratio cannot be tolerated, 7.2. If the top 6 ft of expansive soil is replaced with nonexpansive backfill $\Delta S_{max} = 0.063 + 0.016 = 0.079$ ft or 0.95 inch. Ribbed mat foundations and superstructures may be designed to accommodate differential heave of 1 inch[214].

(2) ΔS_{max} is the difference between S_{max} and S_{min} if soil wetting occurs beneath the columns or 3.6 - 1.4 = 2.2 inches. Replacement of the top 4 ft of soil beneath the ribbed mat will reduce this differential heave to about 1.4 - 1.1 or about 0.3 inch ignoring the difference in settlement beneath the fill and original expansive soil within 1 ft beneath the column.

7.5.3. Collapsible Soil

7.5.3.1. General

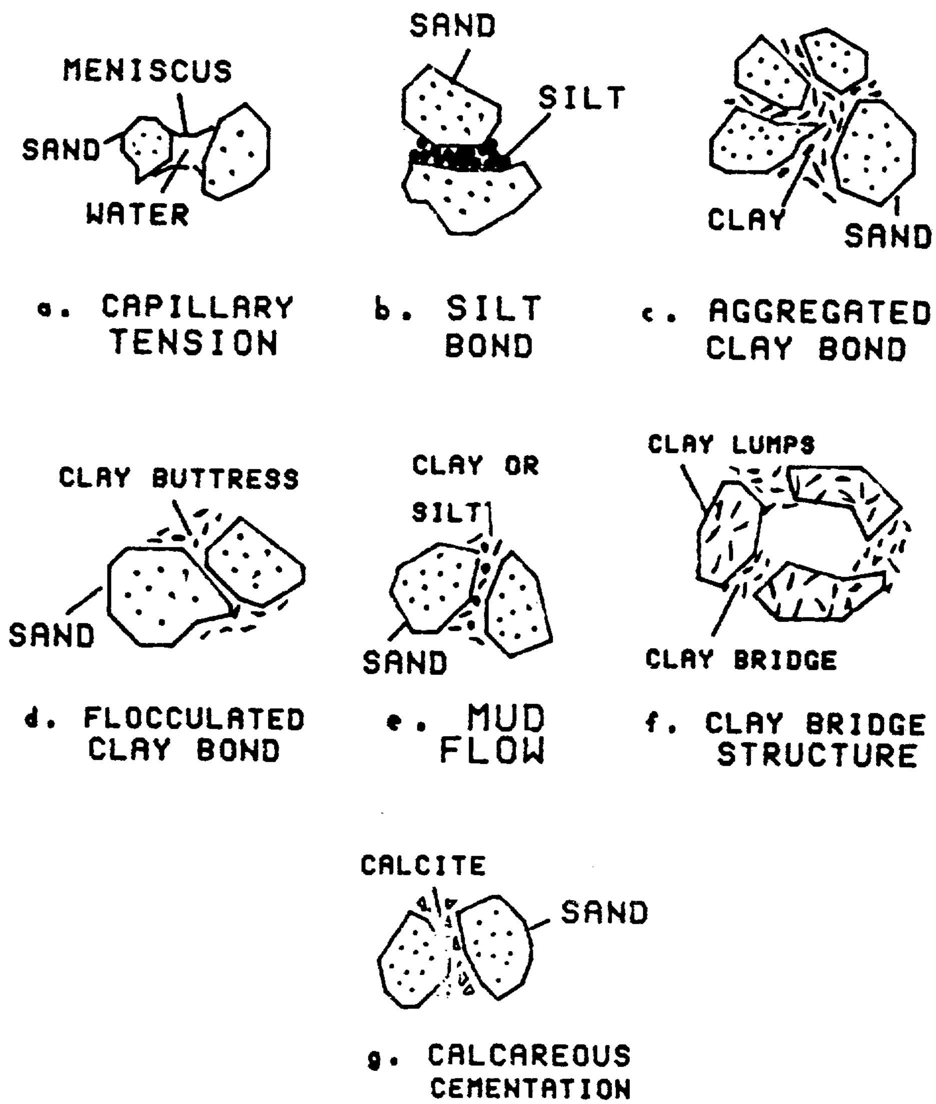
Many collapsible soils are mudflows or windblown silt deposits of loess often found in arid or semiarid climates such as deserts, but dry climates are not necessary for collapsible soil. Loess deposits cover parts of the Western, Midwestern, and Southern United States, Europe, South America, Asia including large areas of Russia and China, and Southern Africa. A collapsible soil at natural water content may support a given foundation load with negligible settlement, but when water is added to this soil the volume can decrease significantly and cause substantial settlement of the foundation, even at relatively low applied stress or at the overburden pressure. The amount of settlement depends on the initial void ratio, stress history of the soil, thickness of the collapsible soil layer, and magnitude of the applied foundation pressure. Collapsible soils exposed to perimeter watering of vegetation around structures or leaking utility lines are most likely to settle. Collapse may be initiated beneath the ground surface and propagate toward the surface leading to sudden and nonuniform settlement of overlying facilities.

7.5.3.1.1. Structure

Soils subject to collapse have a honeycombed structure of bulky shaped particles or grains held in place by a bonding material or force illustrated in Figure 7-35. Common bonding agents include soluble compounds such as calcareous or ferrous cementation that can be weakened or partly dissolved by water, especially acidic water. Removal of the supporting material or force occurs when water is added enabling the soil grains to slide or shear and move into voids.

[214] Johnson, L. D. 1989. "Design and Construction of Mat Foundations," Miscellaneous Paper GL-89-27. Available from Research Library, US Army Engineer Waterways Experiment Station, Vicksburg, MS 39180.

Figure 7-35 Mechanisms for collapse of loose, bulky grains



7.5.3.1.2. Collapse Trigger

Table 7-26 illustrates four types of wetting that can trigger the collapse of soil. Dynamic loading may also cause a shear failure in the bonding material and induce collapse. This mechanism is particularly important for roads, airfields, railways, foundations supporting vibrating machinery, and other foundations subject to dynamic forces.

Table 7-26 Wetting That Can Trigger Soil Collapse

Type of Wetting	*Description*
Local, shallow	Wetting of a random nature caused by water sources from pipelines or uncontrolled drainage of surface water; no rise in groundwater level; settlement occurs in upper soil layer within wetted area
Intense local	Intense deep, local wetting caused by discharge of deep industrial effluent, leaking underground utility lines, or irrigation. Flow rates sufficient to

	cause a continuous rise in groundwater level may saturate the entire zone of collapsible soil within a short time (i.e., < 1 year) and cause uneven and damaging settlement under existing structural loads or only the soil weight
Slow, uniform	Slow, relatively uniform rise of groundwater from sources rise in outside of the collapsible soil area will cause uniform and groundwater gradual settlement
Slow increase	Gradual increase in water content of thick collapsible soil in water layer from steam condensation or reduction in evaporation content from the ground surface following placement of concrete or asphalt will cause incomplete settlement

7.5.3.2. Identification

Typical collapsible soils are lightly coloured, low in plasticity with liquid limits below 45, plasticity indices below 25, and relatively low dry densities between 65 and 105 lbs/ft^3 (60 to 40% porosity). Collapse rarely occurs in soil with porosity less than 40%. Most past criteria for determining the susceptibility of collapse are based on relationships between the void ratio, water content, and dry density, Table 7-27. The methods in Table 7-27 apply to fine-grained soil.

Table 7-27 Relationships for Estimating Susceptibility of Soil to Collapse

Source	***Soil Susceptible to Collapse***	
Northey 1969[215]	Denisov introduced a coefficient of subsidence $k_d = e_{LL}/e_o$; the soil is collapsible if	
	$0.5 < k_d < 0.75$	Where
		∞ e_{LL} = void ratio at liquid limit
		∞ LL = liquid limit, percent
		∞ e_o = natural void ratio
After Gibbs and Bara 1962[216]	$\gamma_d < 162.3/(1 + 0.026LL)$	Where
	or $e_o > 2.6LL/100$	∞ γ_d = natural dry density, lbs/ft^3

[215] Northey, R. D. 1969. "Collapsing Soils: State of the Art," Seventh International Conference of Soil Mechanics and Foundation Engineering, Vol 5, pp 445. Available from Sociedad Mexicana de Mecanica de Suelos, A. C., Mexico City, Mexico.

[216] Gibbs, H. J. and Bara, J. P. 1962. "Predicting Surface Subsidence From Basic Soil Tests," Field Testing of Soils, Special Technical Publication No. 322, pp 231-247. Available from American Society for Testing and Materials, 1916 Race Street, Philadelphia, PA 19103.

Feda 1966[217]	$$\frac{\frac{e_o}{100G_s} - PL}{PI} > 0.85$$	∞ PL = plastic limit, percent ∞ PI = plasticity index, percent ∞ G_s = specific gravity of soil
Jennings and Knight 1975[218]	Measure of collapse potential CP of a specimen tested in a one-dimensional consolidometer in percent $$CP = 100\frac{e_o - e_c}{1 + e_o}$$	∞ e_o = void ratio at $\sigma = 2$ tsf at natural water content prior to wetting ∞ e_c = void ratio after soaking at σ - 2 tsf
	CP Percent	***Severity of Collapse***
	0 - 1	Negligible
	1 - 5	Moderate trouble
	5 - 10	Trouble
	10 - 20	Severe trouble
	> 20	Very severe trouble

(1) The Gibbs and Bara method assumes collapse of soil with sufficient void space to hold the liquid limit water.

(2) Fine-grained soils that are not susceptible to collapse by the criteria in Table 7-27 may have potential for expansion.

7.5.3.3. Potential Collapse

When water becomes available to collapsible soil, settlement in addition to elastic settlement will occur without any additional applied pressure. This settlement will occur quickly in a free draining or pervious soil, but more slowly in a poor draining or less pervious soil. When construction occurs on soil where surface water filters through the collapsible soil over time, some collapse will occur *in situ* and reduce collapse that will occur on wetting following construction. Procedures for estimating the potential for collapse are uncertain because no single criterion can be applied to all collapsible soil. The amount of settlement depends on the extent of the wetting front and availability of water, which rarely can be predicted prior to collapse. Laboratory classification and consolidation tests can fail to indicate soil that eventually does collapse in the field. The following procedures to estimate collapse attempt to follow the stress path to which the soil will be subjected in the field. Immediate settlement prior to collapse may be estimated by methods in 7.3.

[217] Feda, J. 1966. "Structural Stability of Subsident Loess Soil From Prahadejuice," Engineering Geology, Vol 1, pp 201-219. Available from Elsevier Science Publishers B. V., Box 211, 1000 AE, Amsterdam, The Netherlands.

[218] Jennings, J. E. and Knight, K. 1975. "A Guide to Construction on or With Materials Exhibiting Additional Settlement Due to 'Collapse' of Grain Structure," Proceedings Sixth Regional Conference for Africa on Soil Mechanics and Foundation Engineering, pp 99-105, Durban. Available from A. A. Balkema, P.O. Box 1675, Rotterdam, The Netherlands.

7.5.3.3.1. Wetting at Constant Load

An acceptable test procedure is described in detail as Method B of ASTM D 4546 or 7.7.10. A specimen is loaded at natural water content in a consolidometer to the anticipated stress that will be imposed by the structure in the field. Distilled water (or natural site water if available) is added to the consolidometer and the decrease in specimen height following collapse is noted. The settlement of collapsible soil may be estimated by

$$\textbf{Equation 7-62: } \rho_{col} = \frac{e_o - e_c}{1 + e_o} H$$

Where

- ρ_{col} = settlement of collapsible soil stratum, ft
- e_o = void ratio at natural water content under anticipated vertical applied pressure σ_f
- e_c = void ratio following wetting under σ_f
- H = thickness of collapsing soil stratum, ft

The total settlement of the soil will be the sum of the settlement of each stratum.

7.5.3.3.2. Modified Oedometer Test[219]

This test is a modification of the Jennings and Knight double oedometer procedure that eliminates testing of two similar specimens, one at natural water content and the other inundated with distilled (or natural) water for 24 hr.

7.5.3.3.2.1. Procedure

An undisturbed specimen is prepared and placed in a one-dimensional consolidometer at the natural water content. The initial specimen height h is recorded. A seating pressure of 0.05 tsf is placed on the specimen and the dial gauge is zeroed (compression at stress levels less than 0.05 tsf is ignored).

Within 5 minutes, the vertical stress is increased in increments of 0.05, 0.1, 0.2, 0.4 tsf, etc. until the vertical stress is equal to or slightly greater than that expected in the field following construction. For each increment, dial readings are taken every 1/2 hr until less than 0.1% compression occurs in 1 hr. The specimen is subsequently inundated with distilled (or natural) water and the collapse observed on the dial gauge is recorded.

Dial readings are monitored every 1/2 hr at this stress level until less than 0.1% compression occurs in 1 hr. Additional stress is placed on the specimen in increments as previously described until the slope of the curve is established. The dial readings d are divided by the initial specimen thickness h_o and multiplied by 100 to obtain percent strain. The percent strain may be plotted as a function of the applied pressure as shown in Figure 5-5 and a dotted line projected from point C to point A to approximate the collapse strain for stress levels less than those tested.

7.5.3.3.2.2. Calculation of collapse

The soil profile should be divided into different layers with each layer corresponding to a representative specimen such as illustrated in Figure 7-37. The initial and final stress distribution should be calculated for each layer and entered in the compression curve such as Figure 7-36 and the vertical strain recorded at

[219] Houston, S. L., Houston, W. N., and Spadola, D. J. 1988. "Prediction of Field Collapse of Soils Due to Wetting," Journal of geotechnical engineering, Vol 114, pp 40-58. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

the natural water content and the inundated water content. The settlement is the difference in strain between the natural water content and wetted specimen at the same stress level

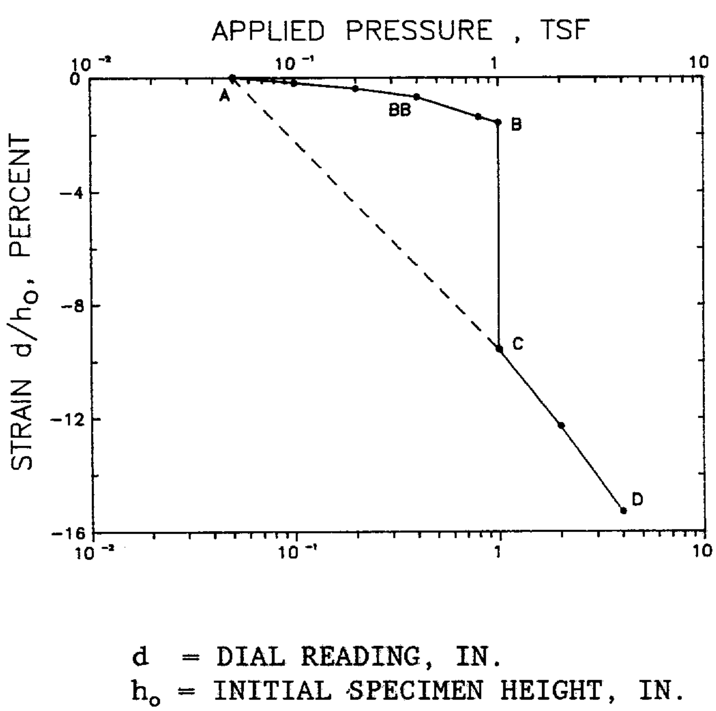
$$\textbf{Equation 7-63: } \rho_{col} = \left[\left(\frac{d}{h_o}\right)_r - \left(\frac{d}{h_o}\right)_o\right]\frac{H}{100}$$

Where

- ρ_{col} = collapse settlement, ft
- $(d/h_o)_f\,100$ = strain after wetting at the field stress level, percent
- $(d/h_o)_o\,100$ = strain at natural water content at the field stress level, percent
- d = dial reading, in.
- h_o = initial specimen height, in.
- H = thickness of collapsible stratum, ft

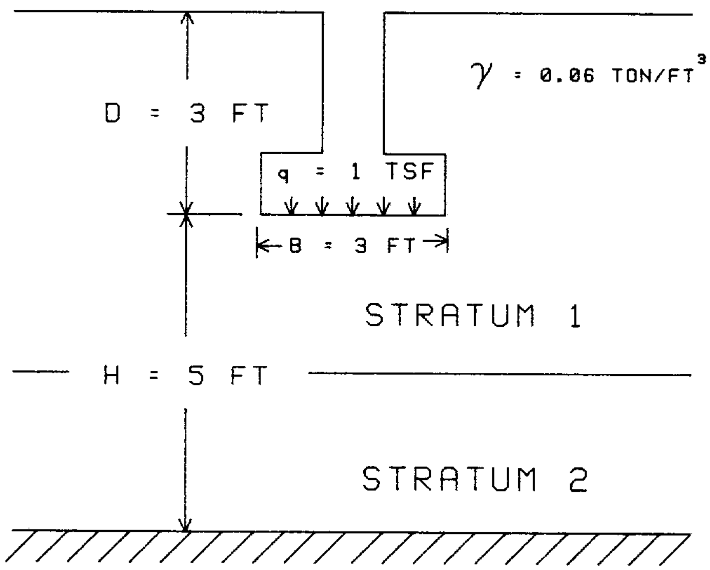
Total settlement is the sum of the collapse settlement of each stratum.

Figure 7-36 Example Compression curve of the Modified Oedometer Test[220]



[220] d/h_o is multiplied by 100 to obtain percent

Figure 7-37 Illustration of footing for calculating settlement on collapsible soil



7.5.3.4. Application

A 3-ft square footing illustrated in Figure 7-37 is to be placed 3-ft deep on a loess soil with a thickness of 5 ft beneath the footing. The results of a modified oedometer test performed on specimens of this soil are provided in Figure 7-36. The footing pressure q = 1 tsf.

7.5.3.4.1. Calculation

Table 7-28 illustrates computation of the vertical stress distribution and collapse settlement at the centre and corner of this footing. The stress levels and vertical strains of the soil in Figure 7-36 are shown in Table 7-28b assuming layers 1 and 2, Figure 7-37, consist of the same soil. The average settlement of (4.5 + 4.0)/2 = 4.3 inches should provide a reasonable estimate of the settlement of this footing.

7.5.3.4.2. Testing Errors

The amount of collapse depends substantially on the extent of the wetting front and initial negative pore water or suction pressure in the soil, which may not be duplicated because soil disturbance and lateral pressures may not be simulated. Collapse may also be stress path dependent and may involve a mechanism other than addition of water such as exposure to dynamic forces.

Table 7-28 Example Calculation of Settlement of a Collapsible Soil Beneath a Square Footing (Figure 7-37)

a. Stress Distribution

Depth Below Footing z, ft	*B/2z*	*Overburden Pressure σ_o, tsf*	*Influence* Factor I_σ[221]*	*Footing Stress q_z, tsf*[222]	*Total Stress σ_{fz}, tsf*

[221] From Figure C-2 where m = n = for the centre and m = n = for the corner.

[222] Centre: $q_z = 4q\ I$; Corner: $q_z = q\ I$; q = 1 tsf

			Centre	Corner	Centre	Corner	Centre	Corner
0.0	∞	0.18	0.250	0.250	1.000	0.250	1.180	0.430
2.5	0.6	0.33	0.106	0.195	0.424	0.195	0.754	0.525
5.0	0.3	0.48	0.038	0.106	0.152	0.106	0.632	0.586

b. Settlement

Depth Below Footing z, ft	Layer	Average Final Stress σ_{fz}, tsf		Average Initial Strain, Percent		Average Final Strain, Percent	
		Centre	Corner	Centre	Corner	Centre	Corner
1.25	1	0.967	0.478	1.55	0.85	9.45	7.30
3.75	2	0.693	0.555	1.25	1.05	8.35	7.75

Settlement from Equation 5-7:

Centre: $\rho_{col} = \left[(9.45 - 1.55) + (8.35 - 1.250)\right]2.5 = 0.375' = 4.5''$

Column: $\rho_{col} = \left[(7.30 - 0.85) + (7.75 - 1.05)\right]2.5 = 0.329' = 4.0''$

7.6. Coping with Soil Movements

7.6.1. Minimizing and Tolerating Soil Movements

7.6.1.1. General

Development of society leads increasingly to construction on marginal (soft, expansive, collapsible) soil subject to potential volume changes. Sufficient soil exploration and tests are necessary to provide reliable soil parameters for evaluating reasonable estimates of total and differential settlement.

7.6.1.1.1. Exploratory Borings

Exploratory borings should be made within soil areas supporting the structure and sufficient tests performed to determine upper and lower limits of the soil strength, stiffness, and other required parameters. Depth of borings should be sufficient to include the significantly stressed zones of soil from overlying structures. These depths should be twice the minimum width of footings or mats with length to width ratios less than two, four times the minimum width of infinitely long footings or embankments, or to the depth of incompressible strata, whichever is least.

7.6.1.1.2. Mitigation for Excessive Deformation Potential

If analysis by methods in this book indicates excessive settlement or heave of the supporting soil, then the soil should be improved and/or various design measures should be applied to reduce the potential volume changes and foundation movements to within tolerable limits.

7.6.1.2. Soil Improvement

Most foundation problems occur from high void ratios, low strength materials and unfavourable water content in the soil; therefore, basic concepts of soil improvement include densification, cementation, reinforcement, soil modification or replacement, drainage, and other water content controls. A summary with description of soil improvement methods is shown in Table 7-29. The range of soil particle sizes applicable for these soil improvement methods is shown in Table 7-30. Methods that densify soil by dynamic forces such as vibro-compaction and dynamic compaction (consolidation) may lead to a temporary, short-term reduction in strength of the foundation soil.

More details on dynamic and vibratory compaction of soils can be found in 10.6.

Table 7-29 Soil Improvement Methods[223]

Method	Principle	Most Suitable Soils and Types	Maximum Effective Treatment, Depth, ft	Advantages and Limitations
		Vibrocompaction		
Blasting	Shock waves cause lique-faction, displacement, remolding	Saturated, clean sands, partly saturated sands and silts after flooding	60	Rapid, low cost, treat small areas, no improvement near surface, dangerous,
Terra-probe	Densify by vertical vibration, liquefaction induced settle-ment under overburden	Saturated or dry clean sand (less effective in finer sand)	60 (ineffect-above 12-ft depth)	Rapid, simple, good under water, soft underlayers may damp vibra-tions, hard to penetrate over-layers
Vibratory rollers	Densify by vi-bration, lique-faction induced settlement under roller weight	Cohesionless soils	6 to 9	Best method for thin layers or lifts
Dynamic compaction (consoli-dation) or heavy tamping	Repeated high intensity impacts at the surface gives immediate settlement	Cohesionless soils best, other soils can be improved	45 to 60	Simple, rapid, must protect from personal injury and property damage from flying debris; ground-water must be > 6 ft below surface faster than pre-loading but less uniform
Vibro-flotation	Densify by horizontal vibration and compaction of backfill material	Cohesionless soil with less than 20 percent fines	90	Economical and effective in saturated and partly saturated granular soils

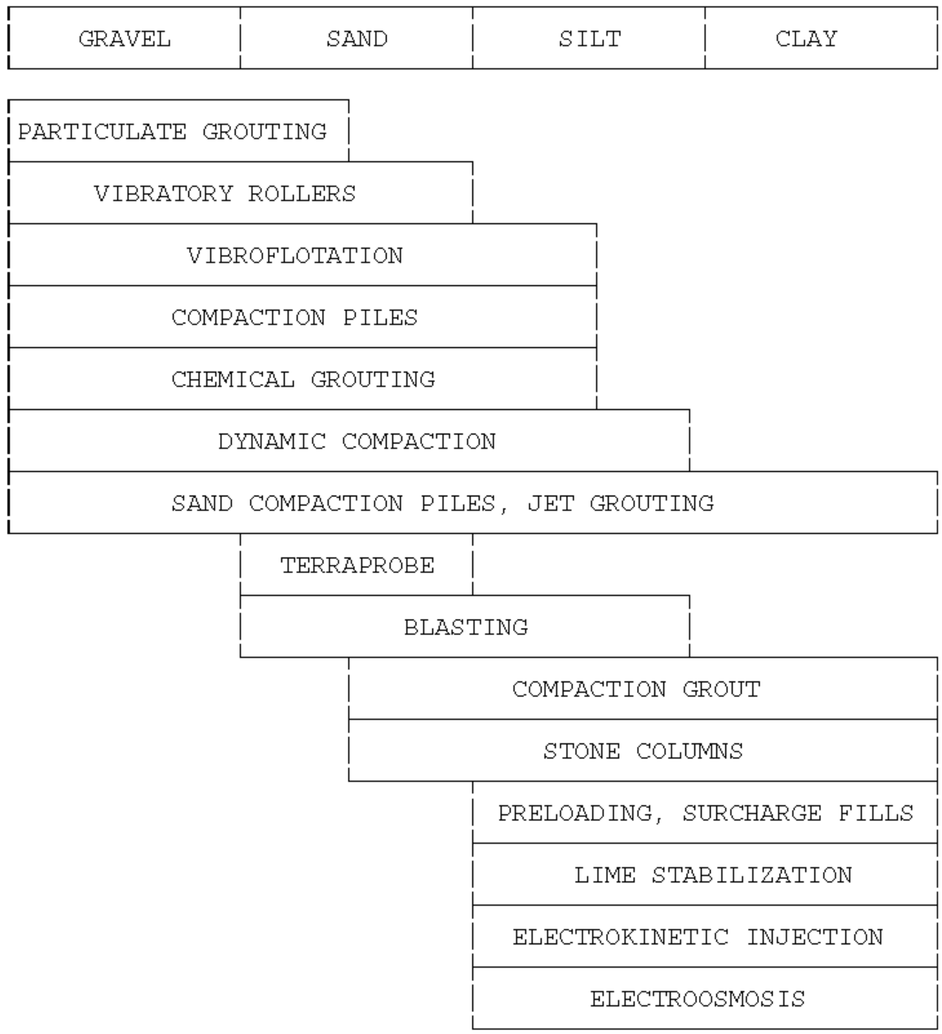
223 Committee on Placement and Improvement of Soils. 1978. "Soil Improvement-History, Capabilities, and Outlook," and 1987, "Soil Improvement - A Ten Year Update." Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Method	Principle	Most Suitable Soils and Types	Maximum Effective Treatment, Depth, ft	Advantages and Limitations
Hydro-compaction	Densify by vibration or repeated impact on surface of prewetted soil	Collapsible soil	< 10	Most effective method to densify silty loose collapsible sands
		Compaction Piles		
Compaction Piles	Densify by displacement of pile volume and by vibration during driving	Loose sandy soils, partly saturated clayey soils, loess	60 (limited improvement above 3 to 6)	Useful in soils with fines, uniform compaction, easy to check results, slow
Sand Compaction Piles	Sand placed in driven pipe; pipe partially withdrawn and redriven using vibratory hammer	All	-	Compressed air may be used to keep hole open as casing partially withdrawn
		Precompression		
Pre-loading	Load applied sufficiently in advance of construction to precompress soil	Normally consolidated soft clays, silts, organic deposits, landfills	-	Easy, uniform, long time required (use sand drains or strip drains to reduce time)
Surcharge Fills	Fill exceeding that required to achieve a given settlement; shorter time; excess fill removed	Same as for preloading	-	Faster than preloading without surcharge (use sand or strip drains to reduce time
Electro-osmosis	DC current causes water flow from anode towards cathode where it is removed	Normally consolidated silts and clays	30 - 60	No fill loading required; use in confined areas; fast; nonuniform properties between electrodes; useless in highly pervious soil

Method	Principle	Most Suitable Soils and Types	Maximum Effective Treatment, Depth, ft	Advantages and Limitations
		Reinforcement		
Mix-in-place Piles and Walls	Lime, cement or asphalt placed by rotating auger or in-place mixer	All soft or loose inorganic soils	> 60	Uses native soil; reduced lateral support required during excavation; difficult quality control
Strips and Membranes	Horizontal tensile strips or membranes buried in soil under footings	All	< 10	Increased allowable bearing capacity; reduced deformations
Vibro-replacement Stone	Hole jetted in soft, fine-grain soil and backfilled with densely compacted gravel	Very soft to firm soils (undrained strength 0.2 to 0.5 tsf)	60	Faster than precompression; avoids dewatering required for remove and replace; limited bearing capacity
Vibro-displacement Stone	Probe displaces soil laterally; backfill discharged through probe or placed in layers after probe removed	Soft to firm soils (undrained strength 0.3 to 0.6 tsf)	50	Best in low sensitivity soils with low groundwater
		Grouting and Injection		
Particulate Grouting	Penetration grout fills soil voids	Medium to Coarse sand and gravel	Unlimited	Low cost; grout high strength
Chemical Grouting	Solutions of 2 or more chemicals react in soil pores to form gel or soil precipitate	Medium silts and coarser	Unlimited	Low viscosity; controllable gel time; good water shutoff; high cost; hard to evaluate

Method	Principle	Most Suitable Soils and Types	Maximum Effective Treatment, Depth, ft	Advantages and Limitations
Moisture Barriers	Water access to foundation soil is minimized and more uniform	Expansive soil	15	Best for small structures and pavements; may not be 100 percent effective
Prewetting	Soil is brought to estimated final water content prior to construction	Expansive soil	6	Low cost; best for small, light structures; soil may still shrink and swell
Structural Fills	Structural fill distributes loads to under-lying soft soils	Soft clays or organic soils; marsh deposits	-	High strength; good load distri-bution to under-lying soft soils

Table 7-30 Range of Particle Sizes for Various Soil Improvement Methods



7.6.1.2.1. Soft Soil

Soft soils have poor volume stability and low strength and may be composed of loose sands and silts, wet clays, organic soils, or combinations of these materials. Most of the methods listed in Table 7-29 and Table 7-30 are used to minimize settlement in soft soil. Applicability of these methods depends on economy; effectiveness of treatment in the existing soil; availability of equipment, materials, and skills; and the effect on the environment such as disposal of waste materials. Some of the more useful methods for improving soft soil are described in more detail below.

7.6.1.2.1.1. Removal by excavation

Soft soil underlain by suitable bearing soil at shallow depths (less than 20 ft) may be economical to remove by excavation and replace with suitable borrow material or with the original soil after drying or other treatment. Compacted lean clays and sands (if necessary, with chemical admixtures such as lime, flyash and/or Portland cement) is an adequate replacement material if the water table is below the excavation line. Granular material such as sand, slag, and gravel should be used if the water table is above the bottom of the excavation. Additional mechanical compaction may be accomplished with vibratory or dynamic methods, Table 7-29.

7.6.1.2.1.2. Precompression

Precompression densifies the foundation soil by placing a load or surcharge fill, usually a weight that exceeds the permanent structure load, on the site. The preload should eliminate most of the postconstruction primary consolidation and some secondary compression and increase the soil strength.

a) For embankments, additional fill beyond that required to construct the embankment is usually placed.

b) For foundations other than earth structures, the preload must be removed prior to construction.

7.6.1.2.1.3. Vertical Drains

Time required for preload may sometimes be appreciably reduced by sand or prefabricated vertical (PV) strip drains to accelerate consolidation of thick layers of low permeability. PV drains commonly consist of a filter fabric sleeve or jacket made of nonwoven polyester or polypropylene surrounding a plastic core. The drain is inserted into the soil using an installation mast containing a hollow mandrel or lance through which the drain is threaded. An anchor plate is attached to the end of the drain. Theoretical estimates of the rate of settlement are largely qualitative unless prior experience is available from similar sites because the analysis is sensitive to soil input parameters, particularly the coefficient of consolidation and existence of pervious bands of soil. Strip drains have largely replaced sand drains in practice.

All drains should be connected at the ground surface to a drainage blanket. Vertical drains are utilized in connection with fills supporting pavements or low- to moderate-load structures and storage tanks. Common types of vertical drains are shown in Table 7-31. Sand drains driven with a closed-end pipe produce the largest displacement and disturbance in the surrounding soil and thus their effectiveness is reduced.

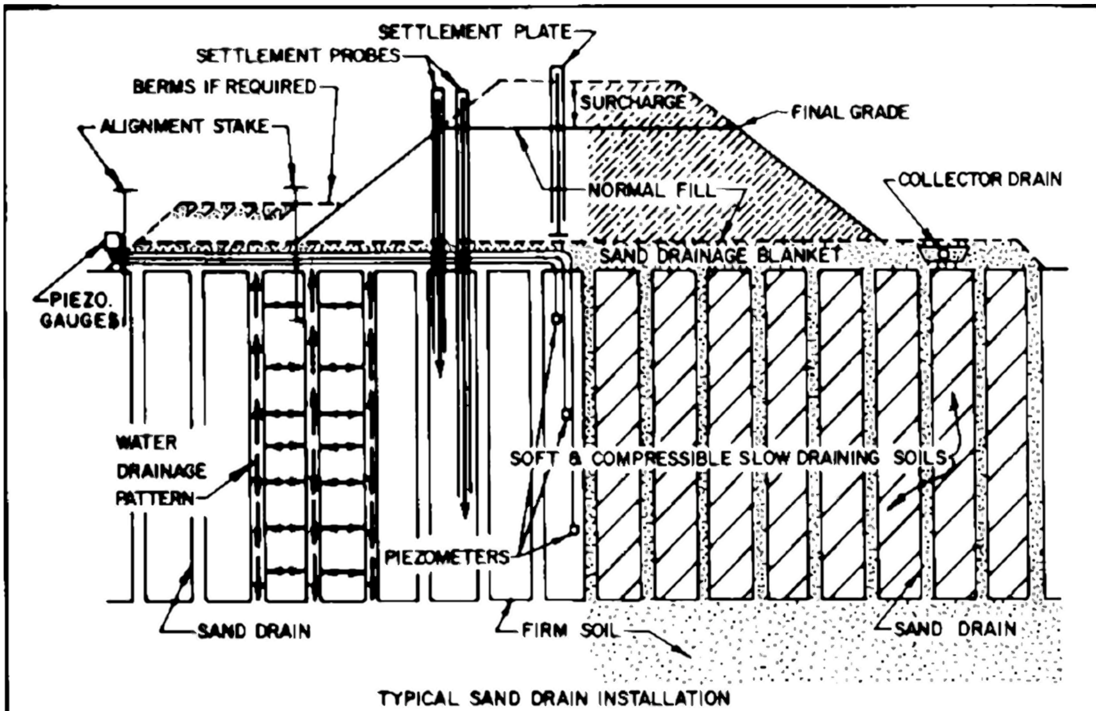
Table 7-31 Common Types of Vertical Drains[224]

General Type	Sub-type	Typical Installation	
		d_w	s
1. Driven Sand Drain	Closed end mandrel	18$\pm$ in	5 - 20 ft
2. Augered Sand Drain	(a) Screw type auger	6 - 30 in	-
	(b) Continuous flight hollow stem auger	18 in	5 - 20 ft
3. Jetted Sand Drain	(a) Internal jetting	18 in	5 - 20 ft
	(b) Rotary jet	12 - 18 in	5 - 20 ft
	(c) Dutch jet-bailer	12 in	4 - 16 ft
4. "Paper" Drain	(a) Kjellman cardboard wick	0.1$\pm$ in by 4$\pm$ in	1.5^+ - 4^+ ft
	(b) Cardboard coated plastic wick	slightly thicker	-
5. Fabric Encased Sand Drain	(a) Sandwick	2.5 - 3 in	4 - 12 ft
	(b) Fabridrain	5 in	-

[224] Ladd, C.C., Use of Precompression and Vertical Sand Drains for Stabilization of Foundation Soils, ASCE New York Metropolitan Section Seminar, 1978.

∞ Characteristics. Vertical drains accelerate consolidation by facilitating drainage of pore water but do not change total compression of the stratum subject to a specific load. Vertical drains are laid out in rows, staggered, or aligned to form patterns of equilateral triangles or squares. See Figure 7-38 for cross-section and design data for typical installation for sand drains.

Figure 7-38 Data for Typical Sand Drain Installation



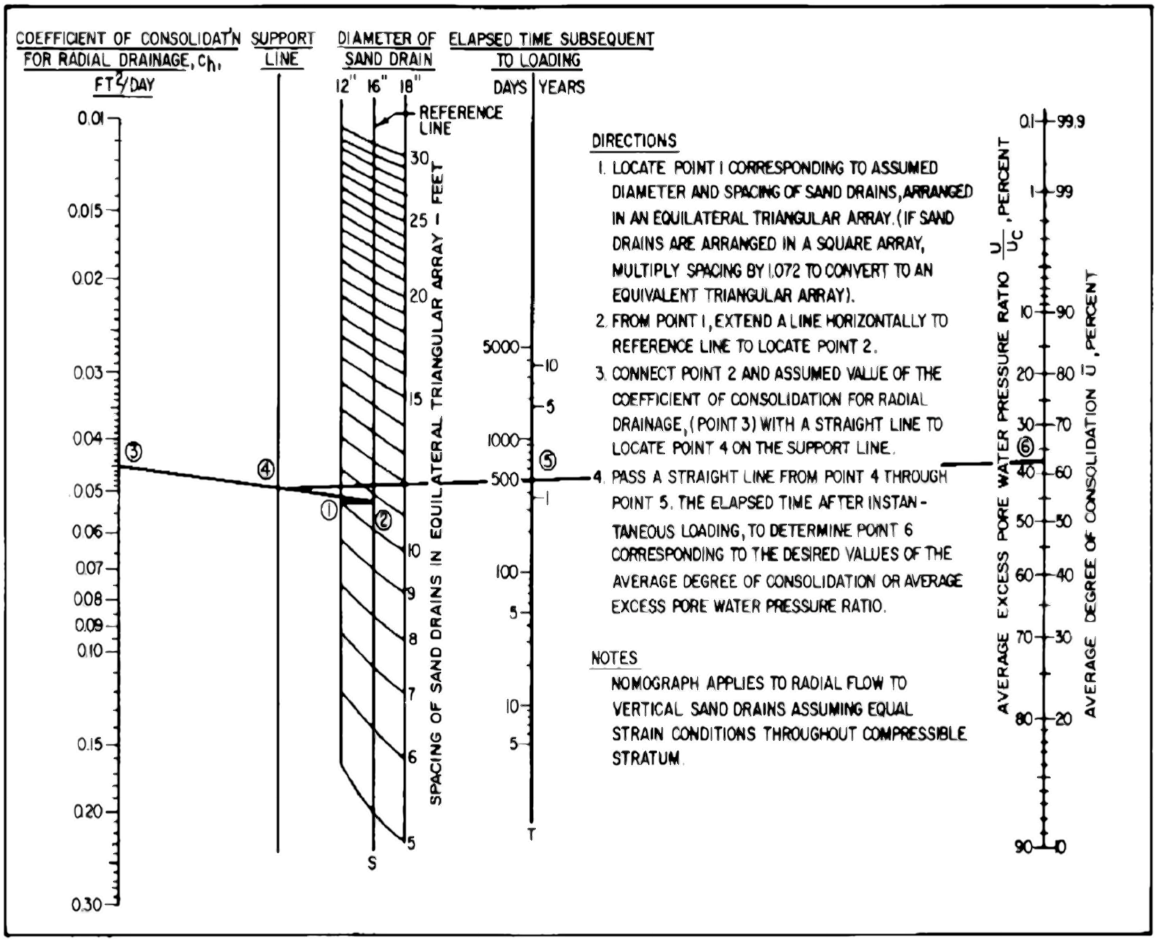
DIAMETER OF DRAINS RANGES FROM 6 TO 30 IN., GENERALLY BETWEEN 16 AND 20 IN.
SPACING OF DRAINS RANGES FROM 6 TO 20 FT ON CENTER, GENERALLY BETWEEN 6 AND 10 FT.
PRINCIPAL METHODS FOR INSTALLING DRAINS ARE CLOSED OR OPEN MANDRELS ADVANCED BY DRIVING OR JETTING, OR ROTARY DRILLING WITH OR WITHOUT JETS. DRIVING CLOSED MANDREL IS THE MOST COMMON.
DRAIN BACKFILL MATERIAL SHOULD HAVE SUFFICIENT PERMEABILITY TO DISCHARGE PORE WATER FLOW ANTICIPATED, BUT USUALLY DOES NOT MEET FILTER REQUIREMENTS AGAINST FOUNDATIONS SOILS. CLEAN SANDS WITH NO MORE THAN 3% BY WEIGHT PASSING No. 200 SIEVE IS USUALLY SUITABLE. A TYPICAL GRADATION IS AS FOLLOWS:

SIEVE No.	4	16	50	100	200
% FINER BY WEIGHT:	90-100	40-85	2-30	0-7	0-3

SAND DRAINAGE BLANKET MATERIAL IS SIMILAR TO THAT USED FOR DRAIN BACKFILL. IN SOME CASES GRAVEL WINDROWS OR PERFORATED, CORRUGATED, METAL PIPE ARE PLACED IN DRAINAGE BLANKET TO REDUCE HEAD LOSS IN DRAINAGE BLANKET. LONGITUDINAL DITCH OR COLLECTOR DRAIN MAY BE PLACED AT TOE. GRANULAR WORKING MAT IS SOMETIMES PLACED BELOW DRAINAGE BLANKET TO SUPPORT EQUIPMENT.
SURCHARGE LOAD IS PLACED TO REDUCE OR ELIMINATE POSTCONSTRUCTION CONSOLIDATION BENEATH NORMAL FILL. GENERALLY THE SURCHARGE LOAD IS NO MORE THAN ABOUT 30% OF NORMAL EMBANKMENT LOAD.
FIELD CONTROL DEVICES:
PIEZOMETERS OF STANDPIPE OR CLOSED SYSTEM TYPE TO OBSERVE PORE WATER PRESSURES;
SETTLEMENT PLATES, MINIMUM 3FT SQUARE, PLACED AT BASE OF FILL TO RECORD TOTAL SETTLEMENT;
SETTLEMENT PROBES DRIVEN OR AUGERED INTO FOUNDATION STRATUM TO MEASURE COMPRESSION WITHIN FOUNDATION.
ALIGNMENT STAKES, T-SHAPED STAKES PLACED AT OR OUTSIDE EMBANKMENT TOE TO OBSERVE LATERAL MOVEMENT AND HEAVE.

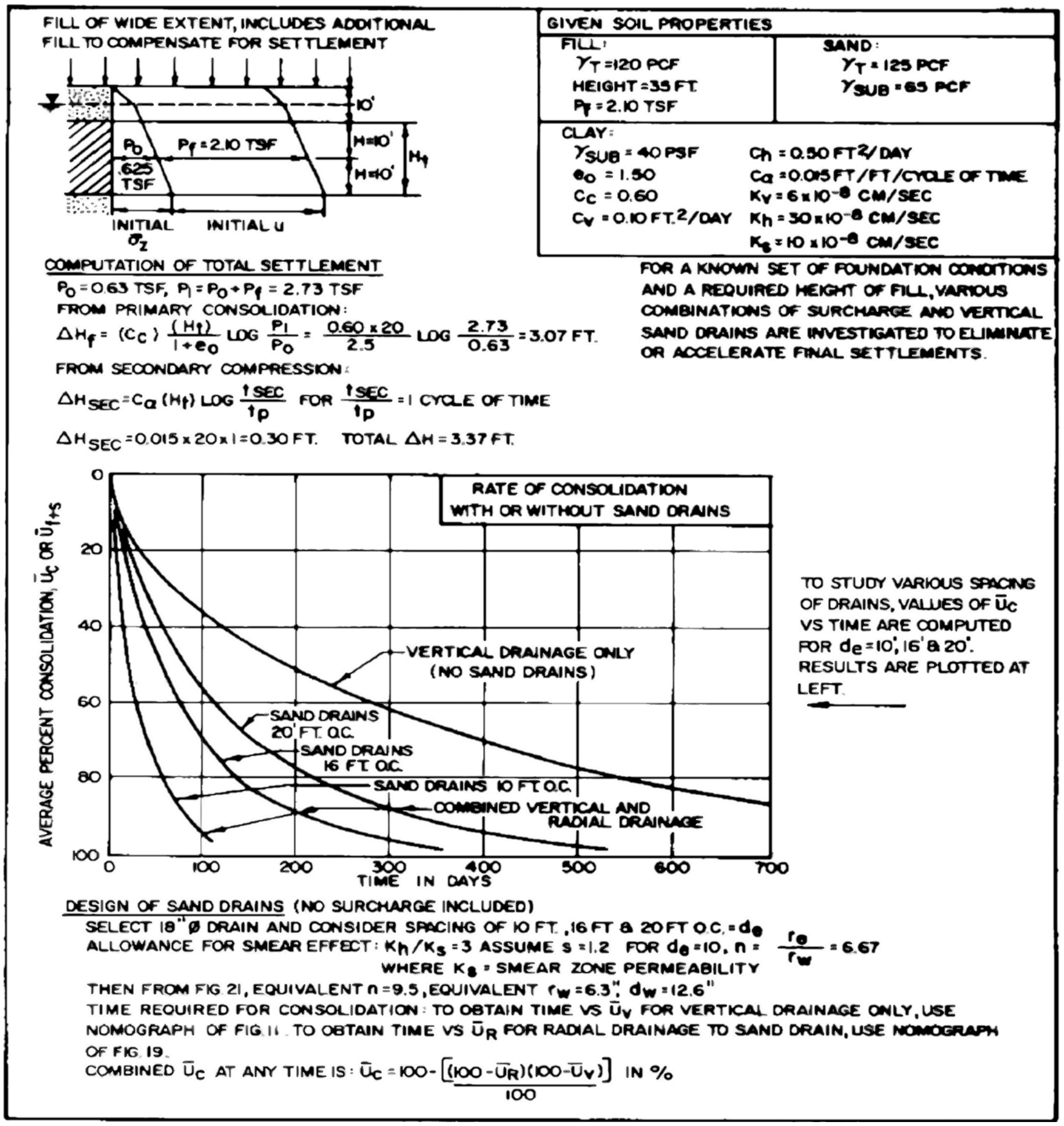
∞ Consolidation Rate. Time rate of consolidation by radial drainage of pore water to vertical drains is defined by time factor curves. For convenience, use the nomograph of Figure 7-39 to determine consolidation time rate. Determine the combined effect of vertical and radial drainage on consolidation time rate.

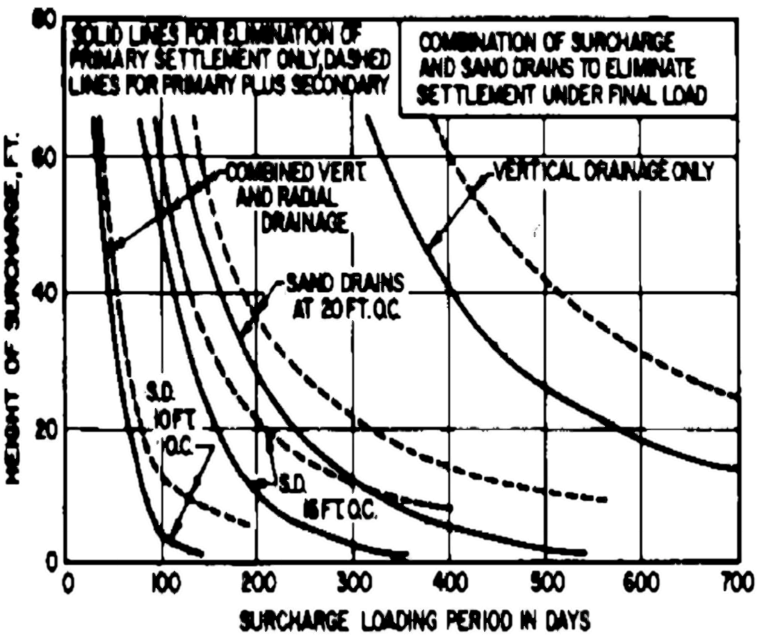
Figure 7-39 Nomograph for Consolidation with Radial Drainage to Vertical Sand Drain



∞ Vertical Drain Design. See Figure 7-40 for an example of design. For a trial selection of drain diameter and spacing, combine percent consolidation at a specific time from vertical drainage with percent consolidation for radial drainage to the drain. This combined percent consolidation U_c is plotted versus elapsed time for different drain spacing in the centre panel of Figure 7-40. Selection of drain spacing depends on the percent consolidation required prior to start of structure, the time available for consolidation, and economic considerations.

Figure 7-40 Example of Surcharge and Sand Drain Design





SELECTION OF SURCHARGE HEIGHT

$\Delta H_f = 3.07'$, $\Delta H_{SEC.} = 0.30'$, $P_f / P_0 = 3.38$

TO ELIMINATE SETTLEMENT UNDER P_f, $\bar{U}_c$ IS TAKEN EQUAL TO $\bar{U}_{f+s}$, $\bar{U}_{f+s} = \dfrac{\Delta H_f}{\Delta H_{f+s}}$ OR $\dfrac{\Delta H_f + \Delta H_{SEC}}{\Delta H_{f+s}}$

RELATION OF $\bar{U}_{f+s}$ AND TIME IS GIVEN ABOVE FOR VARIOUS DRAIN SPACINGS.

SURCHARGE P_s FOR VALUES OF $\bar{U}_{f+s} = \dfrac{\Delta H_f}{\Delta H_{f+s}}$ IS GIVEN IN FIG. 17

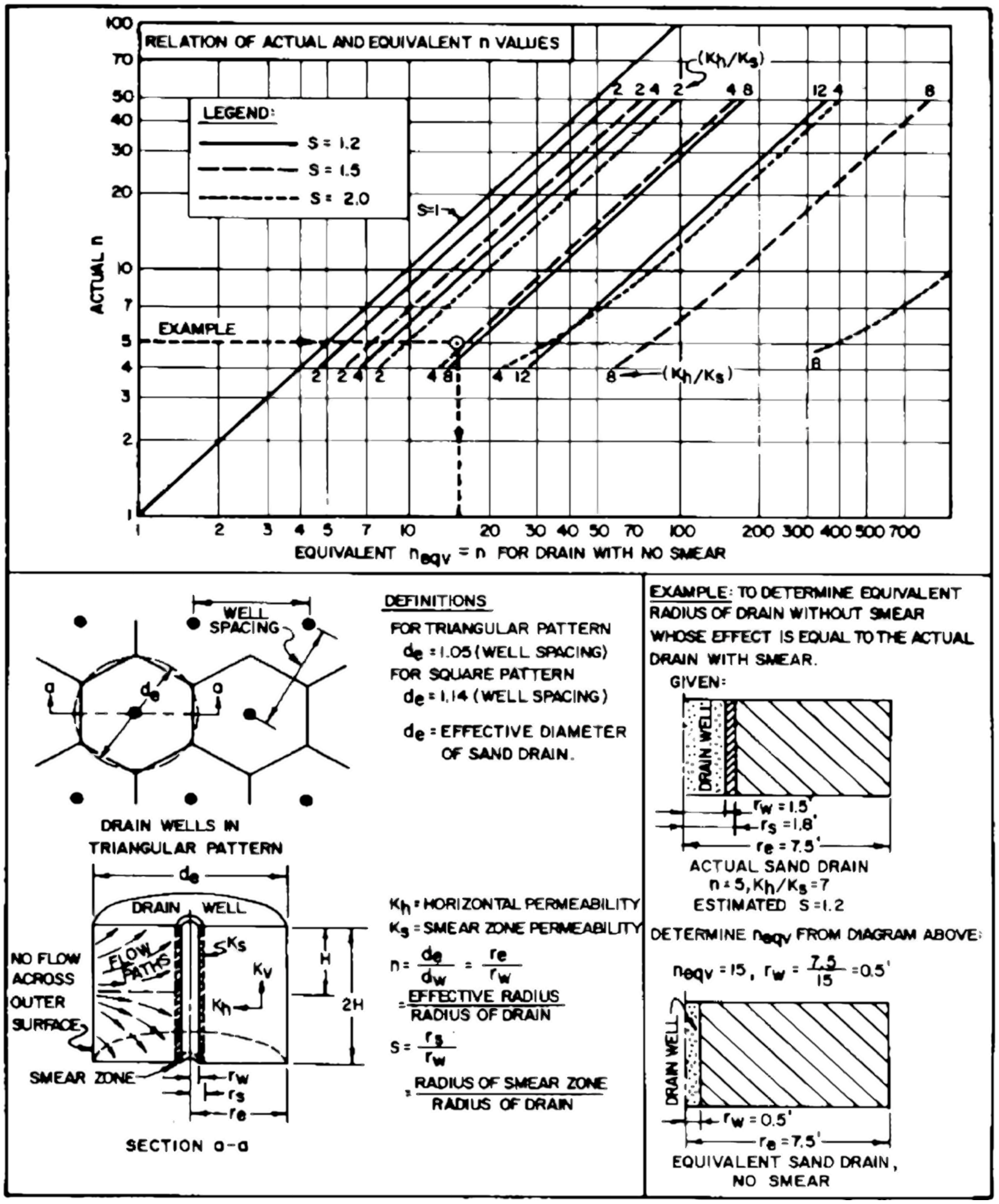
SURCHARGE P_s FOR VALUES OF $\bar{U}_{f+s} = \dfrac{\Delta H_f + \Delta H_{SEC}}{\Delta H_{f+s}}$ IS GIVEN BY FORMULA IN FIG. 17

USING THESE RELATIONSHIPS, P_s (EXPRESSED AS HEIGHT OF SURCHARGE) REPLACES $\bar{U}_{f+s}$ IN FIGURE 17.

COMBINATION OF SAND DRAIN AND SURCHARGE IS SELECTED BASED ON TIME AVAILABLE AND COMPARATIVE COSTS.

- Allowance for Smear and Disturbance. In cases where sand drain holes are driven with a closed-end pipe, soil in a surrounding annular space one-third to one-half the drain diameter in width is remolded and its stratification is distorted by smear. Smear tends to reduce the horizontal permeability coefficient, and a correction should be made in accordance with Figure 7-41.

Figure 7-41 Allowance for Shear Effect in Sand Drain Design



- Sand Drains Plus Surcharge. A surcharge load is normally placed above the final fill level to accelerate the required settlement. Surcharge is especially necessary when the compressible foundation contains material in which secondary compression predominates over primary consolidation. The percent consolidation under the surcharge fill necessary to eliminate a specific amount of settlement under final load is determined as shown in the lowest panel of Figure 7-40.
- General Design Requirements. Analyse stability against foundation failure, including the effect of pore pressures on the failure plane. Determine allowable build-up of pore pressure in the compressible stratum as height of fill is increased.
    - Horizontal Drainage. For major installation, investigate in detail the horizontal coefficient of consolidation by laboratory tests with drainage in the horizontal direction, or field permeability tests to determine horizontal permeability.

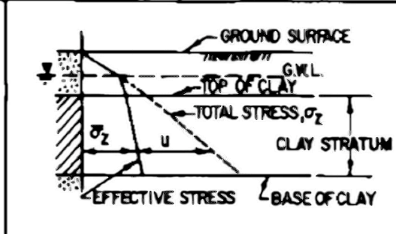
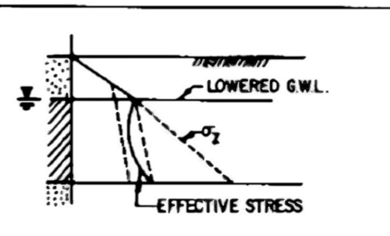
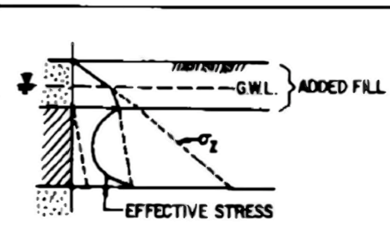
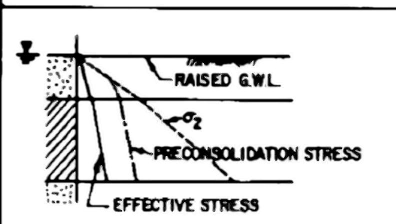
  - Consolidation Tests. Evaluate the importance of smear or disturbance by consolidation tests on remolded samples. For sensitive soils and highly stratified soils, consider nondisplacement methods for forming drain holes.
  - Drainage Material. Determine drainage material and arrangement to handle maximum flow of water squeezed from the compressible stratum.
- Construction Control Requirements. Control the rate of fill rise by installing piezometer and observing pore pressure increase for comparison with pore pressure values compatible with stability. Check anticipated rate of consolidation by pore pressure dissipation and settlement measurements.

7.6.1.2.1.4. Balancing Load By Excavation

To decrease final settlement, the foundation of heavy structures may be placed above compressible strata within an excavation that is carried to a depth at which the weight of overburden, removed partially or completely, balances the applied load.

- Computation of Total Settlement. In this case, settlement is derived largely from recompression. The amount of recompression is influenced by magnitude of heave and magnitude of swell in the unloading stage.
- Effect of Dewatering. If drawdown for dewatering extends well below the planned subgrade, heave and consequent recompression are decreased by the application of capillary stresses. If groundwater level is restored after construction, the load removed equals the depth of excavation times total unit weight of the soil. If groundwater pressures are to be permanently relieved, the load removed equals the total weight of soil above the original water table plus the submerged weight of soil below the original water table. Calculate effective stresses as described in Figure 7-42, and consolidation under structural loads as shown in Figure 7-43.

Figure 7-42 Profiles of Vertical Stresses before Construction

	STRESS CONDITION	DIAGRAM OF VERTICAL STRESSES	DESCRIPTION
	(1) SIMPLE OVERBURDEN PRESSURE		TOTAL STRESS σ_z IS COMPUTED USING TOTAL UNIT WEIGHT γ_T BOTH ABOVE AND BELOW THE G.W.L. PORE WATER PRESSURE U IS DUE TO G.W.L. EFFECTIVE STRESS $\bar{\sigma}_z = \sigma_z - U$.
HYDROSTATIC EXCESS PRESSURE	(2) LOWERING OF GROUND WATER LEVEL		IMMEDIATELY AFTER LOWERING OF THE GROUNDWATER TOTAL STRESS IN TOP SAND LAYER REMAINS PRACTICALLY UNCHANGED, BUT THE EFFECTIVE STRESSES INCREASE. SINCE THE WATER ESCAPES SLOWLY FROM THE CLAY LAYER, THE EFFECTIVE STRESS REQUIRES LONG TIME TO REACH THE NEW EQUILIBRIUM VALUE.
HYDROSTATIC EXCESS PRESSURE	(3) PARTIAL CONSOLIDATION UNDER WEIGHT OF INITIAL FILL		TOTAL STRESSES ON A CLAY LAYER INCREASED BY THE ADDITION OF SURCHARGE LOAD. INITIALLY THIS LOAD IS CARRIED BY PORE WATER IN THE FORM OF EXCESS PORE PRESSURE. AS THE SETTLEMENT PROGRESSES IN THE CLAY LAYER, THE EFFECTIVE STRESS INCREASES TO CORRESPOND TO THE STRESS FROM SURCHARGE LOAD.
PRECONSOLIDATED CONDITIONS	(4) RISE OF GROUND WATER LEVEL		RISE OF GROUND WATER LEVEL DECREASES EFFECTIVE PRESSURE OF OVERBURDEN. EFFECTIVE STRESS LINE MOVES TO LEFT. THEN PRECONSOLIDATION STRESS EQUALS ORIGINAL EFFECTIVE STRESS OVERBURDEN. TOTAL STRESS PRACTICALLY UNCHANGED.

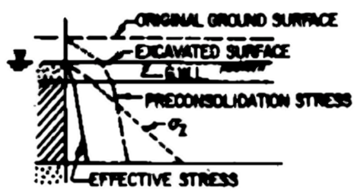
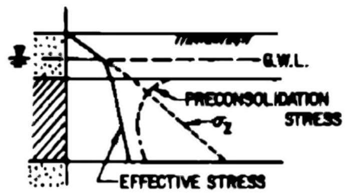
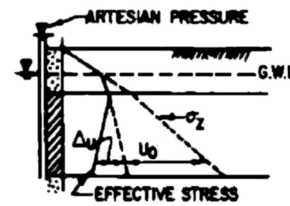
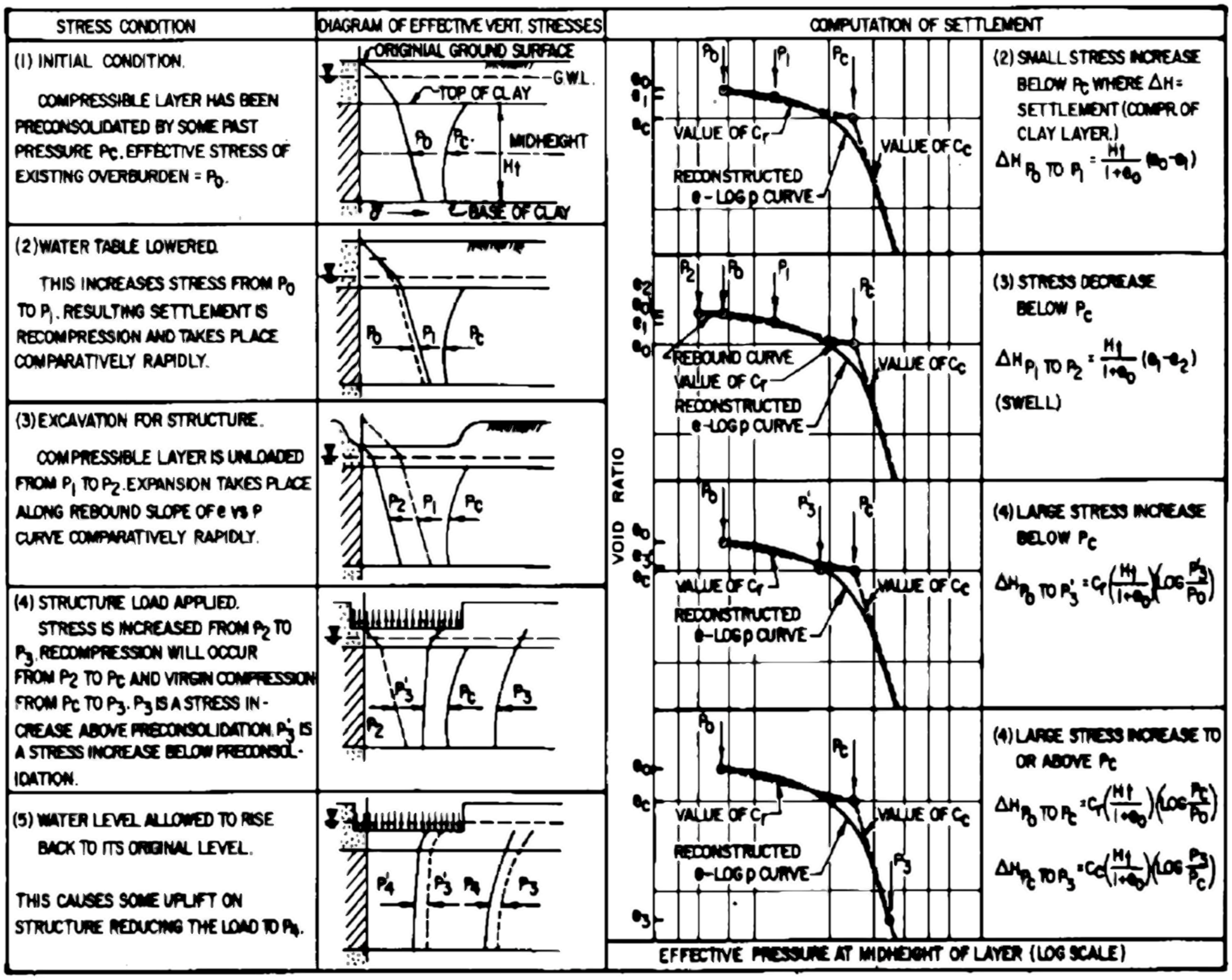
	STRESS CONDITION	DIAGRAM OF VERTICAL STRESSES	DESCRIPTION
PRECONSOLIDATED CONDITIONS	(5) EXCAVATION		EXCAVATION OF OVERBURDEN MATERIAL UNLOADS CLAY LAYER. EFFECTIVE STRESS LINE MOVES TO THE LEFT. THEN PRECONSOLIDATION STRESS EQUALS ORIGINAL EFFECTIVE STRESS OF OVERBURDEN.
PRECONSOLIDATED CONDITIONS	(6) PRECONSOLIDATION FROM LOADING IN THE PAST		PRECONSOLIDATION FROM PAST LOADINGS GREATER THAN THE EXISTING OVERBURDEN MAY HAVE BEEN CAUSED BY WEIGHT OF GLACIAL ICE, EROSION OF FORMER OVERBURDEN, LOWER GROUND WATER LEVEL PLUS DESSICATION, OR REMOVAL OF FORMER STRUCTURES.
(7) ARTESIAN PRESSURE			SAND STRATUM BELOW THE CLAY MAY BE SUBJECT TO ARTESIAN HYDRAULIC PRESSURES THAT DECREASE EFFECTIVE STRESS AT BASE OF CLAY. TOTAL STRESS REMAINS UNCHANGED.

Figure 7-43 Computation of Total Settlement for Various Loading Conditions

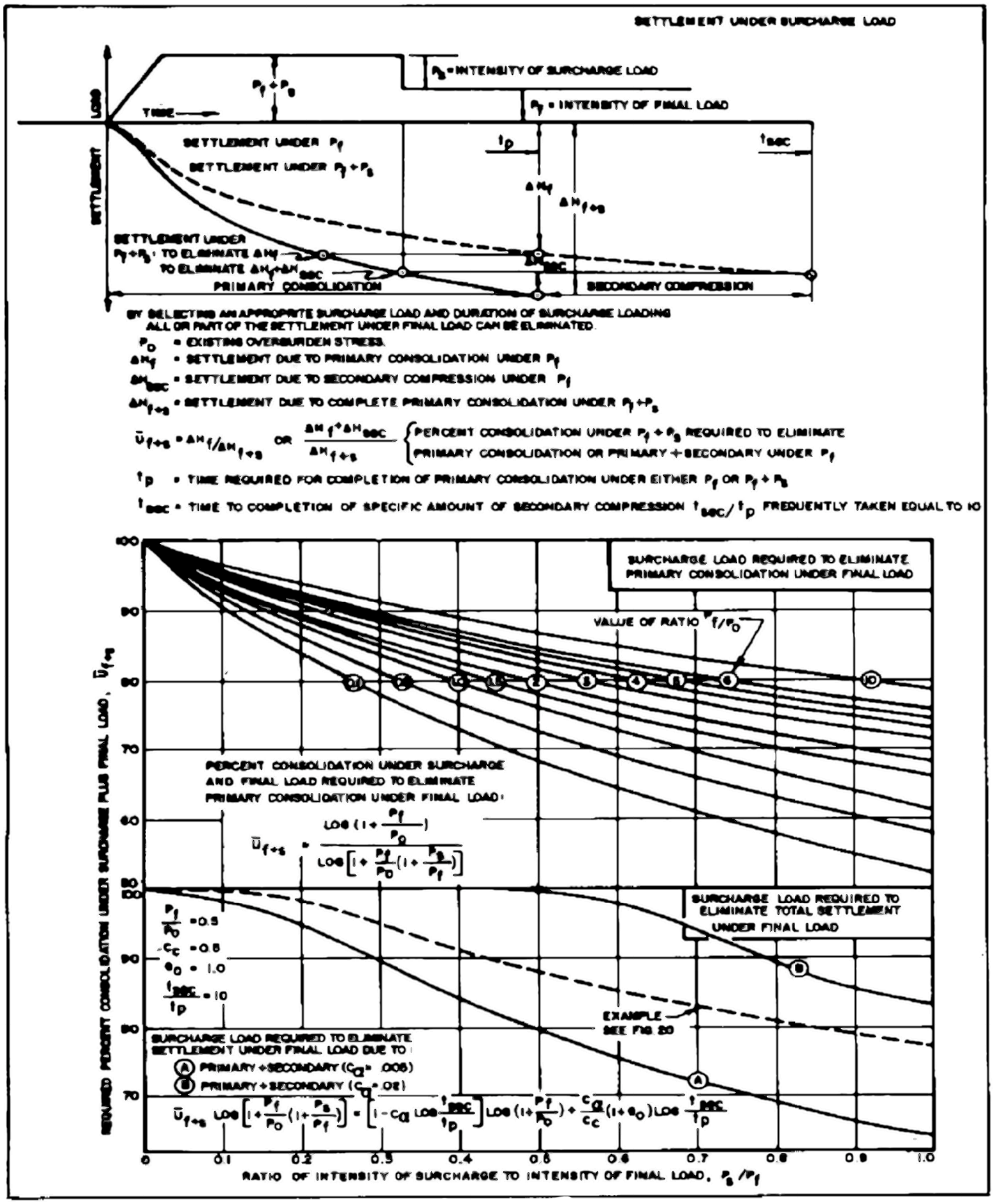


7.6.1.2.1.5. Preconsolidation by Surcharge

This procedure causes a portion of the total settlement to occur before construction. It is used primarily for fill beneath paved areas or structures with comparatively light column loads. For heavier structures, a compacted fill of high rigidity may be required to reduce stresses in compressible foundation soil.

- Elimination of Primary Consolidation. Use Figure 7-44 to determine surcharge load and percent consolidation under surcharge necessary to eliminate primary consolidation under final load. This computation assumes that the rate of consolidation under the surcharge is equal to that under final load.
- Elimination of Secondary Consolidation. Use the formula in the bottom panel of Figure 7-44 to determine surcharge load and percent consolidation under surcharge required to eliminate primary consolidation plus a specific secondary compression under final load.
- Limitations on Surcharge. In addition to considerations of time available and cost, the surcharge load may induce shear failure of the soft foundation soil. Analyse stability under surcharge.

Figure 7-44 Surcharge Load Required to Eliminate Settlement under Final Load



7.6.1.2.1.6. Stone or chemically stabilized soil columns

Columns made of stone or chemically stabilized soil increase the stiffness of the foundation and can substantially decrease settlement. Columns may fail by bulging if the adjacent soil gives inadequate support or fail by shear as a pile because of insufficient skin friction and end bearing resistance.

a) Stone columns are made by vibroreplacement (wet) or vibrodisplacement (dry) methods, Table 7-29. Diameters range from 1.5 to 4 ft with spacings from 5 to 12 ft. A blanket of sand and gravel or a semirigid mat of reinforced earth is usually placed over stone column reinforced soil to improve load transfer to the columns by arching over the *in situ* soil. Stone columns are not recommended for soils with sensitivities greater than 5.

b) Lime columns are made by mixing metered or known amounts of quicklime using drilling rigs to achieve concentrations of 5 to 10% lime by weight of dry soil. Structures are constructed on thin concrete slabs where settlement is assumed uniform over the entire area.

c) Cement columns are made by adding 10 to 20% cement as a slurry. These columns are brittle, have low permeability, and have been used below sea level.

7.6.1.2.1.7. Jet grouting

Jet grouting is the controlled injection of cement grouts to replace most any type of soil; water jets while grouting erode this soil. The most common application has been underpinning of existing structures to reduce total and differential settlement and as cut-off walls for tunnels, open cuts, canals, and dams. Jet grouting may also be used to consolidate soft foundation soils for new structures, embankments, and retaining walls. Other applications include support of excavations for open cuts and shafts and slope stabilization.

a) Jet grouting can break up the soil and mix grout with the natural soil particles or break up the soil, partially remove the soil, and mix grout with the remaining soil particles.

b) Jet grouting can substantially increase the strength and stiffness of soft clay soil to reduce settlement and substantially reduce the permeability of sandy soil.

c) Jet grouting is generally used with rapid set cement and with fly ash. Fly ash when mixed with cement or lime produces a cementatious material with excellent structural properties. Other chemicals may be used instead of cement.

d) A single jet nozzle can be used to both break down the soil structure and force mixing of grout with the natural soil. A water jet can also be sheathed in a stream of compressed air to erode the soil while a grout jet beneath the water jet replaces the broken or disturbed soil. Diameter and discharge pressure of the nozzles, withdrawal and rotation rates, type and quality of grout, and soil type influence volume and quality of the grouted mass. Withdrawal rates and nozzle pressures are the primary design factors. Withdrawal rates vary from 1 to 50 inches/minute and nozzle pressures often range from 3000 to 9000 psi depending on the type of soil.

7.6.1.2.1.8. Removal by displacement

Sufficient cohesionless fill is placed to cause bearing failures in the underlying soft soil. The soft soil is displaced in the direction of least resistance, which is usually ahead of the embankment fill. The displaced soil causes a mudwave that should be excavated at the same rate that the embankment is placed to minimize trapping pockets of soft soil beneath the embankment.

7.6.1.2.1.9. Lightweight fills

Sawdust, expanded foam plastic blocks, expanded shale or clay, oyster shell, and fly ash fills can partially replace excavated heavier soft material and reduce the net increase in pressure on underlying soft soil. The availability of lightweight fill in sufficient quantity at reasonable cost and suitable locations to dispose of the excavated soft soil limit application of this method.

7.6.1.2.1.10.Structural (Self-supporting fills)

Some naturally occurring materials such as dead oyster shell can form a barge-like structure from particle interlocking. Fills of loose shell have been used for highway embankments and foundations for flexible facilities such as warehouses on marsh and swamp deposits.

7.6.1.2.1.11.Blasting

Cohesionless, saturated sands (less than 25% passing the 200 mesh) are most responsive to densification by the detonation of dynamite charges in loose deposits. Soft soils that can be liquefied or displaced by advancing fill can be removed by blasting for embankment construction. Blasting or toe shooting in front of the embankment may displace soft soils. The extent of soil improvement by blasting is often uncertain.

a) The underfill method, where backfill is placed on top of soft soil and explosives are placed under the embankment by lowering down casing into the soft deposits, is most effective when the embankment width is less than 60 ft.

b) The ditching method, where fill is placed immediately into excavations made by blasting, is effective for depths of soft soil less than 15 ft.

c) The relief method may be useful where ditches are blasted along each side of the embankment to provide lateral stress relief and force soft shallow soil to move laterally into the ditches.

7.6.1.2.2. Expansive Soil

Potentially expansive soils are usually desiccated and will absorb available moisture. These soils can be made to maintain volume changes within acceptable limits by controlling the soil water content and by reducing the potential of the soil to heave. Methods for improving the performance of foundations in expansive soil are illustrated in Table 7-32.

Table 7-32 Improving Performance in Expansive Soil

Method	Description
Removal by excavation and replacement with non-expansive fill	Removal of surface expansive soil to depths of from 4 to 8 ft and replacement with compacted nonexpansive fill usually eliminates most potential soil heave because the depth of moisture change is often limited to about 8 ft.
Placement of vertical moisture barriers	Vertical moisture barriers placed adjacent to pavements or around the perimeter of foundations down to the maximum depth of moisture changes is effective in maintaining uniform soil moisture within the barrier. Differential movements are minimized. Long-term soil wetting with uniform heave beneath impervious foundations may occur from lack of natural evapotranspiration
Lime stabilization	Lime injected or mixed into expansive soil can reduce potential for heave by reducing the mass permeability thereby reducing amount of water seeping into the soil, by cementation, and by exchange of sodium for calcium ions. Fissures should exist in situ to promote penetration of lime injected slurry. Lime may be detrimental in soils containing sulfates.
Potassium injection	Potassium solutions injected into expansive soil can cause a base exchange, increase the soil permeability and effectively reduce the potential for swell.
Prewetting	Free water is added by ponding to bring soil to the estimated final water content prior to construction. Vertical sand drains may promote wetting of subsurface soil.
Surcharge	Placing 1 or 2 ft or more of permanent compacted fill on the surface of a level site prior to construction increases the overburden pressure on the underlying soil reducing the negative (suction) pore water pressure; therefore, the potential for swell is less and tends to be more uniform. This fill also increases elevation of the site providing positive drainage of water away from the structure.

7.6.1.2.3. Collapsible Soil

Collapsible soils settle when wetted or vibrated; therefore, the usual approach toward optimizing performance of structures on collapsible soil is prewetting the construction site. Hydrocompaction (see Table 7-29) of the site prior to construction is commonly recommended. Chemical stabilization with lime, sodium silicate, or other chemicals is not always successful. Methods applicable to improving performance of structures on collapsible soil are illustrated in Table 7-33.

Table 7-33 Improving Performance of Collapsible Soil

Depth of Soil Treatment, ft	Description
0 to 5	Wetting, mixing, and compaction
> 5	Overexcavation and recompaction with or without chemical additives such as lime or cement
	Hydrocompaction
	Vibroflotation
	Lime pressure injection
	Sodium silicate injection
	Prewetting by ponding; vertical sand drains promote wetting of subsurface soil

7.6.1.3. Foundation Techniques

Foundation design and construction methods can minimize soil volume changes and differential movement.

7.6.1.3.1. Floating Foundations

Foundation elements such as mats and footings can be placed in excavations of sufficient depth where the pressure applied by the structure to the underlying foundation soil approximately balances pressure applied by the excavated soil. Observed deformation will be elastic recompression settlement. The exposed soil in the bottom of the excavation must be protected from disturbance and deterioration.

7.6.1.3.2. Ribbed Mats

Slab foundations supported by a grid of stiffening beams can transfer structural loads to soil of adequate stiffness and bearing capacity. The stiffness of ribbed mats also reduces differential movement in expansive soil. The depth of stiffening beams normally does not exceed 3 ft. Ribbed mats supported on compacted cohesive nonexpansive fills are commonly constructed in expansive soil areas.

7.6.1.3.3. Leveling Jacks

Jacks on isolated footings in which the elevation can be periodically adjusted to reduce distress from excessive differential movement may support structures. Proper adjustment of leveling jacks requires periodic level surveys to determine the amount and direction of adjustment, whether up or down, and frequency of adjustment to minimize differential movement. Leveling jacks are usually inconvenient to owner/operators of the structure.

7.6.1.3.4. Deep Foundations

Structural loads can be transferred to deep, firm bearing strata by piles or drilled shafts to eliminate or minimize effects of shallow soil movements on structural performance. Uplift thrust from skin friction on the perimeter of deep foundation piles or drilled shafts in expansive soil or downdrag in consolidating or collapsing soil should be considered in the design.

7.6.1.3.5. Construction Aids for Excavations

Settlement or loss of ground adjacent to excavations may become excessive. Causes of loss of ground include lateral rebound of perimeter walls into the excavation, rebound at the bottom of the excavation, and dewatering. Damage may occur in adjacent structures including pavements and utilities if loss of ground exceeds 0.5 inch or lateral movement of perimeter walls into an excavation exceeds 2 inches. Level readings should be taken periodically to monitor elevation changes so that steps may be taken to avoid any damage. Construction aids include placement of bracing or retaining walls, placement of foundation loads as quickly as possible after the excavation is made, avoidance of ponding of water within excavations, and ground freezing. Load bearing soils at the bottom of the excavation must be protected from deterioration and water content changes following exposure to the environment. Ground freezing provides temporary support and groundwater control in difficult soils and it is adaptable to most size, shape, or depth of excavations. Ground freezing is accomplished by circulating a coolant, usually calcium chloride brine, through refrigeration pipes embedded in the soil.

7.6.1.4. Flexible Techniques

Structures may be made flexible to tolerate differential movement by placing construction joints in the superstructure or by using flexible construction materials. Steel or wood frames, metal siding, wood panelling, and asphalt floors can tolerate large differential settlements or angular distortions up to about 1/150.

7.6.2. Remedial Methods

7.6.2.1. General

Remedial work for damaged structures is often aggravated because it is difficult to determine the cause of the problem (e.g., location of source or loss of soil moisture with swelling or settling of expansive/collapsible soil may not be readily apparent). Investigation and repair are specialized procedures that usually require much expertise and experience. Cost of repair work can easily exceed the original cost of the foundation. Repair of structures in heaving soil is usually much more costly than in settling soil. Structures are less able to tolerate the tensile strains from heaving soil than the compressive strains in settling soil. The amount of damage that requires repair also depends on the attitudes of the owner and effected people to tolerate distortion and consequences if the distortion and damage are ignored. Only one remedial procedure should be attempted at a time after a course of action has been decided so its effect on the structure may be determined. Several common remedial methods are discussed below.

7.6.2.2. Underpinning with Piles

Underpinning may be accomplished by a variety of methods: drilled-in-place tangent piles, cast-in-place rigid concrete slurry walls, precast concrete retaining walls, root or pin piles, concrete underpinning pits, and jacked steel piles. Selection of the underpinning method depends on the nature of the subgrade soil and its expected behaviour during underpinning.

7.6.2.2.1. Avoid Ground Loss

Possibility of ground loss during installation may eliminate use of tangent piles, slurry walls, and precast concrete retaining walls.

7.6.2.2.2. Interference with Utilities

Underground utilities may eliminate use of piles or cast-in-place concrete shafts.

7.6.2.3. Grouting

Injecting Portland cement, fine soil, and chemicals into the problem soil may stabilize structures. Grouting mixtures usually consist of fine soil, Portland cement, and water; lime and water; sodium silicate; calcium chloride; polymers; and resins. Jet and compaction grouting, for example, reduce differential settlement of structures. Compaction grouting can raise a structure that has settled. Injecting a grout containing additives such as Portland cement to improve the performance of the soil may increase the stiffness and strength of the soil. Compaction grouting may use 12 to 15% by weight of Portland cement mixed with soil and water to make a viscous, low slump grout that is to be pumped into bored holes at pressures up to 500 psi.

7.6.2.4. Slabjacking

Slabjacking, the lifting or levelling of distorted foundations, is usually faster than other solutions for remedial work. Grouting materials include Portland cement, hydrated lime, fly ash, asphalt bitumen, drilling mud, casting plaster, and limestone dust. Consistency of the grout varies from less commonly used thin fluids to more common heavy pourable or stiff mortar-like mixtures (with nearly zero slump). Cement contents vary from 3 to 33% with sand or soil materials all passing the No. 16 sieve. Leakage from joints and along the edges of slabs can present serious problems, which are commonly offset by increasing the consistency of the grout. Lifts of as much as 1 ft are common. Properly performed slabjacking will not usually cause new fractures in the foundation, but existing cracks tend to open. Experience is required to cause low points to rise while maintaining high points at a constant elevation.

7.6.3. Dealing with Approach Embankment Settlement

7.6.3.1. Elimination of settlement with the approach embankment

A well-constructed soil embankment, using quality control with regard to material and compaction, will not consolidate. Standard specifications and construction drawings should be prepared for the approach embankment area (normally designated as 50 feet behind the wing wall). The structural designer should have the responsibility for selecting the appropriate approach embankment cross section depending on selection of structure foundation type.

Special attention must be given to the interface area between the structure and the approach embankment, as this is where the famous “bump at the end of the bridge” occurs. The reason for the bump is twofold; poor compaction of embankment material near the structure and migration of fine soil into drainage material. Poor densification is caused by restricted access of standard compaction equipment. Proper densification can be achieved by optimizing the soil gradation in this area to permit maximum density with minimum effort.

7.6.3.1.1. General Consideration for Select Structure Backfill

Select structure backfill is usually placed in relatively small quantities and in relatively confined areas. Structure backfill specifications must be designed to insure construction of a durable, dense backfill. The following considerations must be addressed:

Consideration	Reason For
Lift Thickness	6” to 8” , so compaction possible with small equipment
Topsize	Less than 3/4 of lift thickness
Gradation	Well graded for ease of compaction
Durability	Minimize breakdown of particles and settlement
Percent Fines	Minimize to prevent piping and allow rapid drainage
T99 Density Control	Small equipment cannot achieve T180 densities
Compatibility	Particles should not move into voids of adjacent fill or drain material

7.6.3.2. Practical Aspects of Embankment Settlement

Few engineers realize the influence of embankment placement on the subsoils. The total weight of an embankment has an impact on the type of foundation treatment that may be selected. For instance, a relatively low height embankment of 10’ may be effectively surcharged because the additional surcharge weight could be 30 to 40 percent of the proposed embankment. However, when the embankment height exceeds 50’ the influence of a 5 or 10 foot trapezoid of soil on top of this heavy 50 foot mass is small and probably not cost-effective. Conversely, as the embankment height (and, therefore, weight) increase, the use of a spread footing abutment becomes more attractive. A 30’ high, 50’ long approach embankment weighs about 15,000 tons compared to the insignificant weight of a total abutment loading which may equal 1,000 tons. Besides weight, the width of an embankment has an effect on total settlement. Wider embankments cause a pressure increase deeper into the subsoil. As might be expected, wide embankments will cause more settlement and take longer for consolidation to occur.

In addition, the use of geotextiles or geocomposite drains can be an effective method of preventing the bump at the end of the bridge. It is suspected that high dynamic loads are induced in abutment backfill. Inadequate filter layers or non-durable drain aggregate can cause either piping of fines or accelerated pavement subsidence due to breakdown of aggregates. In geographic areas where select materials are not available, the use of geosynthetic reinforcement of the abutment backfill and approach area can reduce the bump at the end of the bridge.

Recent developments in microcomputer software now permit simple computer analysis of approach embankment settlement. These programs permit the user to quickly compute settlements along abutments, piers buried in end slopes or pipes placed diagonally under approach fills.

7.7. Consolidation Tests

7.7.1. Introduction

There are four basic tests for consolidation parameters of compressible soils:

The following laboratory tests are described which may be used to evaluate consolidation parameters of compressible soils.

1. Step Load (SL) test. This is the most satisfactory test; the complete description of this is given below.
2. Rapid Load (RL) test.
3. Constant Rate of Deformation (CRD) test.
4. Controlled Gradient (CG) test.

The last three tests were developed to reduce the time required to complete the test relative to the SL test.

7.7.2. Apparatus

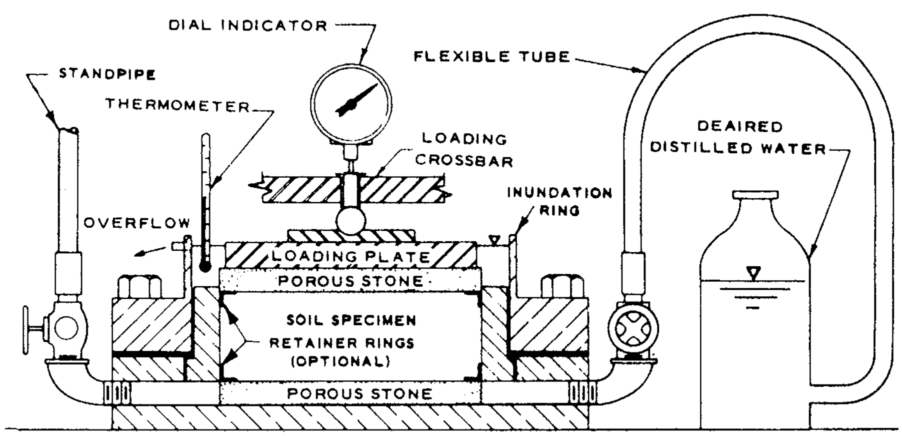
The apparatus should consist of the following:

a. Consolidometer shall consist of a rigid base, a consolidation ring, porous stones, a rigid loading plate, and a support for a dial indicator (Figure 7-45). It may be either the fixed-ring or the floating -ring type both of which are shown in Figure 7-46. The various metal parts of the consolidometer shall be of the same noncorrosive material. All-plastic or combination plastic and metal consolidometers may also be used to reduce electrochemical effects. The consolidometer shall conform to the following requirements:

   (1) Fixed-ring consolidometer shall have a rigid base with a recess for supporting the bottom porous stone and for seating and attaching the consolidation ring. The upper surface of the recess shall be grooved to permit drainage. The base shall also have (a) an inundation ring to permit submergence of the specimen in water to prevent evaporation of water from the specimen during the test, and (b) suitable connections and a standpipe for making permeability tests.

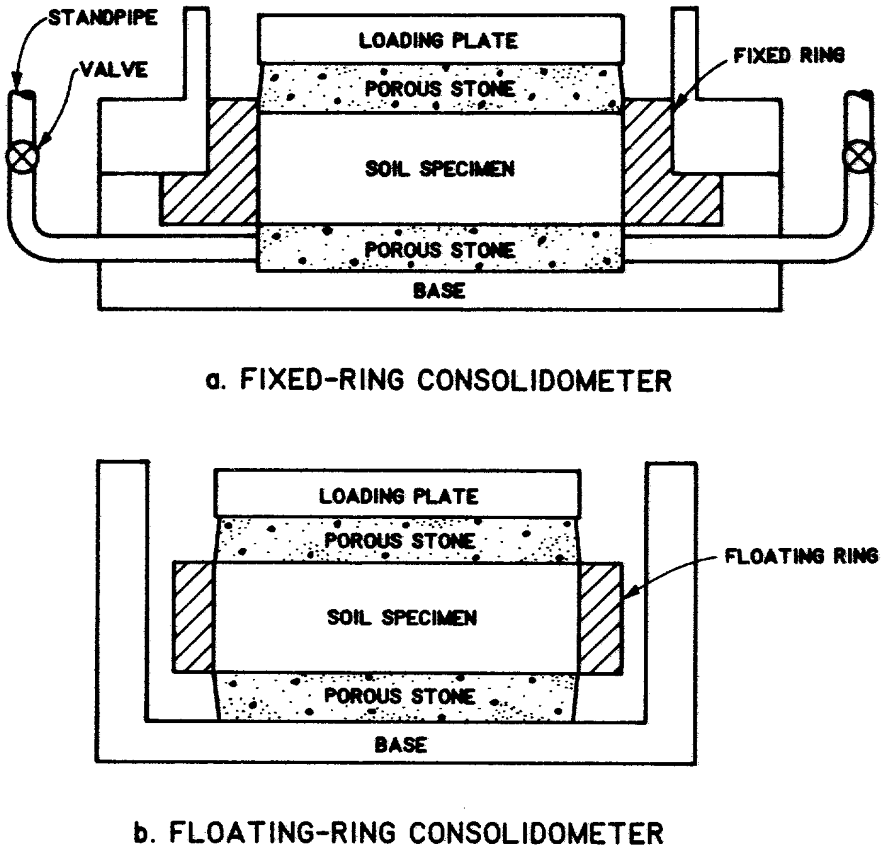
   (2) Floating-ring consolidometer[225] shall have a rigid base for supporting the bottom porous stone. The base shall be large enough to permit free vertical movement of the consolidation ring and shall have a chamber surrounding the ring for submergence of the specimen.

Figure 7-45 Typical Consolidometer



[225] In the floating-ring consolidometer, the friction between the inside of the ring and the specimen is less than that in the fixed-ring type. However, when very soft soils are tested with the floating-ring consolidometer, the side friction will not support the weight of the ring, and compression occurs toward the middle of the specimen from top and bottom. The floating-ring device is suitable only for comparatively stiff soils, and has the disadvantage that it cannot be used for permeability tests.

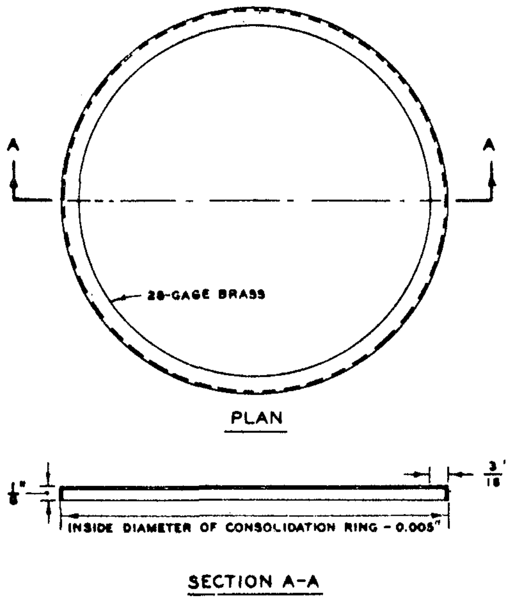
Figure 7-46 Schematic diagrams of fixed-ring and floating-ring consolidometers



b. Consolidation ring shall completely and rigidly confine and support the specimen laterally. The inside diameter of the ring should not be less than 2 3/4" and preferably not less than 4"; use of larger rings for specimens of larger diameter, particularly with the fixed-ring consolidometer, will reduce the percentage of applied load carried by side friction and consequently will provide more accurate results. Normally, the ratio of the height of ring to inside diameter of ring should be between 1:4 and 1:6. The consolidation ring may be lined with a material such as Teflon to reduce the friction between the ring and a specimen of fine-grained soil. A stainless steel ring is preferable for specimens containing abrasive particles.

c. Porous stones more pervious than the specimen of soil should be used to permit effective drainage. For routine testing, stones of medium porosity are satisfactory. The diameter of the porous stones shall be such as to prevent the squeezing out of soil through the clearance spaces between the ring and stone and to permit free compression of the specimen without binding. To minimize the possibility of binding, the sides of the upper porous stone of the fixed-ring consolidometer should be slightly tapered away from the specimen, while both porous stones of the floating-ring consolidometer should be tapered. A clearance of about 0.010" to 0.015" around the stone generally will be adequate; however, if very soft soils are tested, a smaller clearance may be desirable or retainer rings may be used as shown in Figure 7-45. Details of a typical retainer ring are shown in Figure 7-47. The porous stones should be cleaned after every test, preferably in an ultrasonic cleaner or by boiling and flushing.

Figure 7-47 Typical Retainer Ring



d. Loading devices of various types may be used to apply load to the specimen. The most commonly used is the beam-and-weight mechanism as shown in Figure 7-48. The loading device should be capable of transmitting axial load to the specimen quickly and gently. In addition, the equipment should be capable of maintaining the load constant for at least 24 hours. The equipment should be calibrated to ensure that the loads indicated are those actually applied to the soil specimen.

Figure 7-48 Beam and Weight mechanism

e. Dial Indicator. A dial indicator that reads counter clockwise, with a range of 0.50” and graduated to 0.0001”, is recommended.

f. Equipment for Preparing Specimens. A trimming turntable operated as a vertical lathe is commonly used in preparing specimens (see Figure 7-49). Suitable trimming knives notched to fit the thickness of the consolidation ring, a wire saw with 0.01” diameter wire, and a metal straightedge or screed are also required.

g. Other items needed are:

   (1) Balances, sensitive to 0.1 g and 0.01 g.

   (2) Timing device, a watch or clock with second hand

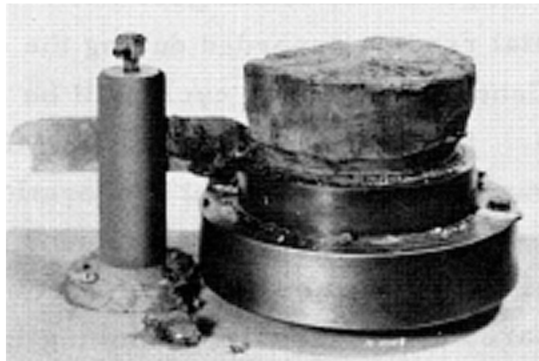
   (3) Centigrade thermometer, range 0 to 50º C, accurate to 0.1º C.

   (4) Distilled or demineralised water

   (5) Filter papers and glass plates

(6) Apparatus necessary to determine water content and specific gravity (see 3.4.1.1 and 3.4.3.1.1.

Figure 7-49 Cutting Specimen into Specimen Ring



7.7.3. Calibration of Equipment

In the consolidation test, it is desired to measure only the volume change of the specimen; therefore, corrections must be applied for any significant deformation due to the compressibility of the apparatus itself. In sandy and stiff soils, an appreciable proportion of the total deformation may be caused by this factor. Therefore, a calibration curve should be prepared for each consolidometer when testing such soils. This is done by placing the consolidometer with submerged porous stones and filter papers in the loading device, applying the load increments to be used in the consolidation test, and reading the dial indicator for each load. After the maximum load has been applied, the loads are decreased in the same order as that in which they were applied, and the dial indicator reading is again recorded. Since the deformations are almost instantaneous, the effect of time can be ignored. The total change in dial reading for each load is the correction to be applied to the dial reading recorded during the consolidation test under that same load. Generally, a single cycle will be sufficient for the calibration.

7.7.4. Preparation of Specimens

Specimens shall be prepared in a humid room to prevent evaporation of soil moisture. Extreme care shall be taken in preparing specimens of sensitive soils to prevent disturbance of their natural structure. Specimens of relatively soft soils may be prepared by progressive trimming in front of a calibrated, ring-shaped specimen cutter as shown in Figure 3-10. More commonly, specimens are prepared using the trimming turntable shown in Figure 7-49 herein; the procedure, based on the use of this equipment, shall be as described in the following subparagraphs. Preferably, specimens of compacted soil should be compacted to the desired density and water content directly into the consolidation ring, in thin (1/4 to 3/8”) layers, using a pressing or kneading action of a tamper having an area less than one-sixth the area of the specimen and thoroughly scarifying the surface of each layer before placing the next. Alternatively, specimens may be trimmed from samples compacted in a compaction mould by a similar kneading action.

a. Using a wire saw, knives, or other tools, trim the specimen into approximately cylindrical shape with a diameter about 1/2” greater than the inside diameter of the specimen ring. Care should be taken to disturb the specimen as little as possible during trimming. Chamfer the lower edge of the specimen until the bottom will fit exactly into the specimen ring.

b. Place the specimen ring on the rotating wheel and the specimen on the ring, starting the bottom into the ring as shown in Figure 7-49. Use a cutting tool to trim the specimen to accurate dimensions, place a glass plate on top of the specimen, and gently force the specimen down during the trimming operation. The specimen shou1d fit snugly in the consolidation ring.

c. Cut off the portion of the specimen remaining above the ring with a wire saw or knife (or other convenient tool for harder specimens). Extreme care must be taken for many soils, especially fissured clays, in cutting off this portion. Carefully true the surface flush with the specimen ring with a straight edge. If a pebble is encountered in the surface, remove it and fill the void with soil. Place a glass plate (previously weighed) over the ring and turn the specimen over.[226] Cut off the soil extending beyond the bottom of the ring in the same manner as that described for the surface portion. Place another glass plate on this surface, and again invert the specimen to an upright position, removing the metal disk if one was used.

7.7.5. Procedure

The procedure shall consist of the following steps:

a. Record all identifying information for the specimen, such as project number, boring number, and other pertinent data, on the data sheet (Figure 11-23 is a suggested form); note any difficulties encountered in preparation of the specimen. Measure and record the height and cross-sectional area of the specimen. Record weight of specimen ring and glass plates. After specimen is prepared, record the weight of the specimen plus tare (ring and glass plates), and from the soil trimmings, obtain 200 g of material for specific gravity[227] and water content determinations. Record the wet weight of the material used for the water content determination on the data sheet.

b. Fill the grooves in the base of the consolidometer with water. Fit the porous stone (previously saturated with water) into the base of the consolidometer. Add sufficient water so that the water level is at the top of the porous stone. Place a moist filter paper (Whatman No. 1 or equal) over the porous stone. (Be very careful to avoid entrapping any air during the assembly operations.) Place the ring with the specimen therein on top of the porous stone. If the fixed-ring consolidometer is used, secure the ring to the base by means of clamps and screws.

c. Place a moist filter paper on top of the specimen, and then place the previously saturated top porous stone and the loading plate in position.

d. Place the consolidometer containing the specimen in the loading device.

e. Attach the dial indicator support to the consolidometer, and adjust it so that the stem of the dial indicator is centred with respect to the specimen. Adjust the dial indicator to permit the approximate maximum travel of the gage but still allow measurement of any swelling.

f. Adjust the loading device until it just makes contact with the specimen. The seating load should not exceed about 0.02 ksf.

g. Read the dial indicator, and record the reading on a data sheet (Figure 11-24 is a suggested form). This is the initial reading of the dial indicator.

h. With the specimen assembled in the loading device, apply a load of 0.5 ksf to the specimen and immediately inundate the specimen by filling the volume within the inundation ring or the chamber surrounding the specimen with water. If a fixed-ring device is used, a low head of water should be applied to the base of the specimen and maintained during the test by means of the standpipe. Place a thermometer in the water, and record the temperature at 2-hour intervals. To obtain reliable time-consolidation curves the temperature should not vary more than ± 2º C during the test. For most fine-grained soils a load of 0.5 ksf is usually enough to prevent swelling, but if swelling occurs apply

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[226] It may be found convenient after the top surface has been prepared to place over the specimen a circular metal plate, approximately 0.05" thick and of the same diameter as the specimen, and force it down until it is flush with the top of the ring. This provides a recess for the top porous stone and prevents the specimen from squeezing out of the consolidation ring.

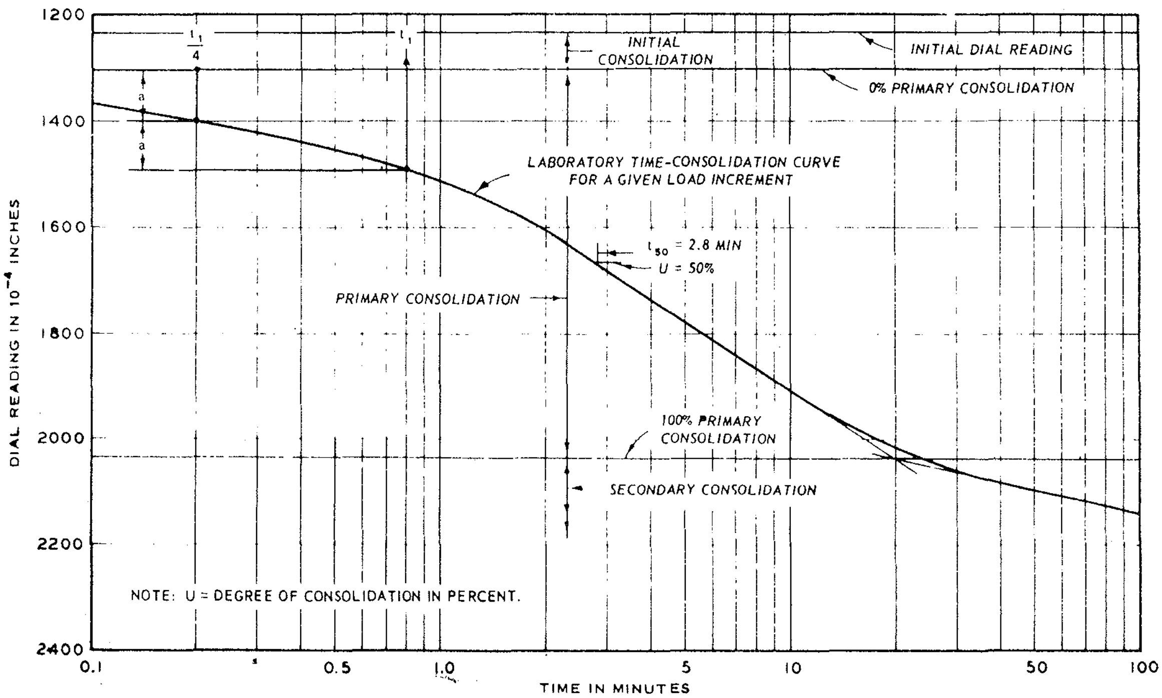
[227] It is recommended that a specific gravity test be made on representative material from every consolidation test specimen.

additional load increments until swelling ceases. Were the specimen permitted to swell, the resulting void ratio-pressure curve would have a more gradual curvature and the preconsolidation pressure would not be well defined. Alternatively to applying a large initial load increment, swelling can be prevented by not inundating the specimen until the load on the specimen has reached such a level that consolidation is obviously occurring along the straight-line portion of the void ratio pressure curve, During the stages before water is added, the humidity around the specimen should be maintained at 100% to prevent evaporation; a moist paper towel, cotton batting, or other cellular material wrapped around the specimen is usually adequate. This alternative procedure permits an initial load increment less than 0.5 kips/ft_ to be applied to the specimen.

i. Continue consolidation of the specimen by applying the next load increment. The following loading schedule is considered satisfactory for routine tests: 0.5, 1.0, 2.0, 4.0, 8.0, and 16.0 and 32.0 ksf, the total load being doubled by each load increment. The maximum load should be great enough to establish the straight-line portion of the void ratio-pressure curve, subsequently described. The designer may modify the loading schedule to simulate the loading sequence anticipated in the field.

j. Observe and record on the data sheet (Figure 11-24) the deformation as determined from dial indicator readings after various elapsed times. Readings at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 minutes, and 1, 2, 4, 8, and 24 hours for each load increment are usually satisfactory. A timing device should be located near the consolidometer to insure accurately timed measurements. Allow each load increment to remain on the specimen for a minimum of 24 hours until it is determined that the primary consolidation is completed. For most plastic, fine-grained soils, a time interval of 24 hours will be sufficient. It is desirable that the duration of all load increments be the same. During the course of the test, plot the dial le reading versus time data for each load increment on a semilogarithmic plot as shown in Figure 11-25. Plot the dial reading on an arithmetic scale (ordinate) and the corresponding elapsed time on a logarithmic scale (abscissa) as shown in Figure 11-25. For saturated fine -grained soils, the dial reading versus time curve will generally be similar to the curve shown in Figure 11-25 and can be converted into a time-consolidation curve using the theory of consolidation. The 100% consolidation or the completion of the primary consolidation is arbitrarily defined as the intersection of the tangent to the curve at the point of inflection, with the tangent to the straight-line portion representing the secondary time effect. The construction necessary for determination of the coordinates representing 100% consolidation and other degrees of consolidation is shown in Figure 11-25.

Figure 7-50 Time-Consolidation Curve



k. Record on a data sheet (Figure 11-26 is a suggested form) the dial reading for each load increment corresponding to a selected time (usually 24 hours) at which primary consolidation has been completed for all increments.

l. After the specimen has consolidated under the maximum load, remove the load in decrements, taking three-quarters of the load off successively for each of the first two decrements and as considered desirable thereafter. Take readings of the dial indicator as each decrement is removed to determine the rebound of the specimen. Observe, record, and plot the dial readings versus time; loads should not be removed until the dial readings are relatively constant with time or until the dial reading versus logarithm of time curve indicates completion of rebound. The final load at the end of the rebound cycle should be 0.2 kips/ft_ or less, and this load should be maintained for 24 hours in order to reduce to a tolerable amount the error in the final water content determination caused by swelling.

m. When the dial readings indicate no further significant rebound, remove the dial indicator and disassemble the apparatus. Carefully blot any excess water from the ring and surface of the specimen, eject the specimen into a dish of known weight, and weigh the dish and wet specimen; then oven-dry the wet specimen to constant weight.

7.7.6. Basic Computations

The computations shall consist of the following:

a. From the recorded data compute and record on the data sheet, Figure 11-23, the initial and final water contents. Compute also the height of solids, void ratio before and after test, initial and final degree of saturation, and dry density before the test using the following formulas:

Equation 7-64: $$H_s = \frac{W_s}{AG_s\gamma_w} = \left[\frac{W_s}{2.54AG_s} inches\right]$$

Equation 7-65: $$e_o = \frac{H - H_s}{H_s}$$

Equation 7-66: $$e_f = \frac{H_f - H_s}{H_s}$$

Equation 7-67: $$S_o = 100\frac{H_{wo}}{H - H_s}$$

Equation 7-68: $$S_f = 100\frac{H_{wf}}{H_f - H_s}$$

Equation 7-69: $$\gamma_d = \frac{W_s}{HA} = \left[\frac{62.4W_s}{2.54HA} lb/ft^3\right]$$

Equation 7-70: $$H_{wo} = \frac{W_{wo}}{A\gamma_w} = \left[\frac{W_{wo}}{2.54A} in.\right]$$

Equation 7-71: $$H_{wf} = \frac{W_{wf}}{A\gamma_w} = \left[\frac{W_{wf}}{2.54A} in.\right]$$

Where

- Hs = Height of Solids
- e_o = Void Ratio Before Test
- e_f = Void Ratio After Test
- S_o = Initial Degree of Saturation, Percent
- S_f = Final Degree of Saturation, Percent
- γ_d = Dry Density Before Test
- H_{wo} = Original Height of Water
- H_{wf} = Final Height of Water
- W_s = Weight of Dry soil, g
- A = Area of Specimen, cm^2
- G_s = Specific Gravity of Solids
- γ_w = Unit Weight of Water, g/cm^3
- H = Height of Specimen, inches
- H_f = Height of Specimen at End of Test = H – ΔH is the net change in height of specimen

The purpose of computing the degree of saturation at the beginning and end of the test is to obtain a check on the accuracy of the data observed and recorded. An appreciable variation from 100% in the computed degree of saturation at the beginning of the test for specimens that are known to the completely saturated may indicate the presence of gas or air in the specimen, or an error in the data or computations.

b. From Figure 11-24 or from Figure 11-25, obtain the final dial reading for each load increment that corresponds to the selected time interval (usually 24 hours) and record these values on Figure 11-26. On the same data sheet, record the dial reading correction. The dial reading correction is the dial reading corresponding to the deformation of the apparatus for each load, and is obtain from a calibration curve for the apparatus as described earlier. The height of the voids H_v corresponding to any given load is equal to the initial height of voids ($H - H_s$) minus the corrected dial reading (ΔH). The change in height of the specimen equal to the accumulative change of the corrected dial readings. Compute the void ratios of the specimen corresponding to different load increments. The void ratio is numerically equal to the height of the voids divided by the height of the solids.

7.7.7. Other Quantities Obtainable from Consolidation Testing

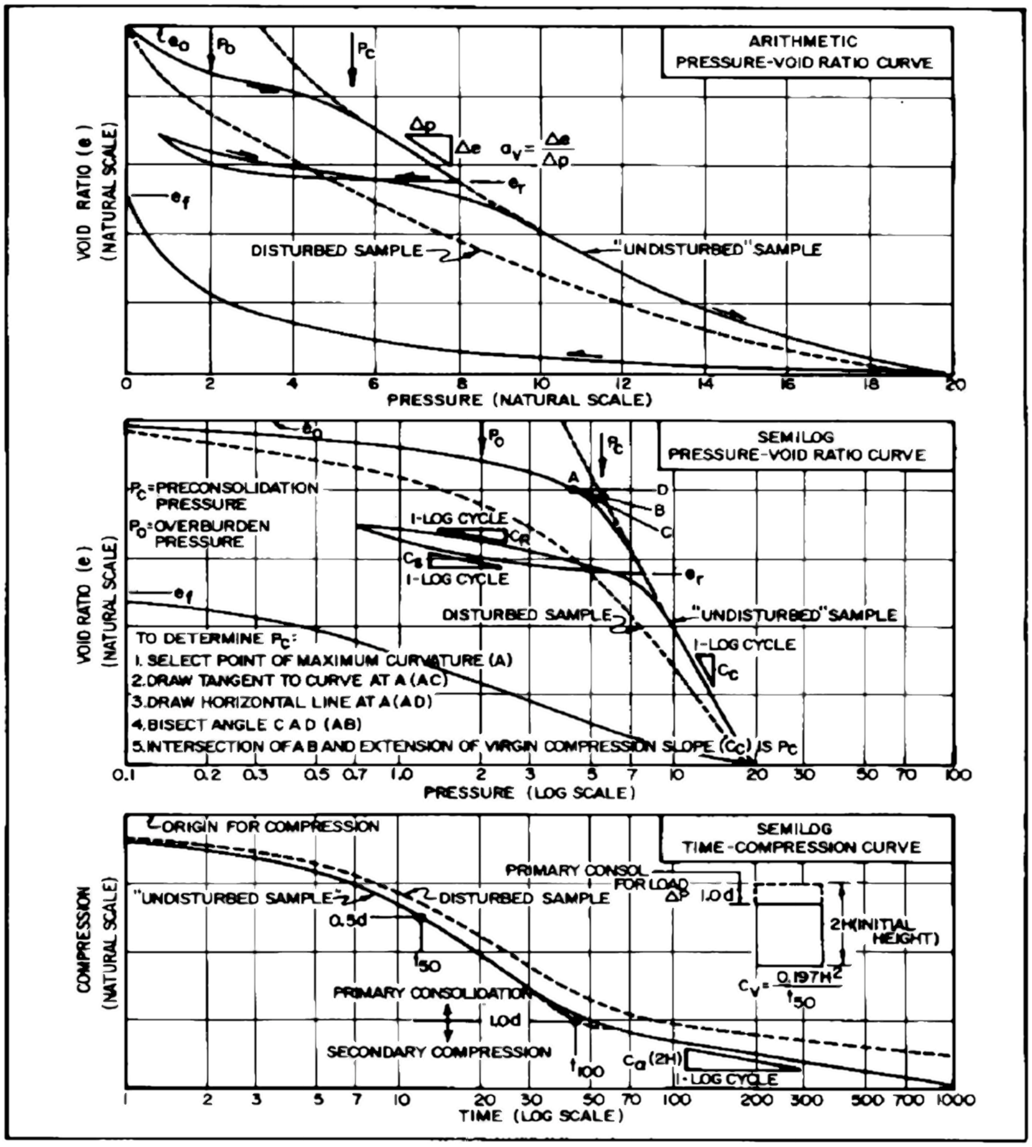
7.7.7.1. Preconsolidation Pressure

The maximum pressure to which a soil has been loaded in the past will have a major influence on the amount of settlement to be expected under a proposed loading. In fact, ten times more settlement may occur in an unconsolidated soil than a preconsolidated soil for equal load increments. Normally, a maximum and minimum value of P_c will be established and plotted as a range with depth. This forms the boundary between recompression and virgin compression ranges and is approximately the maximum normal effective stress to which the material *in situ* has been consolidated by a previous loading. Desiccation produces a similar effect. The preconsolidation pressure cannot be determined precisely, but can be estimated from consolidation tests on high quality undisturbed samples.

Estimate preconsolidation pressure from semilogarithmic pressure-void ratio curve using the procedure given in the central panel of Figure 7-51.[228] On the semilogarithmic pressure-void ratio plot (e-log P), the engineer can readily see the sharp break in the curve at Pc which indicates compression will increase rapidly for additional increases in load beyond the preconsolidation pressure. Maximum test pressures should exceed preconsolidation by an amount sufficient to define the slope of virgin compression. Generally, this requires application of three or more load increments exceeding the preconsolidation value.

[228] Alternative methods are given in Leonards, G.A., Editor, Foundation Engineering, McGraw Hill, 1962, and Schmertmann, J.M., The Undisturbed Consolidation of Clay, Transactions, American Society of Civil Engineers, Vol. 120, p 1201, 1955.

Figure 7-51 Consolidation Test Relationships



7.7.7.2. Virgin Compression

Virgin compression is deformation caused by loading in the range of pressures exceeding that to which the sample has been subjected in the past. The semilogarithmic, pressure-void ratio curve is roughly linear in the virgin range, and its slope on a semilogarithmic diagram is known as the compression index, C_C. The compression index is defined by the equation

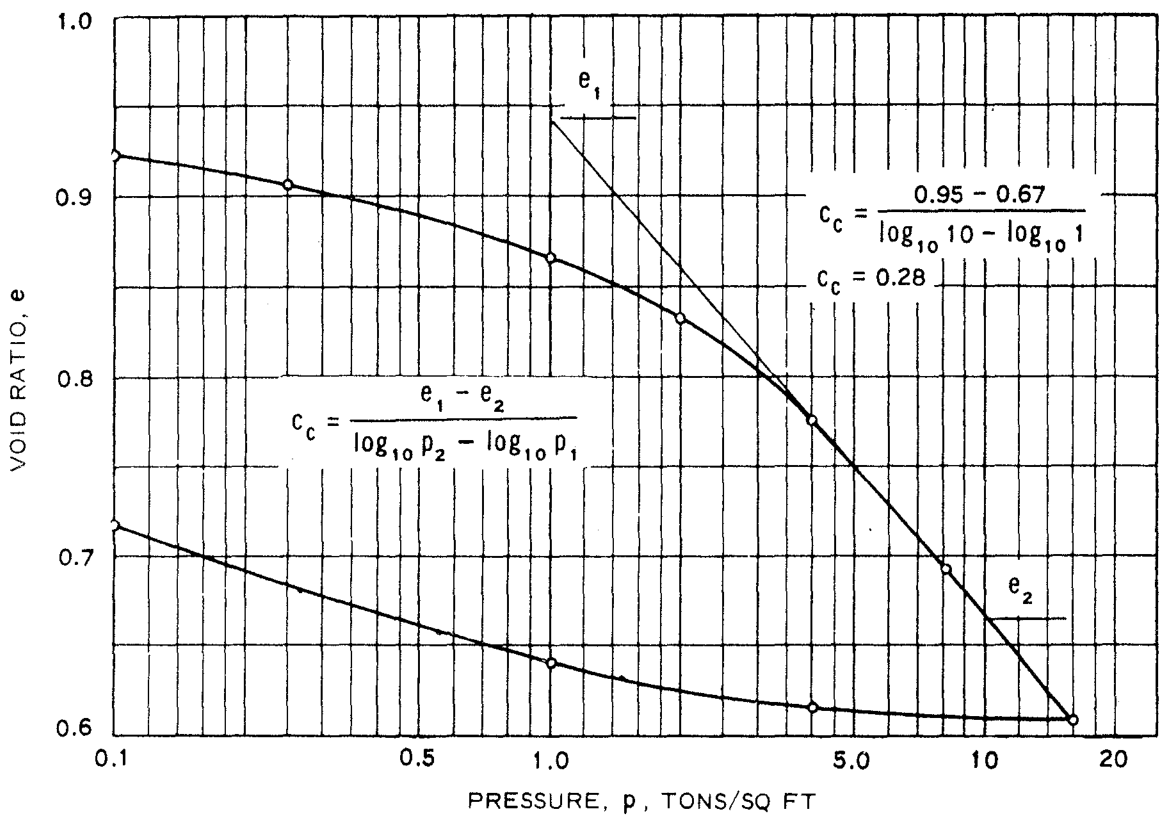
$$\textbf{Equation 7-72: } C_C = \frac{e_1 - e_2}{\log_{10} p_2 - \log_{10} p_1}$$

where

- p_1 and p_2 = selected pressures from the straight-line portion of the curve
- e_1 and e_2 = corresponding void ratios

The compression index is a measure of the compressibility of a soil. An example of the computation of C_c is shown in Figure 7-52. For simplification, p_2 is often chosen to be 10 times p_1, in which case the denominator becomes unity.

Figure 7-52 Void Ratio-Pressure Curve



When soils are preconsolidated, the slopes of virgin compression and recompression portions of the e-log P curve are respectively C_c and C_r. In general, C_c is approximately 10 times greater than C_r. The point where lines drawn tangent to the slopes intersect is the minimum preconsolidation pressure. The C_c and C_r values are respectively estimated by dividing the soil moisture content by 100 and 1000. As their names imply, the values are a direct measure of soil compression. This is shown in Figure 7-14.

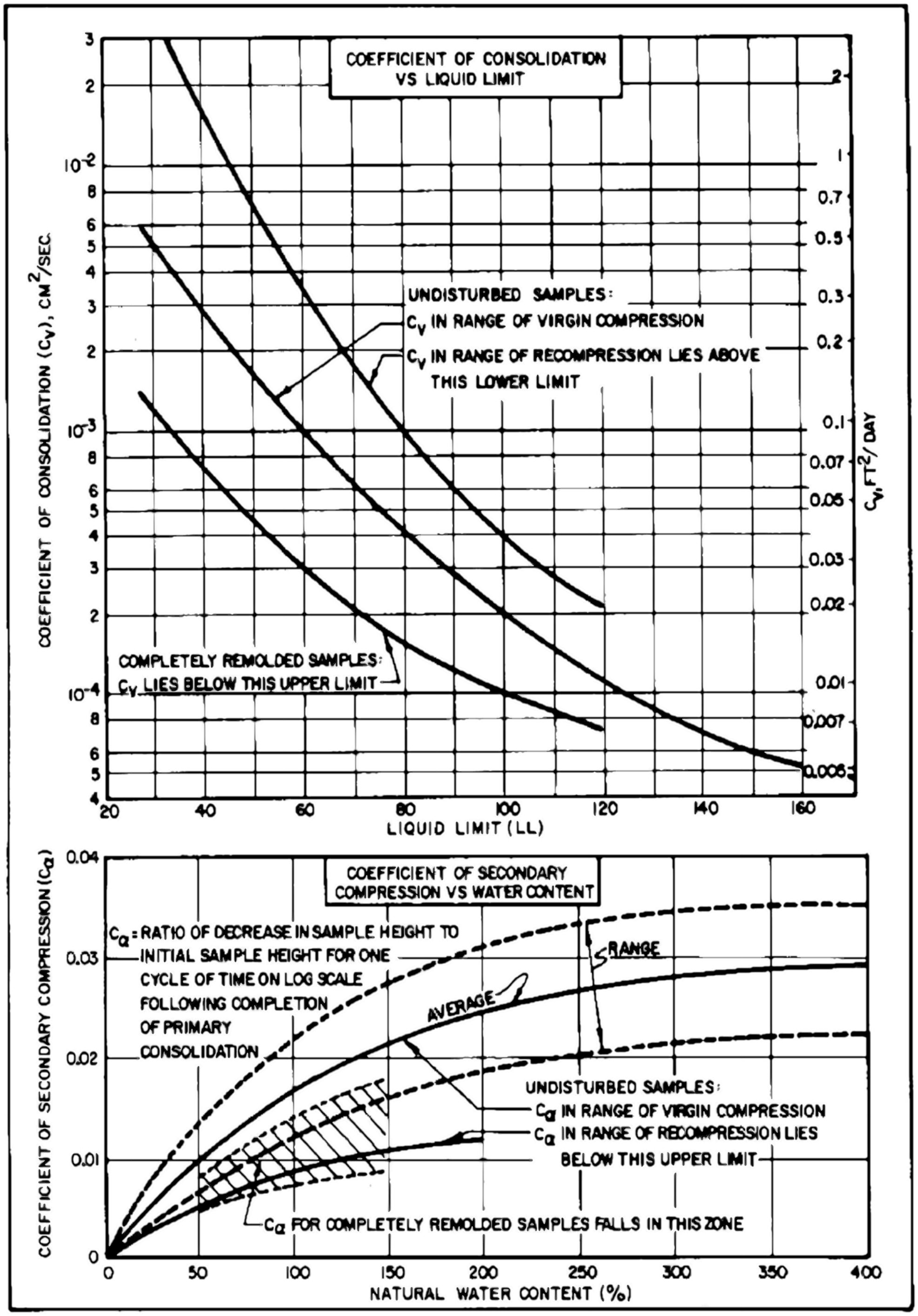
Approximate values can be found for C_c in Table 7-12.

7.7.7.3. Coefficient of Consolidation (C_v)

Those soil properties that control the drainage rate of pore water during consolidation are combined in the coefficient of consolidation. This parameter is an indicator of the rate of drainage during consolidation; or in the case of pile driving an indicator of the time required for remolded soil to gain strength and reconsolidate around the pile. A plot of C_v versus log P will show a sharp decrease at the preconsolidation pressure.

- Determination. Compute C_v from the semilogarithmic time-compression curve for a given load increment (bottom panel of Figure 7-51). Correct the origin for compression for the effect of air or gas in void spaces.
- Approximate Values. Figure 7-53 may be used to determine approximate values of C_v.

Figure 7-53 Approximate Correlations for Consolidation Characteristics of Silts and Clays



7.7.7.4. Secondary Compression

After completion of primary consolidation under a specific load, the semilogarithmic time-compression curve continues approximately as a straight line. This is termed secondary compression. It occurs when the rate of compression is no longer primarily controlled by the rate at which pore water can escape; there are no excess pore pressures remaining.

1. Organic Materials. In organic materials, secondary compression may dominate the time-compression curve, accounting for more than one-half of the total compression, or even obliterating the change in slope used to establish the limit of primary compression.
2. Approximate Values. The coefficient of secondary compression C_α is a ratio of decrease in sample height to initial sample height for one cycle of time on log scale. See bottom panel of Figure 7-53 for typical values.

7.7.8. Presentation of Results

The results of the consolidation tests shall be shown on the report forms, Figure 11-25 and Figure 11-26. The data shall be shown graphically in terms of time-consolidation curves in the form shown as Figure 11-25 and in terms of void ratio -pressure curves in the form shown as Figure 11-27. To obtain the void ratio pressure curve, the void ratio, e, is plotted on the arithmetic scale (ordinate) and the corresponding pressure, p, in tons/square foot on the logarithmic scale (abscissa) as shown in Figure 7-52. The overburden pressure, p_o, preconsolidation pressure, p_c and compression index, C_c, shall be determined and shown on the report form (Figure 11-27). The determinations of the preconsolidation and overburden pressures of a soil are normally made by design engineers; this is discussed elsewhere in this book.

If permeability tests are performed in conjunction with the consolidation test (see 6.8), the coefficient of permeability for each load increment shall also be plotted in the form shown as Figure 11-27. A brief description of undisturbed specimens should be given on the report form. The description should include color, approximate consistency, and any unusual features (such as stratification, fissures, shells, roots, sand pockets, etc.). For compacted specimens, give the method of compaction used and the relation to maximum density and optimum water content.

7.7.9. Possible Errors

Following are possible errors that would cause inaccurate determinations of consolidation characteristics:

a. Specimen disturbance during trimming. As in all laboratory determinations of the engineering properties of undisturbed soils, changing the natural structure of the soil while preparing the test specimen causes the largest errors; disturbance will affect the time deformation relation and will obscure the preconsolidation pressure. Trimming must be done in the humid room with every care taken to minimize the disturbance. Since the zone of disturbance caused by trimming a given soil is essentially constant in depth, using a larger specimen can reduce the effect of this zone.

a. Specimen not completely filling ring. The volume of the specimen must be exactly that of the consolidation ring. Otherwise, there will not be complete lateral confinement.

b. Galvanic action in consolidometer. To prevent changes in the consolidation characteristics of the specimen due to galvanic currents, all metal parts of the consolidometer should be of the same noncorrosive material; it is preferable that all such parts be made of plastic.

c. Permeability of porous stones too low. The measured rate of consolidation can be markedly affected by the permeability of the porous stones. The stones should be cleaned after every test to remove embedded soil particles.

d. Friction between specimen and consolidation ring. Tests have shown that over 20% of the load applied to a specimen can be lost by side friction in a fixed-ring consolidometer and about one-half this amount in a floating-ring conso1idometer.[229] The effect of side friction can be reduced by (i)

[229] D. W. Taylor, Research on the Consolidation of Clays, Serial 82, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology (Cambridge, Mass., 1942).

using a larger diameter specimen, (2) using a thinner specimen, and (3) lining the consolidation ring with Teflon.

e. Inappropriate load increment factor. Depending on the purpose of the test, a load increment factor of 2.0 (that is, of doubling the total load by each load increment) may not be satisfactory; for example, to sharply define the preconsolidation pressure, a factor of 1.50 or even 1.25 may be better.

f. Unsatisfactory height (or thickness) of specimen. The height of the specimen will determine how clearly can be detected the break in the time-consolidation curve that represents completion of primary consolidation. Depending on the character of the soil, if the specimen is too thin, the time to 100% consolidation may be too rapid, while if too thick, the break in the curve may be obscured by secondary compression. In addition, when a load increment factor smaller than 2.0 is used, the thickness of the specimen may have to be increased to cause enough deformation during primary consolidation to define the break in the curve.

7.7.10. Other Test Methods

7.7.10.1. Rapid Load Test (RL)

The RL test is similar to the SL test except much larger pressure increment ratios may be used and the duration of each pressure increment is restricted. The time duration is usually limited to allow only 90% of full consolidation as evaluated by Taylor's square root of time method, Table 7-16.[230]

7.7.10.1.1. Time Requirements

This test may be performed in a single day.

7.7.10.1.2. Accuracy

Accuracy is similar to the SL procedure.

7.7.10.1.3. Pressure Increments

Large pressure increments exceeding those of the SL test for applied pressures exceeding the maximum past pressure reduces the amount of secondary compression contained in the void ratio-logarithm time curve, Figure 7-19.

7.7.10.2. Constant Rate of Deformation Test (CRD)

A thin, cylindrical soil specimen similar to that of the SL test is saturated at constant volume under a backpressure and loaded vertically without lateral strain at a constant rate of vertical strain. Drainage is permitted only from the upper surface of the specimen. A general-purpose consolidometer capable of this test procedure is shown in Figure 7-54.

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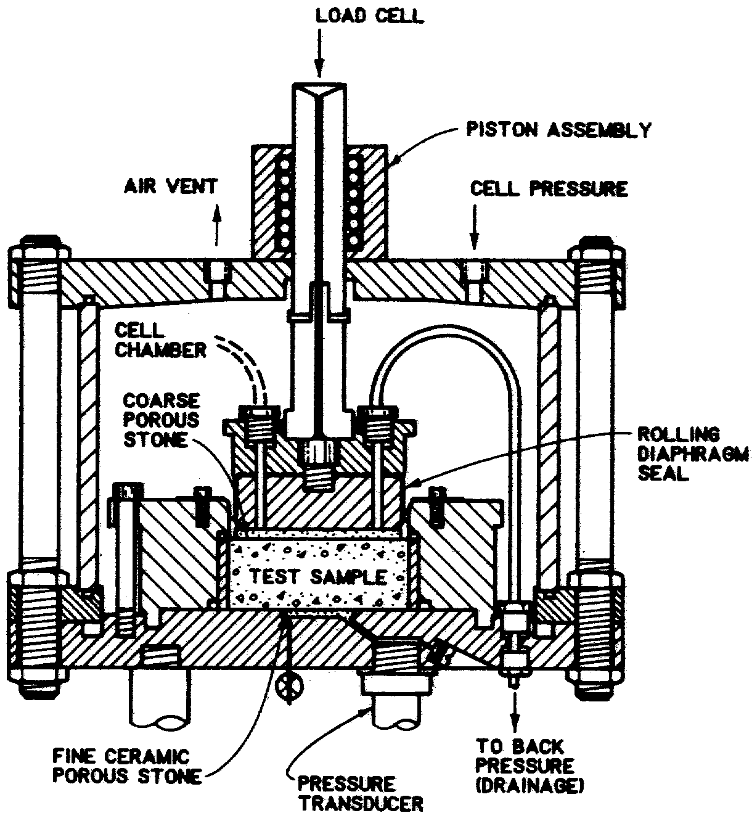
[230] Newland, P. L. and Allely, B. H. 1960. "A Study of the Consolidation Characteristics of a Clay," Geotechnique, Vol 10, pp 62-74, The Institution of Civil Engineers. Available from Thomas Telford Ltd., 1-7 Great George Street, Westminster, London, SW1P 3AA, England.

Figure 7-54 General-purpose consolidometer[231]



7.7.10.2.1. Evaluation of Maximum Past Pressure

The maximum past pressure determined by this procedure is dependent on the rate of strain and increases with increase in strain rate. The strain rate should be consistent with expected field rates. Typical field strain rates are about 10^{-7} per minute.

7.7.10.2.2. Evaluation of Coefficient of Consolidation

c_v should be evaluated by[232]

Equation 7-73: $$c_v = \frac{h^2 \log_{10} \frac{\sigma_2}{\sigma_1}}{2\Delta t \log_{10}\left[1 - \frac{u_h}{\sigma_a}\right]}$$

Where

[231] Reprinted by permission of the American Society of Civil Engineers from the Journal of the Soil Mechanics and Foundations Division, Vol 97, 1971, "Consolidation at Constant Rate of Strain", by A. E. Z. Wizza, J. T. Christian, and E. H. Davis, p 1394

[232] Wissa, A. E. Z., Christian, J. T., and Davis, E. H. 1971. "Consolidation at Constant Rate of Strain," Journal of the Soil Mechanics and Foundations Division, Vol 97, pp 1393-1414. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

- h = average thickness of drainage path (specimen thickness), in.
- σ_1 = total vertical stress at time t_1, psi
- σ_2 = total vertical stress at time t_2, psi
- $\Delta t = t_2 - t_1$, minutes
- u_h = average excess pore water pressure at the bottom of the specimen over the time interval t_1 and t_2, psi
- $\sigma_a = (\sigma_1 + \sigma_2)/2$, average total vertical stress over time interval t_1 and t_2, psi

The coefficient of consolidation c_v determined by this method appears comparable with that of the SL test.

7.7.10.2.3. Evaluation of Void Ratio-Logarithm Pressure Relationship

The void ratio-logarithm pressure relationship may be obtained by determining the void ratio and effective stress at any time during the test.

(1) The change in void ratio Δe over a pressure increment is $(1 + e_1)\varepsilon$, where e_1 is the void ratio at the beginning of the pressure increment and ε is the strain over the pressure increment.

(2) The average effective stress over a pressure increment is $\sigma_a - u_h$.

(3) The excess pore water pressure u_h is measured at the bottom of the specimen, Figure 7-54.

7.7.10.2.4. Assumptions

This method assumes that the coefficient of consolidation and compression index are both constant for the soil.

7.7.10.3. Controlled Gradient Test (CG)

This test is similar to the CRD test except that the applied vertical pressure is adjusted so that the pore water pressure at the bottom of the specimen remains constant throughout the test[233]. This restriction requires a feedback mechanism that significantly complicates the laboratory equipment.

7.7.10.3.1. Evaluation of Coefficient of Consolidation

The coefficient of consolidation may be estimated by

Equation 7-74: $$c_v = \frac{\Delta\sigma h_a^2}{2\Delta t u_h}$$

Where

- $\Delta\sigma$ = change in total pressure between time increment t_2 and t_1, psi
- $\Delta t = t_2 - t_1$, minutes
- h_a = average height of specimen between time t_2 and t_1, in.
- u_h = excess pore water pressure at bottom of the specimen, psi

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233 Lowe, J., Jonas, E., and Obrician, V. 1969. "Controlled Gradient Consolidation Test," Journal of the Soil Mechanics and Foundations Division, Vol 95, pp 77-97. Available from American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

7.7.10.3.2. Evaluation of Void Ratio-Logarithm Pressure Relationship

The void ratio-logarithm pressure relationship is evaluated similar to the above procedures. The excess pore water pressure at the bottom of the specimen should be kept as small as possible to maintain a nearly uniform void ratio within the specimen.

7.7.10.3.3. Assumptions

This method assumes that the coefficient of consolidation and coefficient of volume change are constant. The coefficient of volume change is the change in strain divided by the change in total vertical stress.

7.8. Analysis of Volume Expansion

7.8.1. Causes of Volume Expansion

Volume expansion is caused by:

1) Reduction of effective stresses
2) Mineral changes, and
3) Formation and growth of ice lenses.

Swell with decrease of effective stress is a reverse of the consolidation process. For description of swelling problems and suggested treatment, see Table 7-34. Where highly preconsolidated plastic clays are present at the ground surface, seasonal cycles of rainfall and desiccation produce volume changes. The most severe swelling occurs with montmorillonite clays although, in an appropriate climate, any surface clay of medium to high plasticity with relatively low moisture content can heave.

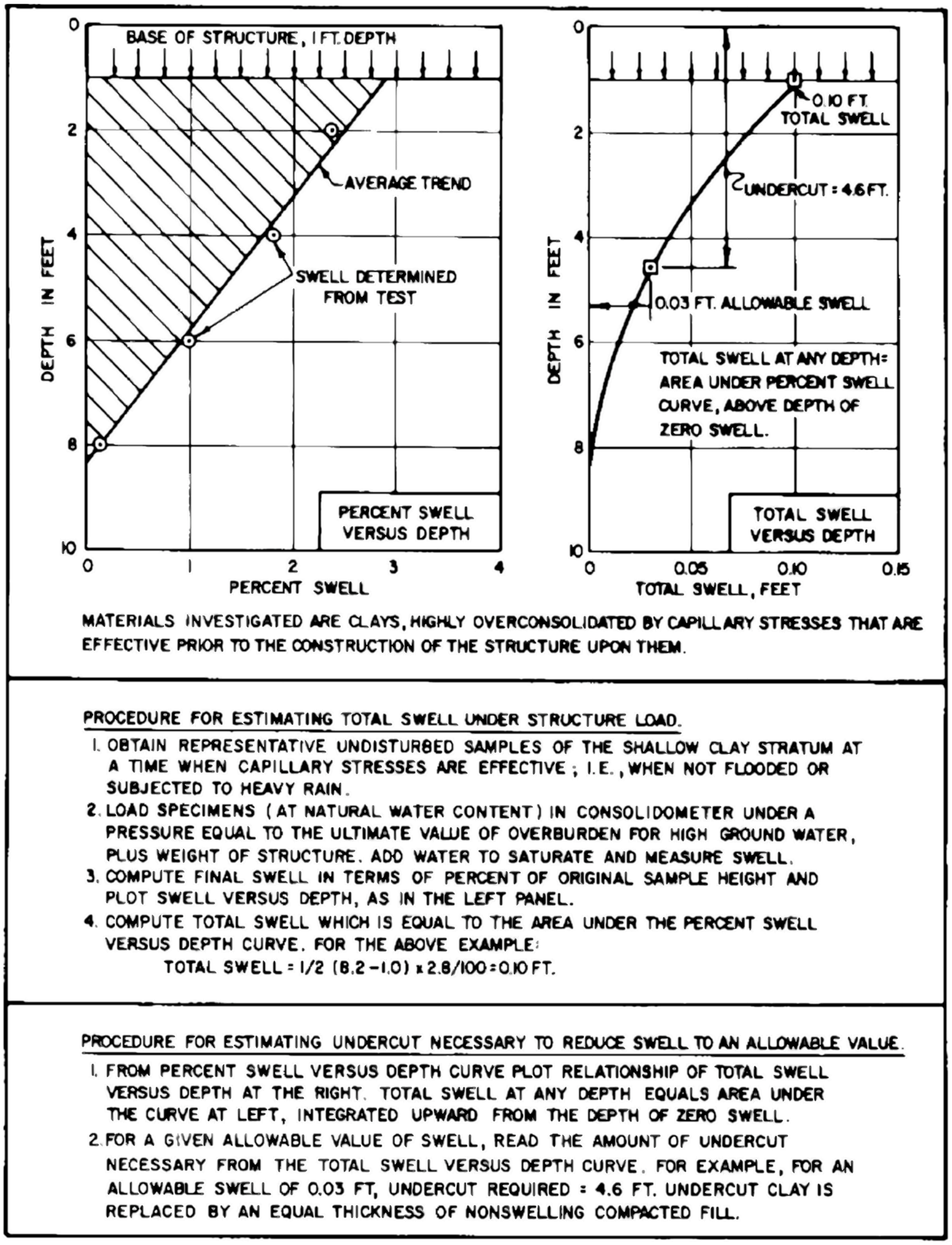
Table 7-34 Heave from Volume Change

Conditions and materials	Mechanism of heave	Treatment
Reduction of effective stress of overburden:		
Temporary reduction of effective stress by excavation for structure foundation in preconsolidated clays.	Soil swells in accordance with laboratory e-p curves. Heave is maximum at center of excavation. Total potential heave may not have occurred by time the load is reapplied. Final structural load will recompress foundation materials.	Provide drainage for rapid collection of surface water. Avoid disturbance to subgrade by placing 4-in.-thick working mat of lean concrete immediately after exposing subgrade. Heave is minimized if the groundwater is drawn down 3 or 4 ft below base of excavation at its center to maintain capillary stresses.
Permanent reduction of effective stress by excavation in chemically inert, uncemented clay-shale or shale.	In sound shale where water cannot obtain access to the shale, swelling may be insignificant.	Protect shale from wetting and drying during excavation by limiting area opened at subgrade and with concrete working mat. Pour concrete floors and foundations directly on protected shale with no underfloor drainage course. Backfill around walls with impervious soils to prevent access of water. Provide proper surface drainage and paving if necessary to avoid infiltration.
	For hydraulic structures or construction below the ground water table, reduction of effective stresses will cause permanent heave in accordance with laboratory e-p curves. Alternate wetting and drying during excavation increases swelling potential.	Where an increase in water content is probable, special structural designs must be considered. These include (1) anchoring or rock bolting the floor to a depth in shale that provides suitable hold down against swelling pressures; (2) a floor supported on heavily loaded column footings with an opening or compressible filler beneath floors; and (3) a mat foundation designed to resist potential swelling pressures. In any case, excavation in the shale should be protected by sealing coats or working mat immediately after exposure at subgrade.
Reduction of effective stress of overburden and release of capillary stress:		
Construction of earth dams of heavily compacted plastic clays.	Intrusion of seepage from reservoir releases capillary pressures and reduces effective stress of overburden and may produce swelling leading to sloughing of the slopes. Most critical material are CH clays with swelling index exceeding 0.07. Compaction at relatively low water contents, where the water deficiency in the clay mineral lattice is high and the degree of saturation is low, will accentuate swelling.	Avoid placing highly plastic fill on or near embankment slopes. Compact clays at a relatively high moisture content consistent with strength and compressibility requirements. Avoid overcompaction to an unnecessarily high dry unit weight.
Construction of structural fill for light buildings of compacted plastic clay.	Rise of groundwater, seepage, leakage, or elimination of surface evaporation increases degree of saturation and reduces effective stress, leading to expansion.	Compact clays as wet as practicable consistent with compressibility requirements. Avoid overcompaction of general fill and undercompaction of backfill at column footings or in utility trenches which would accentuate differential movements. Stabilization of compacted fills with various salt admixtures reduces swelling potential by increasing ion concentration in pore water.
Changes of capillary stresses:		
Construction of light buildings on surface strata of highly preconsolidated clays in temperate climates subject to substantial seasonal fluctuations in rainfall. (Southern England, as an example.)	Seasonal movements 1 or 2 in. upwards and downwards occur within the upper 3 to 5 ft. Settlement occurs in early summer and expansion in the fall. Caused by change of capillary stresses produced by transpiration to nearby trees, plant, or grass cover surrounding the structure. Movements are maximum at edge of building. Groundwater is shallow. Change of capillary stresses by evaporation is not of prime importance.	Light reinforcing or stiffening minimize effects in small houses. Basements carried to usual depths usually eliminate movements.
Construction of light buildings on clays of high activity, highly preconsolidated with fractures and slickensides, in climate where hot summers alternate with wet winters. (South-central Texas for example.)	Even in the absence of vegetal cover, seasonal cycles of settlement and heave occur because of the alternate increase and release of capillary stresses. Buildings constructed during wet season may undergo small but nonuniform settlement beneath exterior footings. Buildings constructed in the dry season undergo uneven heave up to 3 or 4 in. maximum, distributed irregularly over the structure.	Support light footings and slabs on compacted, coarse-grained fill about 4 to 6 ft thick. Pave peripheral areas to minimize subsoil moisture content change. Consider the use of belled caissons with supported floor. Open block wall foundations have been utilized for small houses. Collect rainwater falling on structure and surrounding areas and convey runoff away from structures.
Construction of light to medium load structures in hot, arid climate where the free surface evaporation is several times larger than annual rainfall. Difficulties are greatest in fractured and slickensided clay of high activity, with low water table and maximum deficiency of evaporation over rainfall. (South Africa, as an example.)	Permanent moisture deficiency exists in the ground. Construction eliminates evaporation over building area, reducing capillary stresses and causing movement of moisture to beneath building. This leads to continuing heave with minor seasonal fluctuations. Thermoosmotic gradients directed toward cooled subsoil beneath structure contribute to increase in moisture, which may extend to depths of 10 to 15 ft.	Damage is minimized by use of slab or raft foundation, dry wall construction, steel or reinforced concrete framing, reinforced foundation beams, and provision for jacking. Heave is eliminated by removal of desiccated material to a depth of 8 to 12 ft.and replacement by granular fill; or belled caissons, founded near the water table and reinforced to resist tensile forces, supporting floor between caissons with opening or compressible filler beneath floors. Divert rainwater and surface runoff away from structure.
Chemical changes:		
Excavation and exposure of clay-shales or shales containing pyrite (iron sulphide) or anhydrite (calcium sulphate).	Exposure to air and water causes oxidation and hydration of pyrites with a volumetric expansion of as much as ten times their original volume, or hydration of anhydrite to gypsum.	Rough excavate no closer than one-half foot to final subgrade and protect exposed shale with a spray or mop coat of bitumen. When ready for foundations, excavate to final grade and pour concrete immediately over a spray or mop coat of bitumen.

7.8.2. Magnitude of Volume Expansion

Figure 7-55 outlines a procedure for estimating the magnitude of swelling that may occur when footings are built on expansive clay soils. This figure also indicates a method of determining the necessary undercut to reduce the heave to an acceptable value.

Figure 7-55 Computation of Swell of Desiccated Clays



7.8.3. Swell and Swell Pressure Tests

7.8.3.1. Introduction

Swell is the process of imbibing available moisture to cause an increase in soil volume until the pore water pressure increases to an equilibrium determined by the environment. The amount of swell to satisfy the new pore pressure equilibrium depends on the magnitude of the vertical loading and soil properties that include soil composition, natural water content and density, and soil structure. The rate of swell depends on the coefficient of permeability (hydraulic conductivity), thickness, and soil properties. Soils that are more likely to swell appreciably include clays and clay shales with plasticity indices greater than 25, liquid limits greater than 40, and natural water contents near the plastic limit or less.

The presence of capillary stress or negative pore water pressure arising from molecular forces in swelling soils causes available moisture to be absorbed. The vertical confining pressure required to prevent volume expansion from absorbed moisture is defined as the swell pressure.

The swell and swell pressure are generally determined in the laboratory with the one-dimensional consolidometer (7.7). Swell is determined by subjecting the laterally confined soil specimen to a constant vertical pressure and by giving both the top and bottom of the specimen access to free water (usually distilled) to cause swell. The swell pressure is determined by subjecting the laterally confined soil specimen to increasing vertical pressures, following inundation, to prevent swell.

7.8.3.2. Apparatus, Calibration of Equipment, And Preparation of Specimens

The apparatus is essentially the same as that listed in 7.7.2. Smoothly ground porous stones should be used in the consolidometer to minimize seating displacements. Filter paper should not be used because it is compressible and contributes to displacements. The equipment is calibrated and the sample prepared in the same manner as described for the consolidation test.

7.8.3.3. Procedure

7.8.3.3.1. Swell Test

(1) Record all identifying information for the specimen, such as project number, boring number, and other pertinent data, on the data sheet (Figure 11-23 is a suggested form); note any difficulties encountered in preparation of the specimen. Measure and record the height and cross-sectional area of the specimen. Record weight of specimen ring and glass plates. After the specimen is prepared, record the weight of the specimen plus tare (ring and glass plates), and from the soil trimmings, obtain 200 g of material for specific gravity and water content determinations. Record the weight of the material used for the water content determination on the data sheet.

(2) Fit an air-dried, smoothly ground porous stone into the base of a dry consolidometer. Place the ring with the specimen therein on top of the porous stone. If the fixed-ring consolidometer is used, secure the ring to the base by means of clamps and screws.

(3) Place the top air-dried, smoothly ground porous stone and loading plate in position. The inside of the reservoir should be moistened to promote a high-humidity environment. The reservoir and loading plate should subsequently be covered with a sheet of impervious material such as plastic film or moist paper towel to inhibit loss of moisture.

(4) Place the consolidometer containing the specimen in the loading device.

(5) Attach the dial indicator support to the consolidometer, and adjust it so that the stem of the dial indicator is centred with respect to the specimen. Adjust the dial indicator to allow for both swell and consolidation measurements.

(6) Adjust the loading device until it just makes contact with the specimen. The seating load should not exceed about 0.02 ksf.

(7) Read the dial indicator and record the reading on a data sheet (Figure 11-24 is a suggested form). This is the initial reading of the dial indicator.

(8) Depending on the particular design considerations, a specific load (e.g., overburden plus design load) is applied to the specimen. After a period of at least 5 minutes but less than 30 minutes (to avoid

shrinkage from drying), record the dial reading on the data sheet (Figure 11-24) and inundate the specimen.

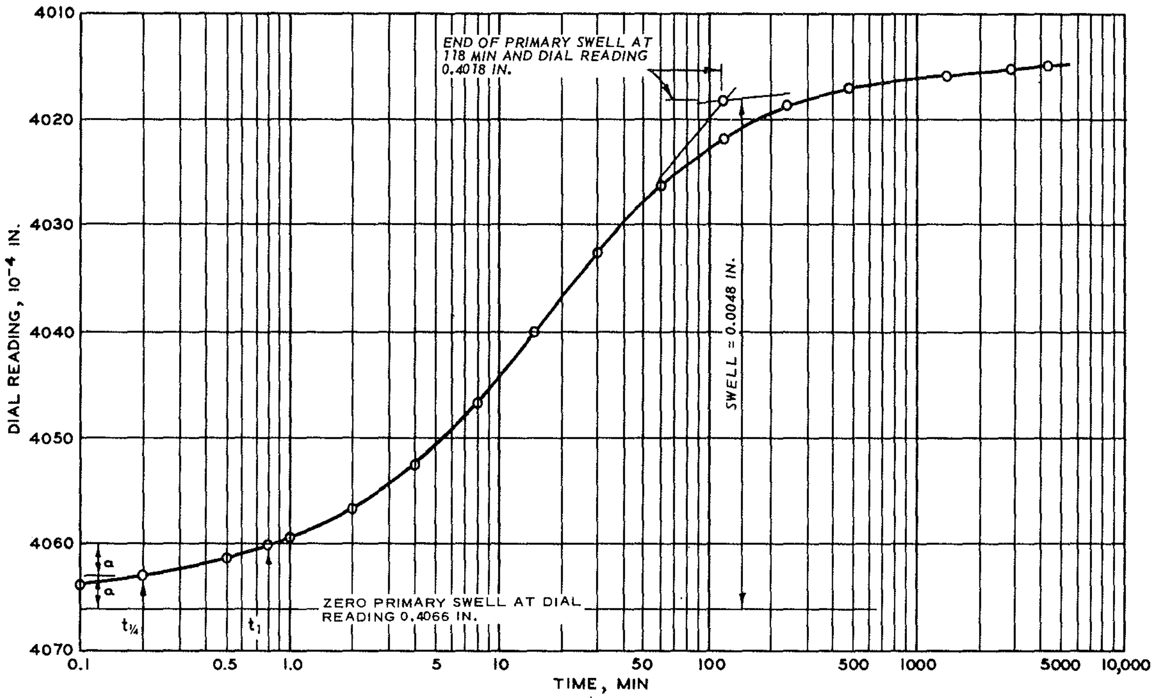
(9) Inundate the specimen by filling the reservoir, within the inundation ring that surrounds the specimen, with water (distilled, tap, or field pore water, actual or reconstituted). Cover with the plastic films and moist paper towel or equivalent. If a fixed-ring device is used, a low head of water should be applied to the base of the specimen and maintained during the test by means of the sand pipe.

(10) Observe and record on the data sheet (Figure 11-24) the deformation as determined from dial indicator readings after various elapsed times. Readings at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 minutes, and 1, 2, 4, 8, 24, 48, and 72 hours are usually satisfactory. A timing device should be located near the consolidometer to ensure accurately timed measurements. Allow the load increment to remain on the specimen until it is determined that the primary swell is completed. Time to complete the primary swell of heavy clay soils and clay shales often requires three or more days. Plot the dial reading versus time data on a semilogarithmic plot as shown in Figure 11-23 to ascertain the completion of primary swell. The completion of primary swell is arbitrarily defined as the intersection of the tangent to the curve at the point of inflection, with the tangent to the straight-line portion representing a secondary time effect as shown in Figure 7-56. This is similar to the procedure in 7.7.5.

(11) Although a falling-head permeability test may be performed at this point of the swell test (see 6.8.7), the specimen may not be fully saturated and the permeability results consequently affected. After primary swell is complete,[234] the load should be removed in decrements according to the procedure in 7.7.5l and 7.7.5m. The final load should be the seating load.

[234] After primary swell is complete, the loading pressure may be increased to consolidate the specimen until the void ratio is less than the initial void ratio under the overburden pressure; thereafter, loads may subsequently be removed to determine rebound characteristics. This procedure permits an alternative measure of the swell pressure, defined as the total pressure required to reduce the void ratio to the initial void ratio.

Figure 7-56 Time-Swell Curve



7.8.3.3.2. Swell Pressure Test

The procedure of this test is identical with the preceding swell test through (9). Following (9), increments of load are applied as needed to prevent swell. Variations from the dial reading at the time the specimen is inundated with water should be kept preferably within 0.0002" and not more than 0.0005". Following 24 hours after the specimen exhibits no further tendency to swell, a falling-head permeability test may be performed (see 6.8.7) and the final load (which is also the swell pressure) should be removed in decrements according to the procedure in 7.7.5l and 7.7.5m. The final load should be the seating load.

7.8.3.4. Computations

The computations for the swell tests are similar to those presented in 7.7.6.

7.8.3.5. Presentation of Results

The results of the swell tests shall be summarised on report forms Figure 11-23 and Figure 11-24. The data shall be shown graphically in terms of time-swell curves on the form shown as Figure 11-23 and in terms of void ratio-pressure curves shown as Figure 11-24. To obtain the void ratio-pressure curve, the void ratio e is plotted on the arithmetic scale (ordinate) and the corresponding pressure p in kips or tons/ft_ on the logarithmic scale (abscissa) as shown in Figure 7-57 and Figure 7-58. The overburden pressure p_o, swell pressure p_s, swell index C_s, and swell at p_o, _H/H, shall be determined and shown on the report form in Figure 11-24. The design engineers normally make the determination of the overburden pressure.

Figure 7-57 Void ratio-pressure curve of swell test

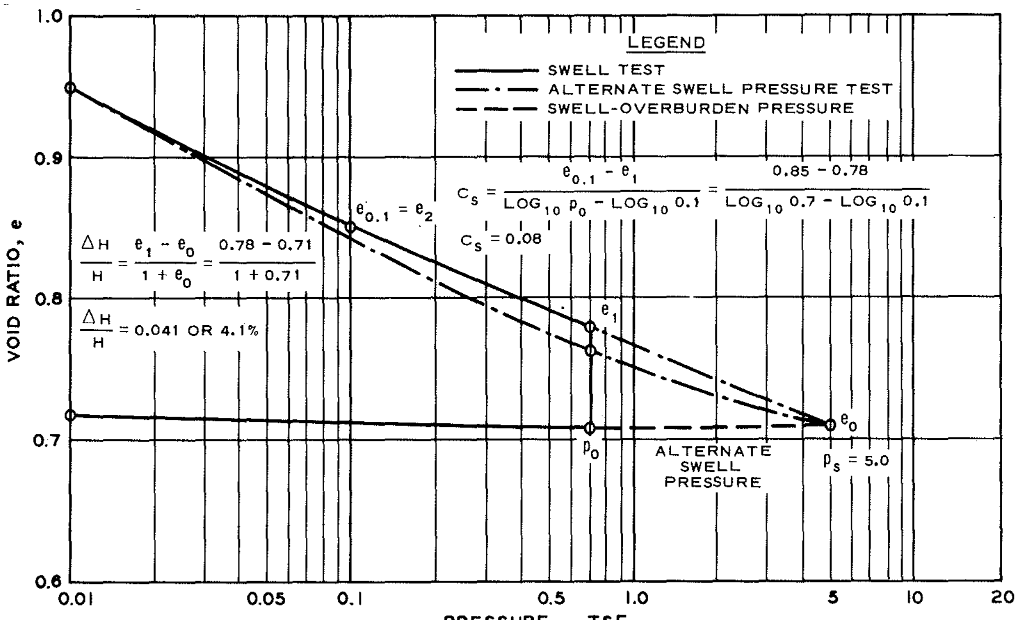
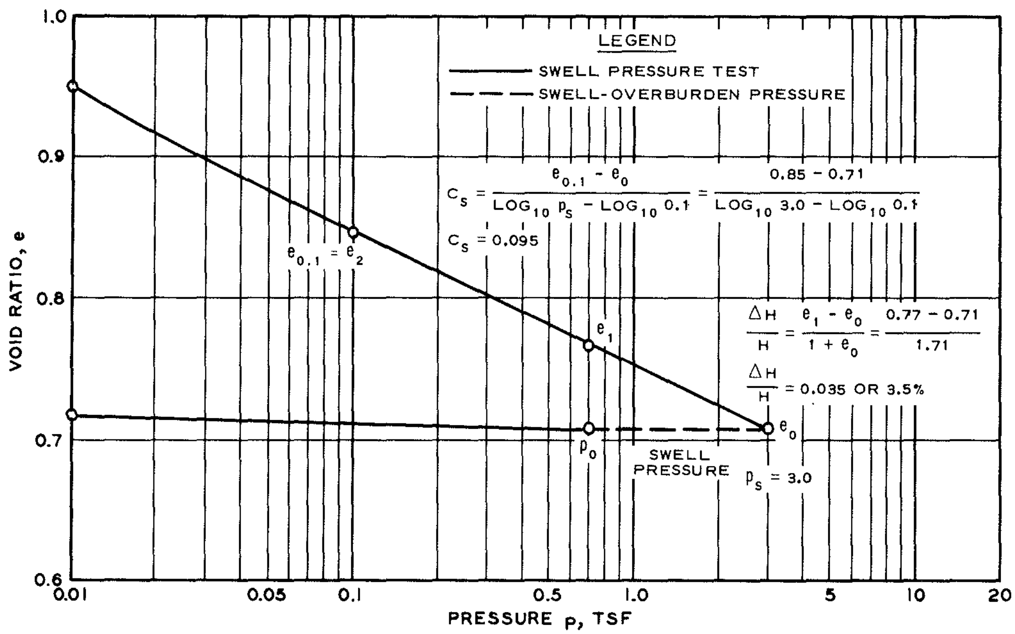


Figure 7-58 Void ratio-pressure curve of swell pressure test



The swell pressure p_s is determined as in Figure 7-58. The swell pressure by the alternative definition[234] may also be determined from the results of the swell test (Figure 7-57) and recorded on Figure 11-24. The initial void ratio is the void ratio determined following placement of the overburden pressure.

The swell index is defined by the equation

Equation 7-75: $$C_s = \frac{e_2 - e_1}{\log_{10} p_1 - \log_{10} p_2}$$

Where the variables are the same as for Equation 7-72. The swell index is the measure of the ability of the soil to swell. Example computations of C_s are shown in Figure 7-57 and Figure 7-58.

The swell is defined by the equation

Equation 7-76: $$\frac{\Delta H}{H} = \frac{e_1 - e_0}{1 + e_0}$$

Where e_1 is the void ratio following swell, and e_0 is the void ratio prior to swell. Example computations are shown in Figure 7-57 and Figure 7-58 for swell at the overburden pressure p_o.

If permeability tests are performed in conjunction with the well tests (see 6.8) the coefficient of permeability determined for each void ratio during rebound shall also be plotted in the form shown as Figure 11-24.

A brief description of undisturbed specimens should be given on the report form. The description should include color, approximate consistency, and any unusual features (such as stratification, fissures, shells, roots, sand pockets, etc.). For compacted specimens, give the method of compaction used and the relation to maximum density and optimum water content.

7.8.3.6. Possible Errors

In addition to the possible errors discussed in 7.7.9, the following may also cause inaccurate determination of swelling characteristics:

a. Displacements caused by seating of the specimen against the surface of the porous stones may be significant, especially if swell displacements and loading pressures are small. Thus, smoothly ground porous stones are recommended.

b. Filter paper is highly compressible and contributes to the observed displacements and hysteresis in displacements on loading and rebound. Filter paper is consequently not recommended.

c. The compressibility characteristics of the consolidometer and the test procedures influence the swell pressure results. Because very small expansions in volume greatly relieve swell pressures, the stiffness of the consolidometer should be as large as possible, and variations in displacements that occur during determination of the swell pressure should be as small as possible.

d. Swelling characteristics determined by consolidometer swell tests for the purpose of predicting heave of foundation and compacted soils are not representations of many field conditions because:

   (1) Lateral swell and lateral confining pressures are not simulated.

   (2) The actual availability of water to the foundation soils may be cyclic or intermittent. Field swell usually occurs under constant overburden pressure depending on the availability of water. The swell index, in contrast, is evaluated by observing swell due to decreases in overburden pressure while the soil specimen is inundated with water.

   (3) Rates of swell indicated by swell tests are not reliable indicators of field rates of swell due to fissures in the mass soil and inadequate simulation of the actual availability of water to the soil.

   (4) Secondary or long-term swell, which is not evaluated by these test procedures, may be significant for some clays and clay shales. These soils may not be fully saturated at the conclusion of the swell test.

(5) Chemical content of the inundating water affects results; e.g., when testing shales, distilled water may give radically different results than natural or reconstituted pore water.

§ 8. Slope Stability and Protection

8.1. Introduction

This section presents methods of analyzing stability of natural slopes and safety of embankments. Diagrams are included for stability analysis, and procedures for slope stabilization are discussed.

Overstressing of a slope or reduction in shear strength of the soil may cause rapid or progressive displacements. Thus, embankment stability must be assured prior to consideration of other foundation related items. Embankment foundation problems involve the support of the embankment by natural soil. Problems with embankments and structures occasionally occur which could be prevented by initial recognition of the problem and appropriate design. Stability problems most often occur where the embankment is to be built over soft weak soils such as low strength clays, silts, or peats. Once the soil profile, soil strengths, and depth of water table have been determined by field explorations and field and lab testing, the stability of slopes may be evaluated by comparison of the forces resisting failure with those tending to cause rupture along the assumed slip surface. The ratio of these forces is the factor of safety.[235]

8.2. Types of Failures

8.2.1. Modes of Slope Failure

Principal modes of failure in soil or rock are

1) Rotation on a curved slip surface approximated by a circular arc;
2) Displacement of a wedge-shaped mass along one or more planes of weakness;
3) Lateral squeeze of foundation soil.

Other modes of failure include toppling of rockslopes, falls, block slides, lateral spreading, earth and mud flow in clayey and silty soils, and debris flows in coarse-grained soils. Table 8-1 and Table 8-2 show more detailed examples of potential slope failure problems in both natural and man-made slopes.[236]

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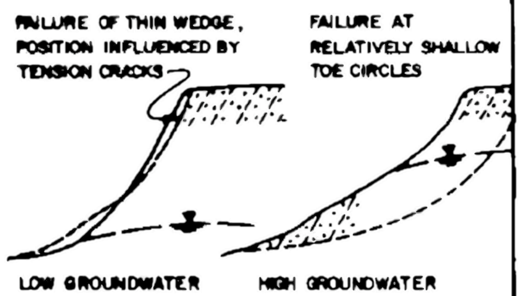
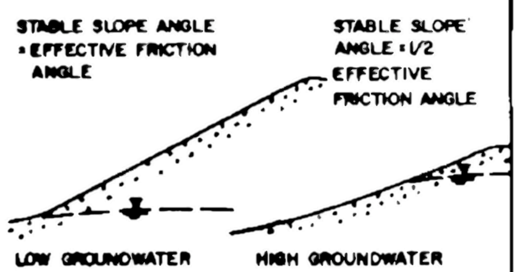
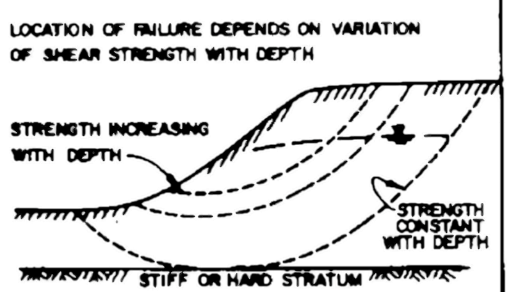
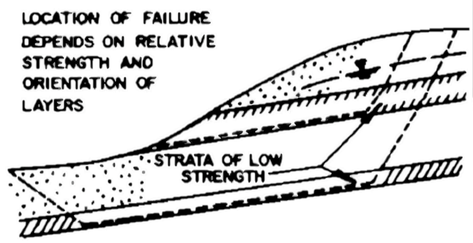
[235] For detailed treatment on the subject, see Transportation Research Board, Landslide Analysis and Control, Special Report 176, 1978.

[236] The stability problems outlined above are "external" stability problems. "Internal" embankment stability problems involve selection of embankment materials and placement requirements. Internal stability may be "ordered" in project specifications by specifying minimum gradation and compaction requirements.

Table 8-1 Analysis of Stability of Natural Slopes

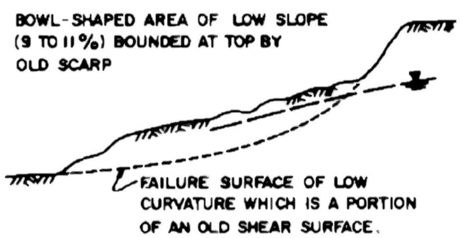
 (1) SLOPE IN COARSE-GRAINED SOIL WITH SOME COHESION	WITH LOW GROUNDWATER, FAILURE OCCURS ON SHALLOW, STRAIGHT, OR SLIGHTLY CURVED SURFACE. PRESENCE OF A TENSION CRACK AT THE TOP OF THE SLOPE INFLUENCES FAILURE LOCATION. WITH HIGH GROUNDWATER, FAILURE OCCURS ON THE RELATIVELY SHALLOW TOE CIRCLE WHOSE POSITION IS DETERMINED PRIMARILY BY GROUND ELEVATION. ANALYZE WITH EFFECTIVE STRESSES USING STRENGTHS c' AND ϕ' FROM CD TESTS. PORE PRESSURE IS GOVERNED BY SEEPAGE CONDITION. INTERNAL PORE PRESSURES AND EXTERNAL WATER PRESSURES MUST BE INCLUDED.
 (2) SLOPE IN COARSE-GRAINED, COHESIONLESS SOIL	STABILITY DEPENDS PRIMARILY ON GROUNDWATER CONDITIONS. WITH LOW GROUNDWATER, FAILURES OCCUR AS SURFACE SLOUGHING UNTIL SLOPE ANGLE FLATTENS TO FRICTION ANGLE. WITH HIGH GROUNDWATER, STABLE SLOPE IS APPROXIMATELY 1/2 FRICTION ANGLE. ANALYZE WITH EFFECTIVE STRESSES USING STRENGTH ϕ'. SLIGHT COHESION APPEARING IN TEST ENVELOPE IS IGNORED. SPECIAL CONSIDERATION MUST BE GIVEN TO POSSIBLE FLOW SLIDES IN LOOSE, SATURATED FINE SANDS.
 (3) SLOPE IN NORMALLY CONSOLIDATED OR SLIGHTLY PRECONSOLIDATED CLAY	FAILURE OCCURS ON CIRCULAR ARCS WHOSE POSITION IS GOVERNED BY THEORY, SEE FIG. 3. POSITION OF GROUNDWATER TABLE DOES NOT INFLUENCE STABILITY UNLESS ITS FLUCTUATION CHANGES STRENGTH OF THE CLAY OR ACTS IN TENSION CRACKS. ANALYZE WITH TOTAL STRESSES, ZONING CROSS SECTION FOR DIFFERENT VALUES OF SHEAR STRENGTHS. DETERMINE SHEAR STRENGTH FROM UNCONFINED COMPRESSION TEST, UNCONSOLIDATED UNDRAINED TRIAXIAL TEST OR VANE SHEAR.



(4) SLOPE IN STRATIFIED SOIL PROFILE

LOCATION OF FAILURE PLANE IS CONTROLLED BY RELATIVE STRENGTH AND ORIENTATION OF STRATA. FAILURE SURFACE IS COMBINATION OF ACTIVE AND PASSIVE WEDGES WITH CENTRAL SLIDING BLOCK CHOSEN TO CONFORM TO STRATIFICATION.

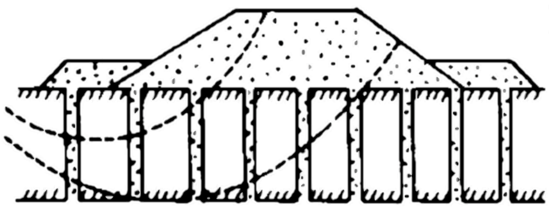
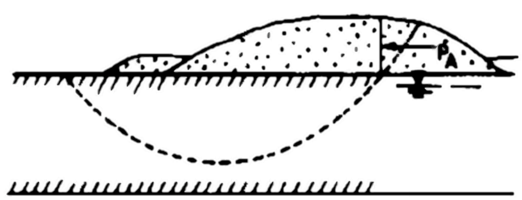
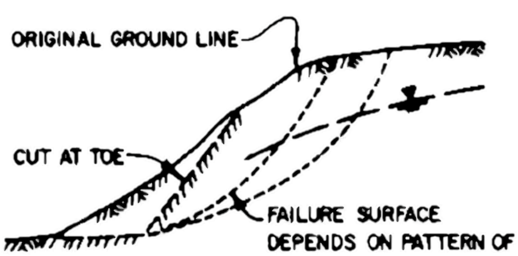
ANALYZE WITH EFFECTIVE STRESS USING c' AND ϕ' FOR FINE-GRAINED STRATA AND ϕ' FOR COHESIONLESS MATERIAL.



(5) DEPTH CREEP MOVEMENTS IN OLD SLIDE MASS

STRENGTH OF OLD SLIDE MASS DECREASES WITH MAGNITUDE OF MOVEMENT THAT HAS OCCURRED PREVIOUSLY. MOST DANGEROUS SITUATION IS IN STIFF, OVER-CONSOLIDATED CLAY WHICH IS SOFTENED, FRACTURED, OR SLICKENSIDED IN THE FAILURE ZONE.

Table 8-2 Analysis of Stability of Cut and Fill Slopes, Conditions Varying with Time

LOCATION OF FAILURE DEPENDS ON GEOMETRY AND STRENGTH OF CROSS SECTION.  (1) FAILURE OF FILL ON SOFT COHESIVE FOUNDATION WITH SAND DRAINS	USUALLY MINIMUM STABILITY OCCURS DURING PLACING OF FILL. IF RATE OF CONSTRUCTION IS CONTROLLED, ALLOW FOR GAIN IN STRENGTH WITH CONSOLIDATION FROM DRAINAGE. ANALYZE WITH EFFECTIVE STRESS USING c' AND ϕ' FROM CU TEST WITH PORE PRESSURE MEASUREMENT. APPLY ESTIMATED PORE PRESSURES OR PIEZOMETRIC PRESSURES. ANALYZE WITH TOTAL STRESS FOR RAPID CONSTRUCTION WITHOUT OBSERVATION OF PORE PRESSURES, USE SHEAR STRENGTH FROM UNCONFINED COMPRESSION OR UNCONSOLIDATED UNDRAINED TRIAXIAL.
FAILURE SURFACE MAY BE ROTATION ON CIRCULAR ARC OR TRANSLATION WITH ACTIVE AND PASSIVE WEDGES.  (2) FAILURE OF STIFF COMPACTED FILL ON SOFT COHESIVE FOUNDATION	USUALLY, MINIMUM STABILITY OBTAINED AT END OF CONSTRUCTION. FAILURE MAY BE IN THE FORM OF ROTATION OR TRANSLATION, AND BOTH SHOULD BE CONSIDERED. FOR RAPID CONSTRUCTION IGNORE CONSOLIDATION FROM DRAINAGE AND UTILIZE SHEAR STRENGTHS DETERMINED FROM U OR UU TESTS OR VANE SHEAR IN TOTAL STRESS ANALYSIS. IF FAILURE STRAIN OF FILL AND FOUNDATION MATERIALS DIFFER GREATLY, SAFETY FACTOR SHOULD EXCEED ONE, IGNORING SHEAR STRENGTH OF FILL. ANALYZE LONG-TERM STABILITY USING C AND ϕ FROM CU TESTS WITH EFFECTIVE STRESS ANALYSIS, APPLYING PORE PRESSURES OF GROUNDWATER ONLY.
 (3) FAILURE FOLLOWING CUT IN STIFF FISSURED CLAY	RELEASE OF HORIZONTAL STRESSES BY EXCAVATION CAUSES EXPANSION OF CLAY AND OPENING OF FISSURES, RESULTING IN LOSS OF COHESIVE STRENGTH. ANALYZE FOR SHORT TERM STABILITY USING c' AND ϕ' WITH TOTAL STRESS ANALYSIS. ANALYZE FOR LONG TERM STABILITY WITH c'_r AND ϕ'_m BASED ON RESIDUAL STRENGTH MEASURED IN CONSOLIDATED DRAINED TESTS.

8.2.2. Causes of Slope Failure

Slope failures occur when the rupturing force exceeds resisting force.

∞ Natural Slopes. Imbalance of forces may be caused by one or more of the following factors:

  - A change in slope profile that adds driving weight at the top or decreases resisting force at the base. Examples include steepening of the slope or undercutting of the toe.
  - An increase of groundwater pressure, resulting in a decrease of frictional resistance in cohesionless soil or swell in cohesive material. Groundwater pressures may increase through the saturation of a slope from rainfall or snowmelt, seepage from an artificial source, or rise of the water table.
  - Progressive decrease in shear strength of the soil or rock mass caused by weatheirng, leaching, mineralogical changes, opening and softening of fissures, or continuing gradual shear strain (creep).
  - Vibrations induced by earthquakes, blasting, or pile driving. Induced dynamic forces cause densification of loose sand, silt, or loess below the groundwater table or collapse of sensitive clays, causing increased pore pressures. Cyclic stresses induced by earthquakes may cause liquefaction of loose, uniform, saturated sand layers.
- Embankment (Fill) Slopes. Failure of fill slopes may be caused by one or more of the following factors:
  - Overstressing of the foundation soil. This may occur in cohesive soils, during or immediately after embankment construction. Usually, the short-term stability of embankments on soft cohesive soils is more critical than the long-term stability, because the foundation soil will gain strength as the pore water pressure dissipates. However, it may be necessary to check the stability for a number of pore pressure conditions. Usually, the critical failure surface is tangent to the firm layers below the soft subsoils.
  - Drawdown and Piping. In earth dams, rapid drawdown of the reservoir causes increased effective weight of the embankment soil thus reducing stability. Another potential cause of failure in embankment slopes is subsurface erosion or piping.
  - Dynamic Forces. Earthquakes, blasting, pile driving, etc., may induce vibrations.
  - Fills on Clays. Excess pore pressures are created when fills are placed on clay or silt. As the pore pressure dissipates, consolidation occurs, and the clay or silt strength increases. This is the reason the factor of safety increases with time.
- Excavation (Cut) Slopes. Failure may result from one or more of the factors described above. Additionally, as a cut is made in clay, the effective stress is reduced. This will allow the clay to expand and absorb water, which will lead to a decrease in the clay strength with time. This is the reason the factor of safety of clay cut slope decreases with time. Cut slopes in clay should be designed using effective strength parameters and the effective stress that will exist after the cut is made. An increase in absorbed moisture is a major factor in the decrease in strength of cohesive soils. Water is absorbed by clay minerals and high water contents decrease cohesion of all clayey soils.
  - In cohesionless soils, water does not affect the angle of internal friction. The effect of water on cohesionless soils below the water table is to decrease the intergranular (effective) pressure between soil grains (due to buoyancy), and this decreases the frictional shearing resistance.

8.2.3. Effect of Soil or Rock Type

- Failure Surface. In homogeneous cohesive soils, the critical failure surface usually is deep whereas shallow surface sloughing and sliding is more typical in homogeneous cohesionless soils. In nonhomogeneous soil foundations the shape and location of the failure depends on the strength and stratification of the various soil types.

- Rock. Slope failure is common in stratified sedimentary rocks, in weathered shales, and in rocks containing platy minerals such as talc, mica, and the serpentine minerals. Failure planes in rock occur along zones of weakness or discontinuities (fissures, joints, faults) and bedding planes (strata). The orientation and strength of the discontinuities are the most important factors influencing the stability of rock slopes. Discontinuities can develop or strength can change because of the following environmental factors:
    - Chemical weathering.
    - Freezing and thawing of water/ice in joints.
    - Tectonic movements.
    - Increase of water pressures within discontinuities. Sudden moisture increase in a dry soil can produce a pore pressure increase in trapped pore air accompanied by local soil expansion and strength decrease. The "slaking" or sudden disintegration of hard shales, claystones, and siltstones result from this mechanism. If placed as rock fill, water percolating through the fill causes these materials to disintegrate to a clay soil, which often leads to settlement and/or shear failure of the fill.
    - Alternate wetting and drying (especially expansive shales).
    - Increase of tensile stresses due to differential erosion.

8.3. Methods of Analysis

Experience and observations of failures of embankments built over relatively deep deposits of soft foundation soils have shown that when failure occurs, the embankment sinks down, the adjacent ground rises and the failure surface follows a circular arc.

- *The failure force* (driving force) consists of the weight of the embankment. The overturning moment is the product of the weight of the embankment (acting through its centre of gravity) times the lever arm distance to the centre of rotation (L_w).
- *The resisting force* against movement is the sum at all soil shear strength (friction and cohesion) acting along the failure arc. The resisting moment is the product of the shear strength times the radius of the circle (L_s).

The factor of safety against overturning is equal to the ratio of the resisting moment to overturning moment.

Equation 8-1: $$F = \frac{\sum s_{total} L_s}{\sum W L_W}$$

Where

- F = factor of safety
- s_{total} = Total shear strength
- $(s_{total})(L_s)$ = Resisting Moment
- W = Weight Force
- $(W)(L_w)$ = Overturning Moment

When the factor of safety is less than 1, failure will take place.

A simple rule of thumb can be used to make a preliminary estimate of the factor of safety against circular arc failure for an embankment built on a clay foundation.

Equation 8-2: $$F_{estimate} = \frac{6c}{\gamma_{fill} H_{fill}}$$

Where

- c = cohesion strength of clay
- γ_{fill} = Fill Soil Unit Weight
- H_{fill} = Fill Height

The factor of safety computed using this rule of thumb should never be used for final design. The simple equation obviously does not take into account such factors as fill strength or fill slope angle and does not identify the location of a critical failure surface. If the factor of safety using the rule of thumb is less than 2.5, a more sophisticated stability analysis is required.

However, this rule of thumb can be helpful very early in the design stage to make a quick preliminary check on whether stability may be a problem and if analyses that are more detailed should be conducted. It can also be of use in the field while the boring and sampling is being done. For example, if *in situ* vane shear tests are being carried out as part of the field investigation, the soils engineer or geologist, can use the vane strength with the rule of thumb equation to estimate the F.S. in the field. This can aid in directing the drilling, sampling, and testing program while the drill crew is at the site and help insure that critical strata are adequately explored and sampled. Finally, the simple rule of thumb factor of safety can be used to check for gross errors in computer output or input.

Various techniques of slope stability analysis may be classified into four broad categories that follow.

8.3.1. Limit Equilibrium Methods

Most limit equilibrium methods used in geotechnical practice assume the validity of Coulomb's failure criterion along an assumed failure surface. A free body of the slope is considered to be acted upon by known or assumed forces. Shear stresses induced on the assumed failure surface by the body and external forces are compared with the available shear strength of the material. This method does not account for the load deformation characteristics of the materials in question. Most of the methods of stability analysis currently in use fall in this category.

The method of slices, which is a rotational failure analysis, is most commonly used in limit equilibrium solutions. The minimum factor of safety is computed by trying several circles. The difference between various approaches stems from (a) the assumptions that make the problem determinate, and (b) the equilibrium conditions that are satisfied. The soil mass within the assumed slip surface is divided into several slices, and the forces acting on each slice are considered. The effect of an earthquake may be considered by applying appropriate horizontal force on the slices.

8.3.1.1. Fellenius Method

8.3.1.1.1. Overview

The simplest and most basic limit equilibrium method is known as the Fellenius Method of Slices.[237] The Fellenius Method assumes that the moment arm is the same for both the driving and resisting forces, i.e, that these forces are collinear. For this method Equation 8-1 reduces to

[237] This method was developed by the Swedish geotechnical pioneer Wolmar Fellenius (1876-1957). His grandson Bengt is in his own right eminent in pile dynamics and deep foundations. The Fellenius Method as shown here is enhanced by details from Coduto, D.P., *Geotechnical engineering: Principles and Practices*. Upper Saddle River, NJ: Prentice-Hall, 1999.

Equation 8-3: $F = \frac{\sum s_{total}}{\sum W}$

The free body diagram shows the failure surface is divided into slices and the following basic assumptions are made:

∞ The available shear strength of the soil can be adequately described by the Mohr-Coulomb model, Equation 5-10.

∞ The factor of safety is the same for all slices

∞ The factors of safety with respect to cohesion (c) and friction (tan ϕ) are equal.

∞ All forces (shear and normal) on the sides of each slide are ignored.

∞ The pore water pressure is taken into account by reducing the total weight of the slice by the water uplift force acting against the slice base

To compute the factor of safety for an approach embankment using the Fellenius method, the step-by-step computational procedure is as follows:

1. Draw cross-section of embankment and foundation soil profile using either 1" = 10 feet or 1" = 20 feet scale both horizontal and vertical.
2. Select a circular failure surface.
3. Divide the circular mass above the failure surface into 10-15 vertical slices.[238]
4. Compute The Total Weight W_t Of Each Slice. The weight of each slice is the product of the cross-sectional area of the slice and the unit weight of the soil (assuming a unit weight of embankment thickness is assumed for computational purposes.) If groundwater is present the buoyant effect of the water must be taken into account, as will be shown in the examples below.
5. Compute $\overline{\sigma}_v \tan\phi$ (frictional resisting force) for each slice. Note that the effect of water is to reduce the normal force against the slice base and thus reduce the frictional resisting force ($\overline{\sigma}_v \tan\phi$).
6. Compute CL (resisting force due to cohesion for each slice.)
7. Compute tangential driving forces.
8. Sum resisting forces and driving forces for all slices and compute factor of safety. The computations for the last four steps should be entered systematically in a form. Obviously, such a computation is ideal for a spreadsheet program.

The Fellenius method is generally conservative but does not taken into consideration the friction between the slices, which compromises its accuracy.

8.3.1.2. Other Methods of Slices

There are many other stability analysis methods available besides the Fellenius Method, such as Bishop's Method, Janbu's Method, etc. These methods are primarily variations and refinements of the basic method of slices. The differences in the more refined methods lie in the assumption made regarding the

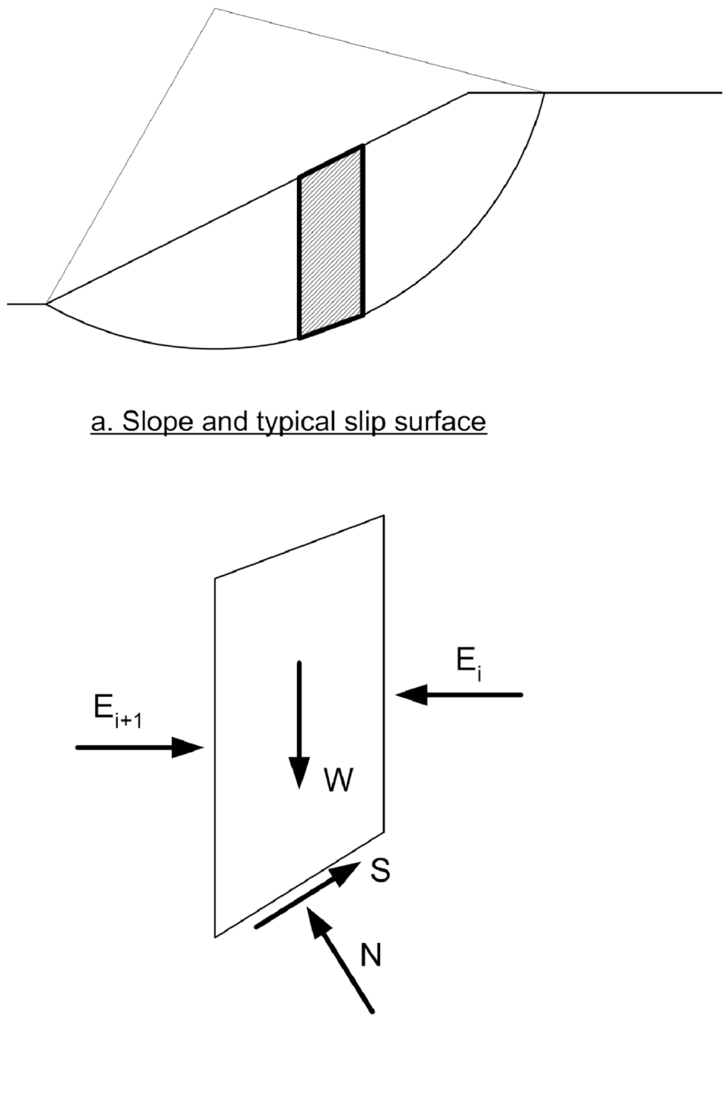
[238] To simplify computation, locate the vertical sides of the slices so that the bottom of any one slice is located entirely in a single soil layer or at the water level-circle intersection, and locate all vertical slice boundaries at breaks in the slope. The slice widths do not have to be equal. For convenience, assume a one foot thick section of embankment (this simplifies computation of driving and resisting forces).

shear and normal forces made on the sides of slices. The method of analysis that should be used to determine a factor of safety depends on the soil type, the source of and confidence in the soil strength parameters, and the type of slope that is being designed. Only qualified experienced geotechnical personnel should perform soil design analyses.

8.3.1.3. Simplified Bishop Method

This procedure is based on the assumption that the interslice forces are horizontal, as shown in Figure 8-1.

Figure 8-1 Typical Slices and Forces for the Simplified Bishop Method



a. Slope and typical slip surface

b. Typical slice

As with the Fellenius method, a circular slip surface is assumed. Forces are summed in the vertical direction. The resulting equilibrium equation is combined with the Mohr-Coulomb equation and the definition of the factor of safety to determine the forces on the base of the slice. Finally, moments are summed about the center of the circular slip surface to obtain the following expression for the factor of safety:

$$\textbf{Equation 8-4: } F = \frac{\sum \dfrac{c'\Delta x + \left(W + P\cos\beta - u\Delta x \sec\alpha\right)\tan\phi'}{m_\alpha}}{\sum W \sin\alpha - \dfrac{\sum M_p}{R}}$$

Where Δx is the width of the slice and

$$\textbf{Equation 8-5: } m_\alpha = \cos\alpha + \frac{\sin\alpha \tan\phi'}{F}$$

Factors of safety calculated from Equation 8-4 satisfy the equilibrium of forces in the vertical direction and overall equilibrium of moments about the center of the circle. The unknowns and equations in this method are summarized in Table 8-3.

Table 8-3 Unknowns and Equations for the Simplified Bishop Method

Unknowns	Number of Unknowns for n Slices
Factor of safety (F)	1
Normal forces on bottom of slices (N)	n
TOTAL NUMBER OF UNKNOWNS	n + 1
Equations	**Number of Equations for n Slices**
Equilibrium of forces in the vertical direction, $\Sigma F_y = 0$	n
Equilibrium of moments of the entire soil mass	1
TOTAL NUMBER OF EQUILIBRIUM EQUATIONS	n + 1

Because m_α depends upon the factor of safety, same factor of safety appears on both side of Equation 8-4. This equation cannot be manipulated such that an explicit expression is obtained for the factor of safety. Thus, an iterative, trial and error procedure is used to solve for the factor of safety.

Horizontal equilibrium of forces is not satisfied by this method. Because of this, the suitabilityof this method for pseudo-static earthquake analyses where an additional horizontal force is applied is questionable. The method is also restricted to analyses with circular shear surfaces.

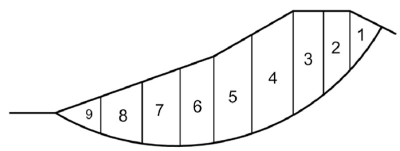
It has been shown that the factors of safety calculated by the Simplified Bishop Method compare well with factors of safety calculated using more rigorous methods, usually within 5 percent. Furthermore, the procedure is relatively simple compared with other methods. Computer solutions execute rapidly and hand calculations are not very time-consuming. The method is used widely throughout the world, and thus a strong record of experience exists. The method is acceptable for calculating factors of safety for circular slip surfaces. It is recommended that, where major structures are designed using this method, the final design should be checked using Spencer's Method.

When this method is used by a prepackaged computer routine, hand calculations or a spreadsheet program can verify results, or slope stability charts can be used. The Fellenius method can also be used as a check, although the factor of safety with this method is generally lower, especially if $\phi > 0$ and pore pressures are high. An example of this method in use is shown in Figure 8-2.

Figure 8-2 Example of Simplified Bishop Method[239]

The Simplified Bishop Method is only applicable to analyses with circular slip surfaces. The computations shown here have been performed using computer spreadsheet software. Detailed steps are presented below for a total stress analysis of a slope with no water and for an effective stress analysis of a slope with water, internal seepage, and external water loads.

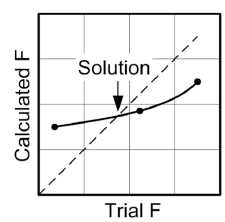
a. Slope without seepage or external water loads – total stress analyses. Computations for the Simplified Bishop Method for slopes, where the shear strength is expressed in terms of total stresses and where there are no external water loads, are illustrated in Figure F-1. As for all of the examples presented, slices are numbered beginning with the uppermost slice and proceeding toward the toe of the slope. Once a trial slip surface has been selected, and the soil mass is subdivided into slices, the following steps are used to compute a factor of safety:



a. Slope and trial slip surface with slices

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Slice	Horizontal Width (b)	Average Height (h_{avg})	Area (A)	Weight (W)	Base Inclination (α)	W sin α	Cohesion (c)	Friction Angle (ϕ)	cb + W tanϕ	m_α (Trial F_1)	Numerator (Col. 10 ÷ Col. 11)	m_α (Trial F_2)	Numerator (Col. 11 ÷ Col. 13)	m_α (Trial F_3)	Numerator (Col. 11 ÷ Col. 15)
1	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
2	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
3	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
4	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
5	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
6	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
7	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
8	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
9	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
					Σ W sinα =	...				Σ_1 =	...	Σ_2 =	...	Σ_3 =	...

b. Table used to compute factor of safety



c. Assumed and calculated trial values for factor of safety

Figure F-1. Simplified Bishop Method with no water - total stress analyses

(1) The width, b, average height, h_{avg}, and inclination, α, of the bottom of each slice are determined (Columns 2, 3, and 6 in Figure F-1b). The sign convention used throughout this appendix for the inclination, α, is illustrated in Figure F-2. The inclination is positive when the base of the slice is inclined in the same direction as the slope.

(2) The area, A, of each slice is calculated by multiplying the width of the slice by the average height, i.e., $A = b\,h_{avg}$ (Column 4 in Figure F-1b).

(3) The weight of each slice is calculated by multiplying the total unit weight of soil by the area of the slice, i.e., $W = \gamma A$. If the slice crosses zones having different unit weights, the slice is subdivided vertically into subareas, and the weights of the subareas are summed to compute the total slice weight (Column 5 in Figure F-1b).

(4) The quantity, $W \sin \alpha$, is computed for each slice, and these values are summed to obtain the term in the denominator of the equation for the factor of safety (Column 7 in Figure F-1b).

239 Adapted from EM 1110-2-1902, *Slope Stability*. Appendix F. U.S. Army Corps of Engineers, 31 October 2003.

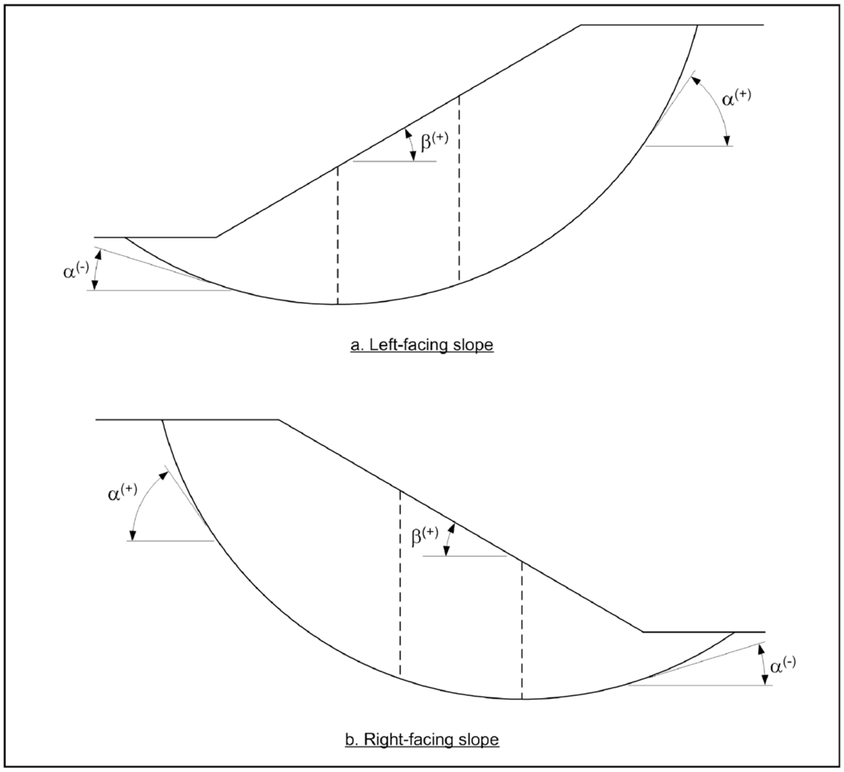


Figure F-2. Sign convention used for angles α and β

(5) The cohesion, c, and friction angle, ϕ, for each slice are entered in Columns 8 and 9 in Figure F-1b. The shear strength parameters are those for the soil at the bottom of the slice; they do not depend on the soils in the upper portions of the slice.

(6) The quantity $c \cdot b + W \tan(\phi)$ is computed for each slice (Column 10 in Figure F-1b).

(7) A trial value is assumed for the factor of safety and the quantity, m_α, is computed from the equation shown below (Column 11 in Figure F-1b):

$$m_\alpha = \cos\alpha + \frac{\sin\alpha \tan\phi'}{F} \qquad \text{(F-1)}$$

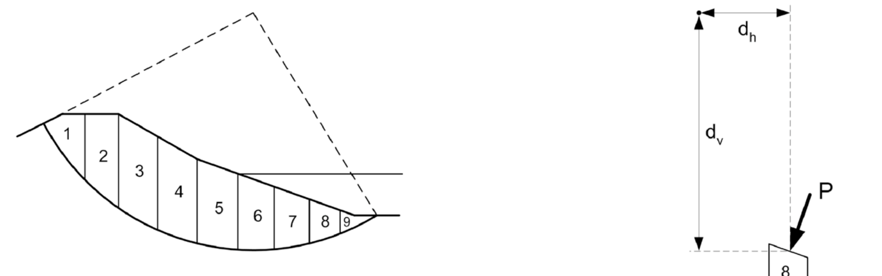
(8) The numerator in the expression for the factor of safety is computed by dividing the term cb + W tan(ϕ) by m_α for each slice and then summing the values for all slices (Column 12 in Figure F-1b).

(9) A new factor of safety is computed from the equation:

$$F = \frac{\sum\left[\frac{c \cdot b + W \cdot \tan\phi}{m_\alpha}\right]}{\sum W \sin\alpha} \qquad \text{(F-2)}$$

This corresponds to dividing the summation of Column 12 by the summation of Column 7 in Figure F-1b.

(10) Additional trial values are assumed for the factor of safety and Steps 7 through 9 are repeated (Columns 13 through 16 in Figure F-1b). For each trial value assumed for the factor of safety, the assumed value and the value computed for the factor of safety using Equation F-2 are plotted as shown in Figure F-1c. The chart in Figure F-1c serves as a guide for selecting additional trial values. Values are assumed and new values are calculated until the assumed and calculated values for the factor of safety are essentially the same, i.e., until the assumed and calculated values fall close to the broken 45-degree line shown in Figure F-1c.

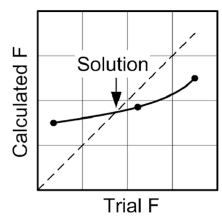


a. Slope and trial slip surface with slices

b. Surface load and moment arms for slice

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
							Surface Loads																	
Slice	Horizontal Width (β)	Average Height (h_{avg})	Area (A)	Weight (W)	Base Inclination (α)	W sin α	Height of surface water (h_s)	Avg. Surface Press. ($p_{surface}$)	Surface Inclination (β)	Surface Load (P)	Horizontal moment arm (d_h)	Vertical moment arm (d_v)	Moment (M_P)	Piezometric height (h_p)	Pore water pressure (u)	Cohesion c'	Friction angle φ'	c'b + (W + P cosβ - ub) tanφ'	m_α (Trial F_1)	Numerator (Col. 19 ÷ Col. 20)	m_α (Trial F_2)	Numerator (Col. 17 ÷ Col. 20)	m_α (Trial F_3)	Numerator (Col. 17 ÷ Col. 22)
1	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
2	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
3	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
4	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
5	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
6	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
7	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
8	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
9	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
					Σ W sinα =	...						Σ M_P =	...						Σ_1 =	...	Σ_2 =	...	Σ_3 =	...

c. Table used to compute factor of safety



d. Assumed and calculated trial values for factor of safety

Figure F-3. Simplified Bishop Method with water - effective stress analyses

(8) The length of the top of the slice is multiplied by the average surface pressure, $p_{surface}$, to compute the external water force, P, on the top of the slice (Column 11 in Figure F-3c). The force P is equal to $p_{surface} \cdot b / \cos(\beta)$.

(9) The horizontal and vertical distances, d_h and d_v, respectively, between the center of the circle and the points on the top center of each slice are determined (Columns 12 and 13 in Figure F-3c). Positive values for these distances are illustrated in Figure F-3b. Loads acting at points located upslope of the center of the circle (to the left of the center in the case of the right-facing slope shown in Figure F-3) represent negative values for the distance, d_h.

(10) The moment, M_P, the result of external water loads is computed from the following (Column 14 in Figure F-3c):

$$M_P = P\cos\beta\, d_h + P\sin\beta\, d_v \tag{F-3}$$

The moment is considered positive when it acts opposite to the direction of the driving moment produced by the weight of the slide mass, i.e., positive moments tend to make the slope more stable. Positive moments are clockwise for a right-facing slope like the one shown in Figure F-3.

(11) The piezometric height, h_p, at the center of the base of each slice is determined (Column 15 in Figure F-3c). The piezometric height represents the pressure head for pore water pressures on the base of the slice.

(12) The piezometric height is multiplied by the unit weight of water to compute the pore water pressure, u (Column 16 in Figure F-3c). For complex seepage conditions, or where a seepage analysis has been conducted using numerical methods, it may be more convenient to determine the pore water pressure directly, rather than evaluating the piezometric head and converting to pore pressure. In such cases Step 11 is omitted, and the pore water pressures are entered in Column 16.

(13) The cohesion, c', and friction angle, φ', for each slice are entered in Columns 17 and 18 in Figure F-3c. The shear strength parameters are those for the soil at the bottom of the slice; they do not depend on the soils in the upper portions of the slice.

(14) The following quantity is computed for each slice (Column 19 in Figure F-3c):

$$c'b + (W + P\cos\beta - ub)\tan\phi' \tag{F-4}$$

(15) A trial factor of safety, F_1, is assumed and the quantity, m_α, is computed from the equation shown below (Column 20 in Figure F-3c):

$$m_\alpha = \cos\alpha + \frac{\tan\phi'\sin\alpha}{F_1} \tag{F-5}$$

(16) The numerator in the equation used to compute the factor of safety is calculated by dividing the term $c'b + (W + P\cos\beta - ub)\tan\phi'$ by m_α for each slice and then summing the values for all slices (Column 21 in Figure F-3c).

(17) A new value is computed for the factor of safety using the following equation:

$$F = \frac{\sum\left[\frac{c'b + (W + P\cos\beta - ub)\tan\phi'}{m_\alpha}\right]}{\sum W\sin\alpha - \frac{1}{R}\sum M_P} \tag{F-6}$$

where R is the radius of the circle.

The summations computed in Columns 7, 14, and 21 of the table in Figure F-3c are used to compute the new value for the factor of safety.

(18) Additional trial values are assumed for the factor of safety and steps 14 through 16 are repeated (Columns 22 through 25 in Figure F-3c). For each trial value assumed for the factor of safety, the assumed and calculated values of the factor of safety are plotted as shown in Figure F-3d, to provide a guide for selecting additional trial values. Values are assumed and new values are calculated until the assumed and calculated values for the factor of safety are essentially equal, i.e., until the assumed and calculated values fall close enough to the broken 45-degree line shown in Figure F-3d.

8.3.1.4. Critical Failure Surface

Both the Fellenius and Bishop procedures – along with any other procedures done by the method of slices – require that a large number of assumed failure surfaces be checked in order to find the most critical one – the surface with the lowest factor of safety. This is obviously a tedious and time-consuming job if done by hand.

This is where the computer becomes such a valuable design tool. The stability analysis is easily adapted to computer solution. A grid of possible circle centers is defined, and a range of radius values established for each. The computer can be directed to print out all the safety factors or just the minimum one (and its radius) for each circle centre. A plot of the minimum safety factor for each circle centre in the form of contours can be used to define the location of the most critical circle and the minimum safety factor.

8.3.2. Limit Analysis

This method considers yield criteria and the stress-strain relationship. It is based on lower bound and upper bound theorems for bodies of elastic - perfectly plastic materials.[240]

8.3.3. Finite Element Method

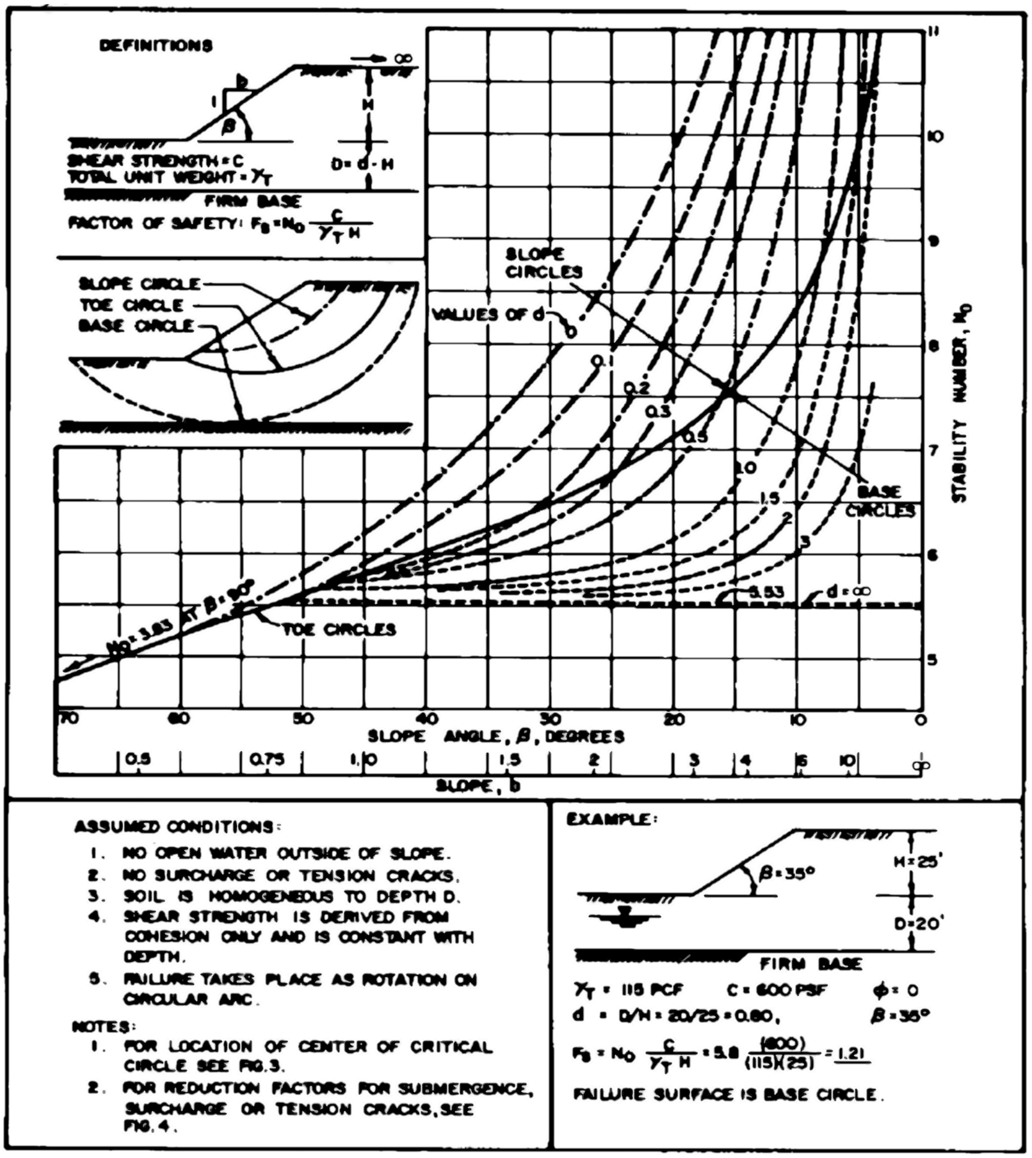
This method is extensively used in more complex problems of slope stability and where earthquake and vibrations are part of total loading system. This procedure accounts for deformation and is useful where significantly different material properties are encountered.

[240] Fang, H.Y., Stability of Earth Slopes, Foundation Engineering Handbook, Winterkorn and Fang, ed., Chapter 10, Van Nostrand Reinhold Co., New York, 1975.

8.3.4. Slope Stability Charts

- Rotational Failure in Cohesive Soils ($\phi = 0$)
  - For slopes in cohesive soils having approximately constant strength with depth use Figure 8-3 to determine the factor of safety.
  - For slope in cohesive soil with more than one soil layer, determine centers of potentially critical circles from Figure 8-4. Use the appropriate shear strength of sections of the arc in each stratum. Use the following guide for positioning the circle.
  - If the lower soil layer is weaker, a circle tangent to the base of the weaker layer will be critical.
  - If the lower soil layer is stronger, two circles, one tangent to the base of the upper weaker layer and the other tangent to the base of the lower stronger layer, should be investigated.
  - With surcharge, tension cracks, or submergence of slope, apply corrections of Figure 8-5 to determine safety factor.
- Embankments on Soft Clay. See Figure 8-6 for approximate analysis of embankment with stabilizing berms on foundations of constant strength. Determine the probable form of failure from relationship of berm and embankment widths and foundation thickness in top left panel of Figure 8-6.

Figure 8-3 Stability Analysis in Cohesive Soils, Undrained Conditions, i.e., Assumed ϕ = 0[241]



[241] Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics Series No. 46, Harvard University, Cambridge, MA.

Figure 8-4 Centre of Critical Circle, Slope in Cohesive Soils[241]

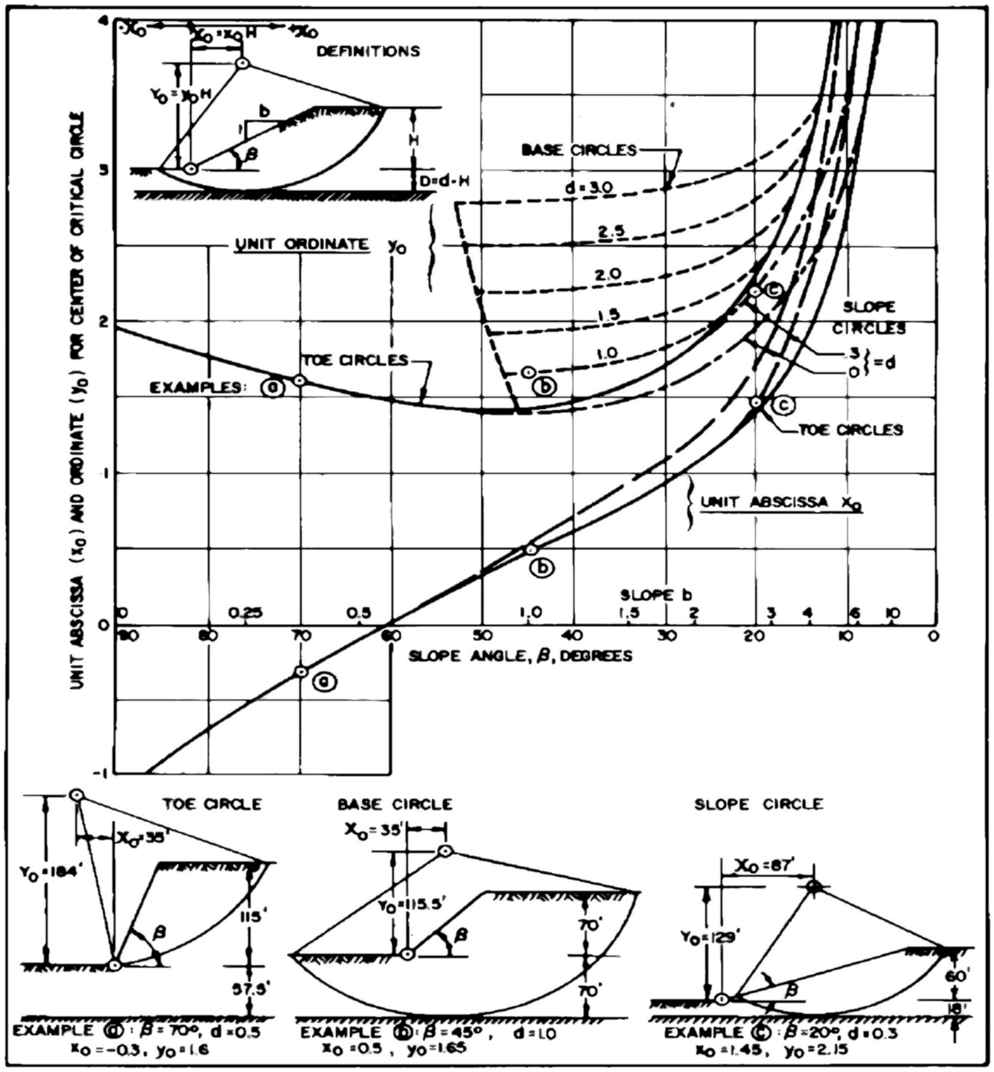


Figure 8-5 Influence of Surcharge, Submergence, and Tension Cracks on Stability

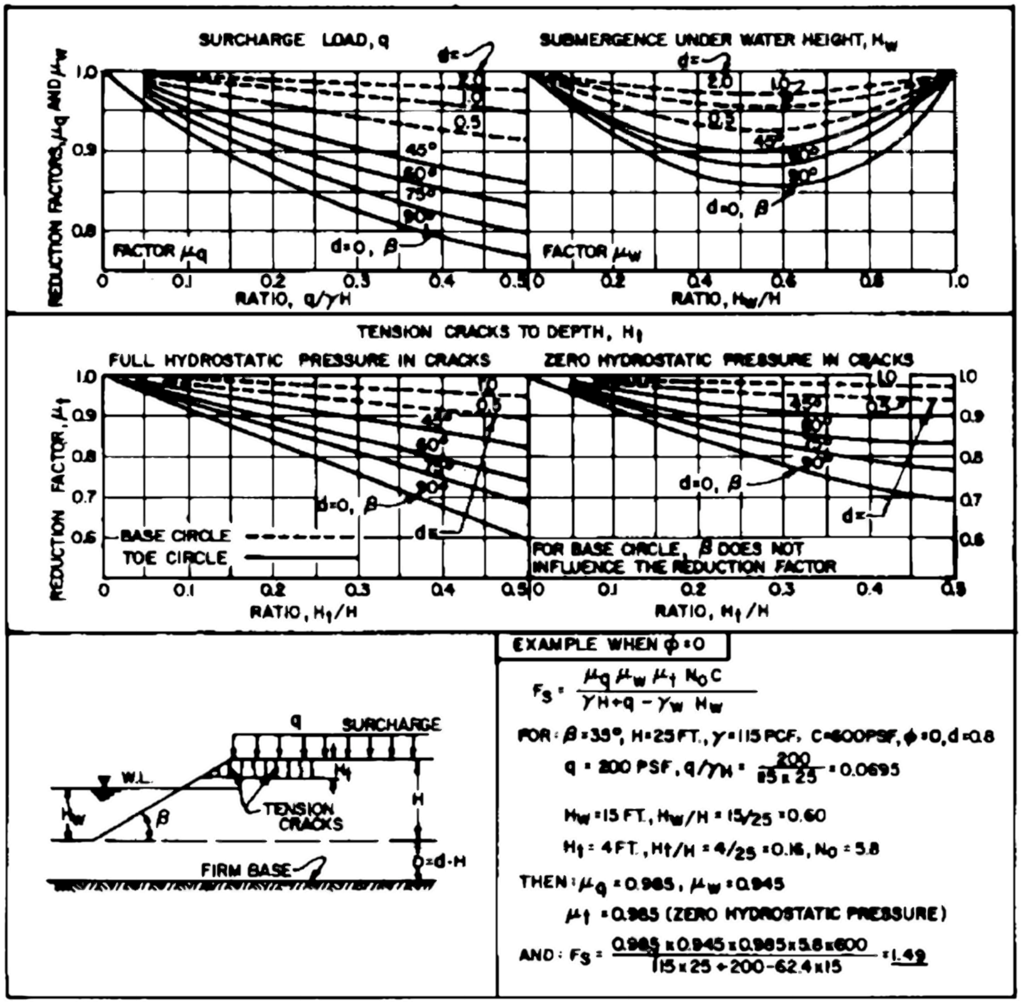
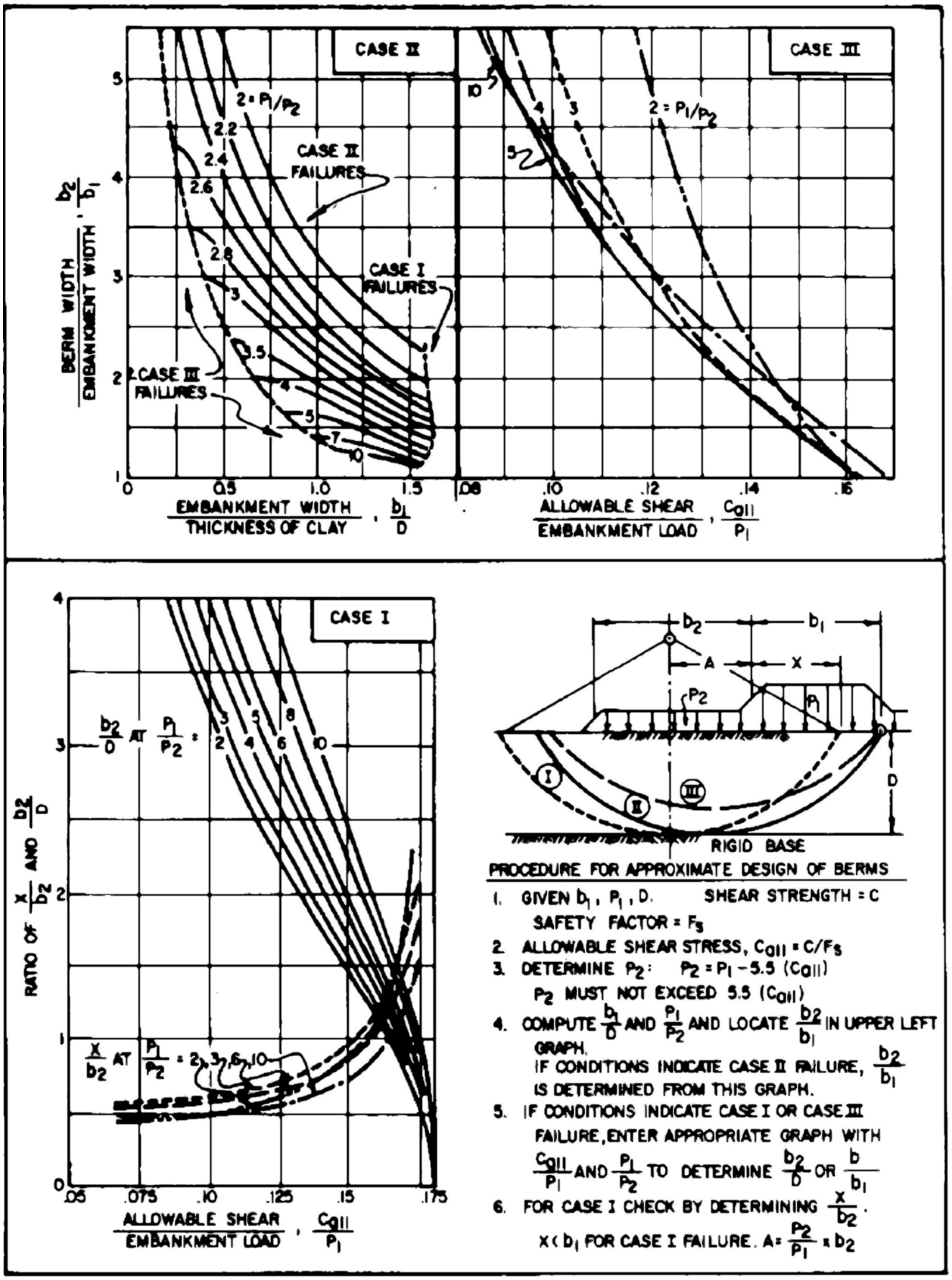


Figure 8-6 Design of Berms for Embankments on Soft Clays[242]



[242] Jakobsen, R.E., The Design of Embankments on Soft Clays, Geotechnique, 1948.

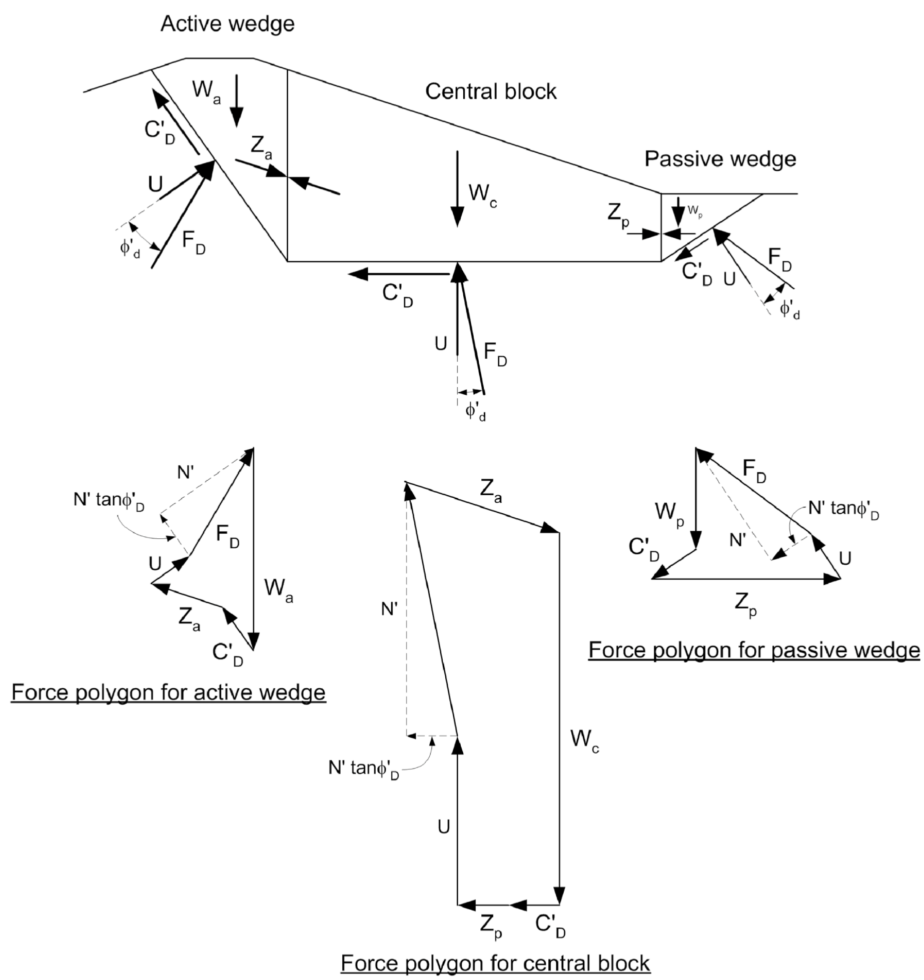
8.3.5. Translational (Sliding Block) Failure

A "sliding block" type failure can occur (1) where the foundation soil contains thin seams of weak clay or organic soils, (2) where a shallow layer of weak soil exists at the ground surface and is underlain by firm soil, and (3) where the foundation soil contains thin sand or silt lenses sandwiched between more impermeable soil. The weak layer or lens provides a plane of weakness along which sliding can occur. In the case of sand or silt lenses trapped between impervious soil, the mechanism that can cause sliding is as follows: As the fill load is placed, the water pressure is increased in the sand or silt lens. Since the water cannot escape due to the impermeable soil above and below, the sand or silt loses frictional strength because of the intergranular effective stress between soil grains being decreased due to the water pressure.

When sliding occurs, an active wedge type failure occurs through the fill (similar to the active wedge that forms behind a retaining wall), and a passive wedge type failure occurs below the fill toe as soil in the toe area is pushed up out of the way. The sliding mass moves essentially as a block, thus the term "sliding block."

A sliding block analysis to estimate factor of safety against sliding is fairly simple and straightforward and can be easily and quickly performed by hand. For the analysis, the potential sliding block is divided into three parts; (1) An active wedge at the head of the slide, (2) A central block, and (3) A passive wedge at the toe, as shown in Figure 8-7.

Figure 8-7 Layout of Simple Sliding Block Analysis



For the problem illustrated above, summing forces would compute the factor of safety horizontally, to give:

$$\textbf{Equation 8-6: } F = \frac{\vec{Z}_p + \vec{C}'_D}{\vec{Z}_a}$$

Where

- Z_a = Active Force (Driving)
- Z_p = Passive Force (Resisting)
- C'_D = Resisting Force due to cohesion of clay

IMPORTANT: Keep in mind that these are *vector* forces; their orientation must be taken in to account, especially in the case of Z_a. Only the horizontal component is to be used to compute the factor of safety!

For convenience of computation of 1 unit thick slice of embankment is assumed.

Several trial locations of the active and passive wedges must be checked to determine the minimum factor of safety. Note that since wedge type failures occur at the head and toe of the slide, similar to what occurs behind retaining walls, the active and passive forces are taken as acting against vertical planes which are treated as "imaginary" retaining walls, and the active and passive forces are computed the same as for retaining wall problems.

These are two important design tips that should be kept in mind when performing a sliding block analysis.

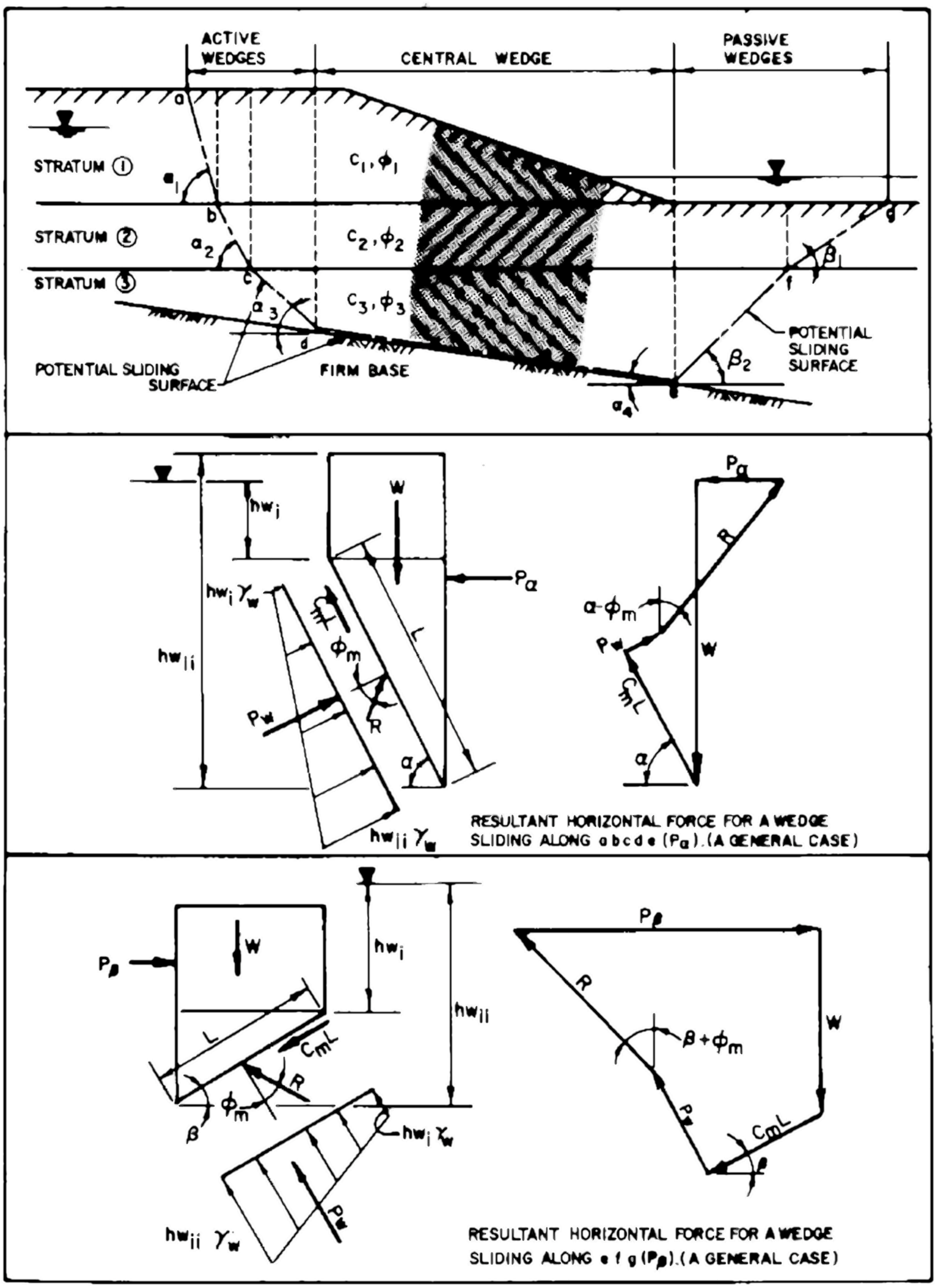
First, be aware that if the active or passive wedge passes through more than one soil type with different soil strengths or soil weights, then the active or passive pressure changes as you go from one soil layer into the next (due to change in either the soil weight and/or the earth pressure coefficient K_a or K_p). The easiest way to handle this is to first compute the active or passive pressure diagram, and then compute the active or passive force from the area of the pressure diagram.

Second, when computing the active or passive pressure, remember to use buoyant (effective) soil unit weight below the water table.

8.3.5.1. Computation of Forces - Complicated Sliding Block Analysis

For more difficult problems in translational or sliding block failures, analyze the stability of the potentially translating mass as shown in Figure 8-8 by comparing the destabilizing forces of the active pressure wedge with the stabilizing force of the passive wedge at the toe plus the shear strength along the base of the central soil mass. The procedure given shows how the effects of water pressure, cohesive, friction, and a sloping failure plane can be taken into account in the analysis. This analysis procedure can be used either to estimate factor of safety for assumed failure surfaces in design or to "back analyze" sliding block type landslides problems. See Figure 8-8 for an example of translational failure analysis in soil and Figure 8-10 for an example of translational failure in rock.

Figure 8-8 Stability Analysis of Translation (Sliding Block) Failure



DEFINITION OF TERMS

P_α = RESULTANT HORIZONTAL FORCE FOR AN ACTIVE OR CENTRAL WEDGE ALONG POTENTIAL SLIDING SURFACE a b c d e.

P_β = RESULTANT HORIZONTAL FORCE FOR A PASSIVE WEDGE ALONG POTENTIAL SLIDING SURFACE e f g.

W = TOTAL WEIGHT OF SOIL AND WATER IN WEDGE ABOVE POTENTIAL SLIDING SURFACE.

R = RESULT OF NORMAL AND TANGENTIAL FORCES ON POTENTIAL SLIDING SURFACE CONSIDERING FRICTION ANGLE OF MATERIAL.

P_w = RESULTANT FORCE DUE TO PORE WATER PRESSURE ON POTENTIAL SLIDING SURFACE CALCULATED AS:

$$P_w = \left[\frac{hw_i + hw_{ii}}{2}\right](L)(\gamma_w)$$

ϕ = FRICTION ANGLE OF LAYER ALONG POTENTIAL SLIDING SURFACE.

C = COHESION OF LAYER ALONG POTENTIAL SLIDING SURFACE.

L = LENGTH OF POTENTIAL SLIDING SURFACE ACROSS WEDGE.

h_w = DEPTH BELOW PHREATIC SURFACE AT BOUNDARY OF WEDGE.

γ_w = UNIT WEIGHT OF WATER.

PROCEDURES

1.) EXCEPT FOR CENTRAL WEDGE WHERE α IS DICTATED BY STRATIGRAPHY USE $\alpha = 45° + \frac{\phi}{2}$, $\beta = 45° - \frac{\phi}{2}$ FOR ESTIMATING FAILURE SURFACE.

2.) SOLVE FOR P_α AND P_β FOR EACH WEDGE IN TERMS OF THE SAFETY FACTOR (F_s) USING THE EQUATIONS SHOWN BELOW. THE SAFETY FACTOR IS APPLIED TO SOIL STRENGTH VALUES (TAN ϕ AND C).

MOBILIZED STRENGTH PARAMETERS ARE THEREFORE CONSIDERED AS $\phi_m = \tan^{-1}\left(\frac{\tan\phi}{F_s}\right)$ AND $C_m = \frac{C}{F_s}$.

$$P_\alpha = \left[W - C_m L \sin\alpha - P_w \cos\alpha\right] \tan\left[\alpha - \phi_m\right] - \left[C_m L \cos\alpha - P_w \sin\alpha\right]$$

$$P_\beta = \left[W + C_m L \sin\beta - P_w \cos\beta\right]\left[\tan(\beta + \phi_m)\right] + \left[C_m L \cos\beta + P_w \sin\beta\right]$$

IN WHICH THE FOLLOWING EXPANSIONS ARE TO BE USED:

$$\tan(\alpha - \phi_m) = \frac{\tan\alpha - \frac{\tan\phi}{F_s}}{1 + \tan\alpha \frac{\tan\phi}{F_s}} \qquad \tan(\beta + \phi_m) = \frac{\tan\beta + \frac{\tan\phi}{F_s}}{1 - \tan\beta \frac{\tan\phi}{F_s}}$$

3.) FOR EQUILIBRIUM $\Sigma P_\alpha = \Sigma P_\beta$. SUM P_α AND P_β FORCES IN TERMS OF F_s, SELECT TRIAL F_s, CALCULATE ΣP_α AND ΣP_β. IF $\Sigma P_\alpha \neq \Sigma P_\beta$, REPEAT. PLOT P_α AND P_β VS. F_s WITH SUFFICIENT TRIALS TO ESTABLISH THE POINT OF INTERSECTION (i.e., $\Sigma P_\alpha = \Sigma P_\beta$), WHICH IS THE CORRECT SAFETY FACTOR.

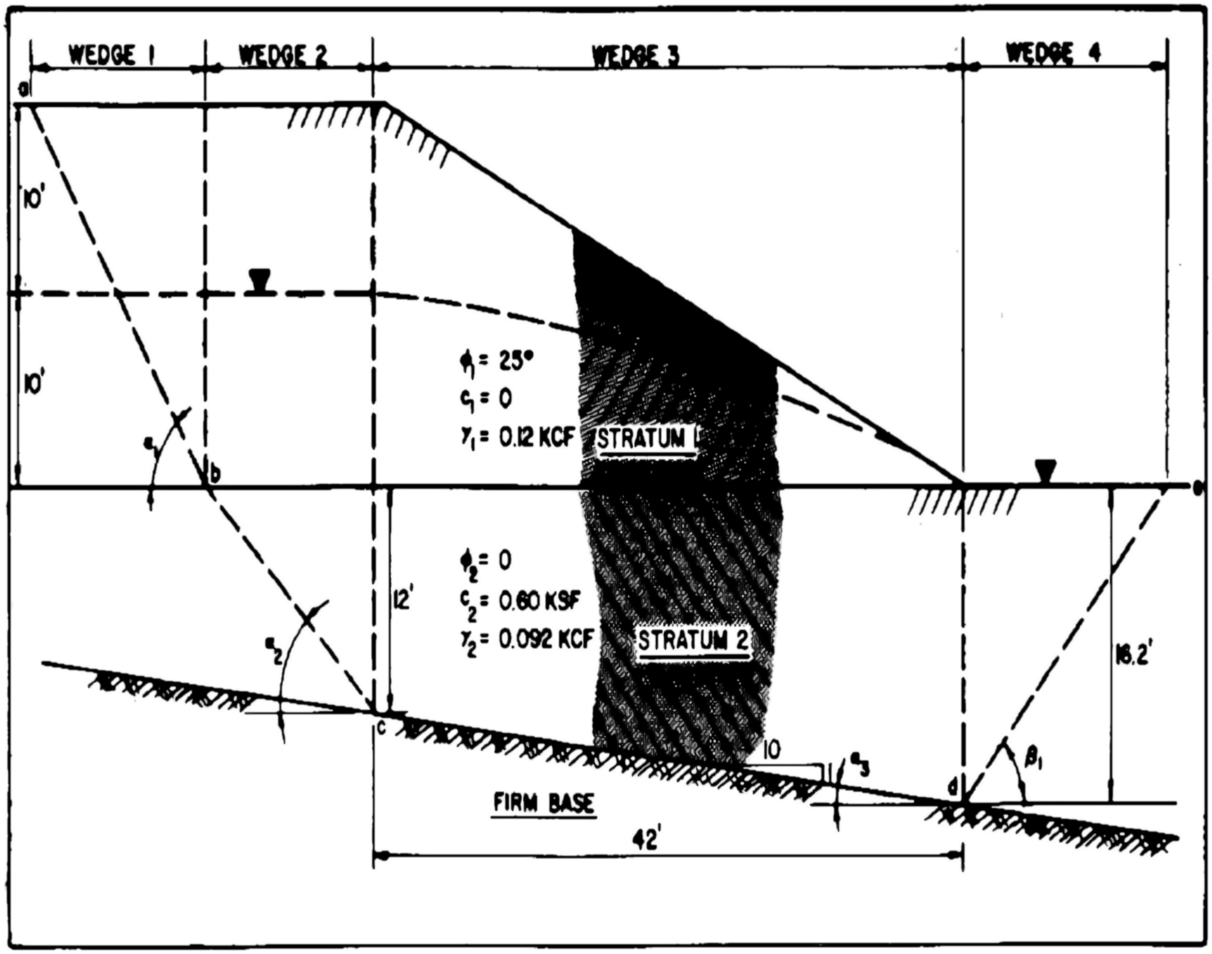
4.) DEPENDING ON STRATIGRAPHY AND SOIL STRENGTH, THE CENTER WEDGE MAY ACT TO MAINTAIN OR UPSET EQUILIBRIUM.

5.) NOTE THAT FOR $\phi = 0$, ABOVE EQUATIONS REDUCE TO:

$$P_\alpha = W \tan\alpha - \frac{C_m L}{\cos\alpha}, \quad P_\beta = W \tan\beta + \frac{C_m L}{\cos\beta}$$

6.) THE SAFETY FACTOR FOR SEVERAL POTENTIAL SLIDING SURFACES MAY HAVE TO BE COMPUTED IN ORDER TO FIND THE MINIMUM SAFETY FACTOR FOR THE GIVEN STRATIGRAPHY.

Figure 8-9 Example of Stability Analysis of Translation (Sliding Block) Failure



FORCES P_a

WEDGE 1: $\phi = 25°$, $C = 0$, $\gamma = 0.12$ KCF (SLIDING SURFACE ab)

$\alpha_1 = 45 + \phi_1/2 = 57.5°$

$$W = \frac{20}{2} \times 20 \tan 32.5° \times 0.12 = 15.29 \text{ KIPS}$$

$$P_w = \left(\frac{0+10}{2}\right)(0.062) \times \left(\frac{10}{\sin 57.5°}\right) = 3.68 \text{ KIPS}$$

$$P_{a1} = (W - P_w \cos \alpha_1)\frac{\left(\tan \alpha_1 - \frac{\tan \phi_1}{F_S}\right)}{\left(1 + \tan \alpha_1 \frac{\tan \phi_1}{F_S}\right)} + P_w \sin \alpha_1$$

$$= (15.29 - 1.98)\frac{\left(1.57 - \frac{0.47}{F_S}\right)}{\left(1 + \frac{0.73}{F_S}\right)} + 3.10 = \left(\frac{20.90F_S - 6.26}{F_S + 0.73}\right) + 3.10$$

WEDGE 2: $\phi = 0$, $C = 0.60$ KSF, $\gamma = 0.092$ KCF (SLIDING SURFACE bc)

$\alpha_2 = 45°$

$$W = 12 \times 10 \times 0.12 + 12 \times 10 \times 0.12 + \frac{12}{2} \times 12 \times 0.092 = 35.42 \text{ KIPS}$$

$$P_{a2} = W \tan \alpha_2 - \frac{\frac{CL}{F_S}}{\cos \alpha_2} \quad (\text{FOR } \phi = 0)$$

$$= 35.42 - \frac{\left(\frac{0.60}{F_S} \times \frac{12}{.707}\right)}{(.707)} = 35.42 - \frac{14.40}{F_S}$$

WEDGE 3: $\phi = 0$, $C = 0.60$ KSF, $\gamma = 0.092$ KCF (SLIDING SURFACE cd)

$\alpha_3 = \tan^{-1} 0.1 = 5.7°$

$$W = \frac{20}{2} \times 42 \times 0.12 + \frac{12 + 16.2}{2} \times 42 \times 0.092 = 104.88 \text{ KIPS}$$

$$P_{a3} = W \tan \alpha_3 - \frac{\frac{CL}{F_S}}{\cos \alpha} \quad (\text{FOR } \phi = 0)$$

$$= [104.9 \times 0.10] - \left[\frac{\left(\frac{0.60}{F_S} \times \frac{42}{0.99}\right)}{0.99}\right] = 10.49 - \frac{25.71}{F_S}$$

$$\Sigma P_a = \frac{20.90 F_S - 6.26}{F_S + 0.73} + 49.01 - \frac{40.11}{F_S}$$

FORCES P_β

WEDGE 4: $\phi = 0$, $C = 0.60$ KSF, $\gamma = 0.092$ KCF (SLIDING SURFACE de)

$\beta_1 = 45°$

$$W = \frac{16.2}{2} \times 16.2 \times 0.092 = 12.07 \text{ KIPS}$$

$$P_{\beta 1} = W \tan \beta + \frac{\frac{CL}{F_S}}{\cos \beta} \quad (\text{FOR } \phi = 0)$$

$$= 12.07 + \left[\frac{\frac{0.60}{F_S} \times \frac{16.20}{0.707}}{0.707}\right] = 12.07 + \frac{19.44}{F_S}$$

$$\Sigma P_\beta = 12.07 + \frac{19.44}{F_S}$$

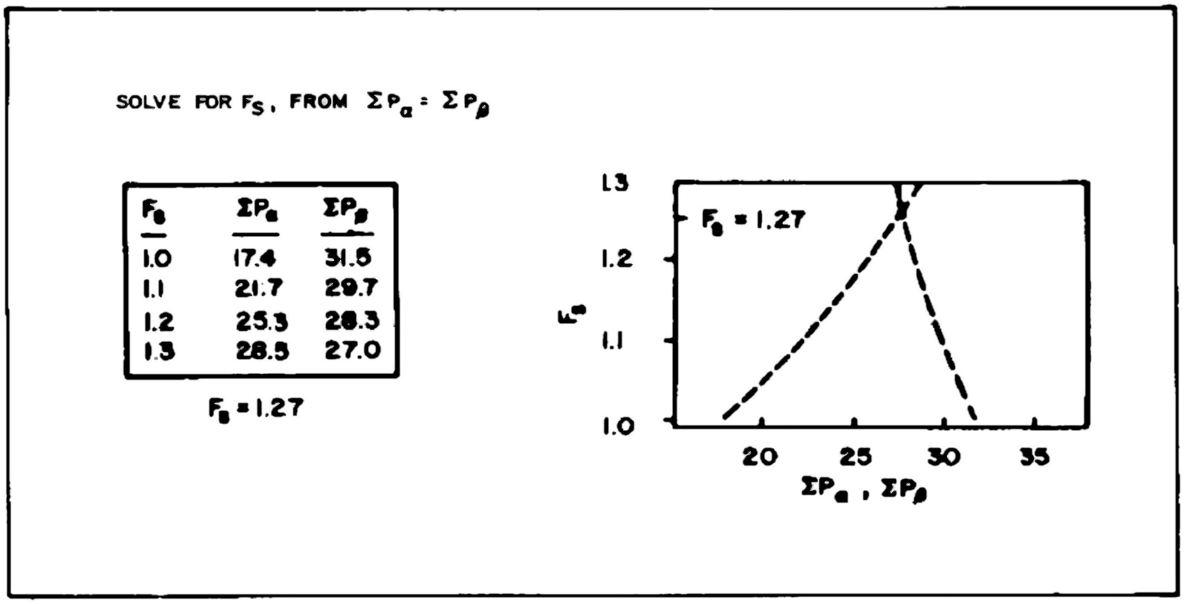
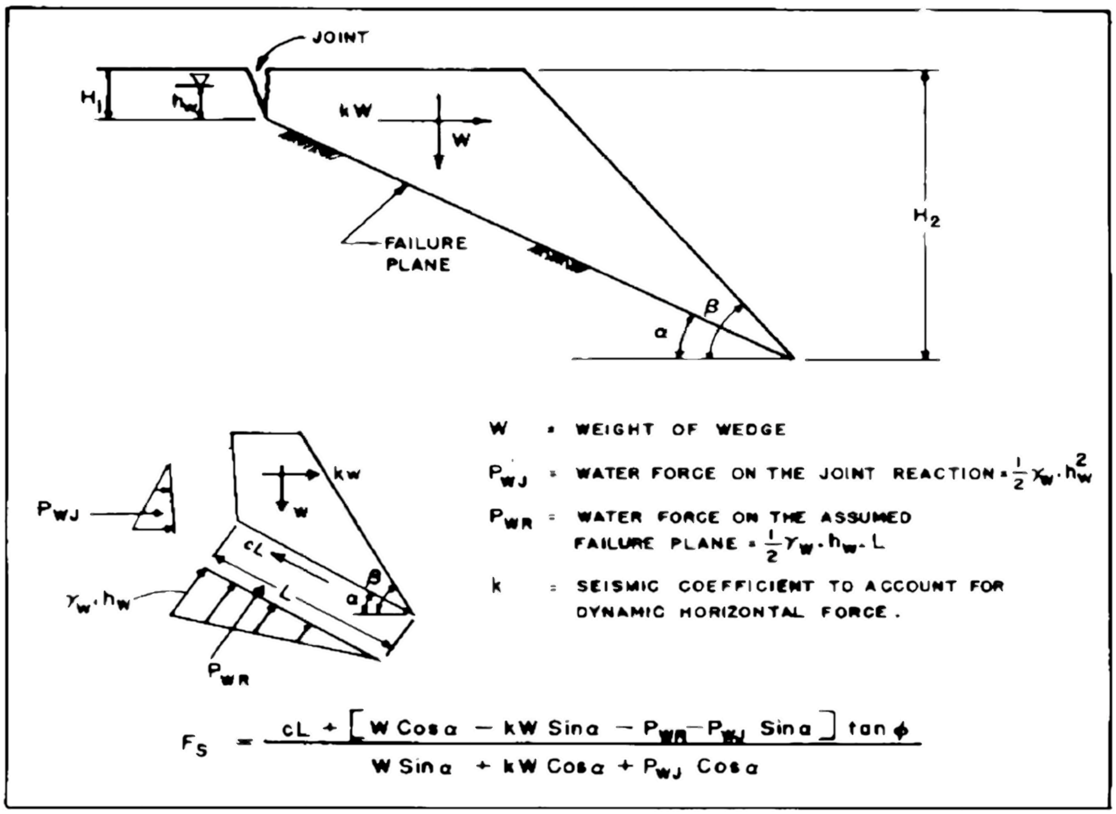


Figure 8-10 Stability of Rock Slope



Jointed rocks involve multiple planes of weakness. This type of problem cannot be analyzed by two-dimensional cross-sections.[243]

[243] Von Thun, J.L., The Practical and Realistic Solution of Rock Slope Stability Problems, Design Methods in Rock Mechanics, Proceedings of Sixteenth Symposium on Rock Mechanics, ASCE, September 22-24, 1977.

8.3.6. Required Safety Factors

The following values should be provided for reasonable assurance of stability:

- Safety factor no less than 1.5 for permanent or sustained loading conditions.
- For foundations of structures, a safety factor no less than 2.0 is desirable to limit critical movements at foundation edge.
- For temporary loading conditions or where stability reaches a minimum during construction, safety factors may be reduced to 1.3 or 1.25 if controls are maintained on load application.
- For transient loads, such as earthquake, safety factors as low as 1.2 or 1.15 may be tolerated.

8.4. Effects of Soil Parameters and Groundwater on Stability

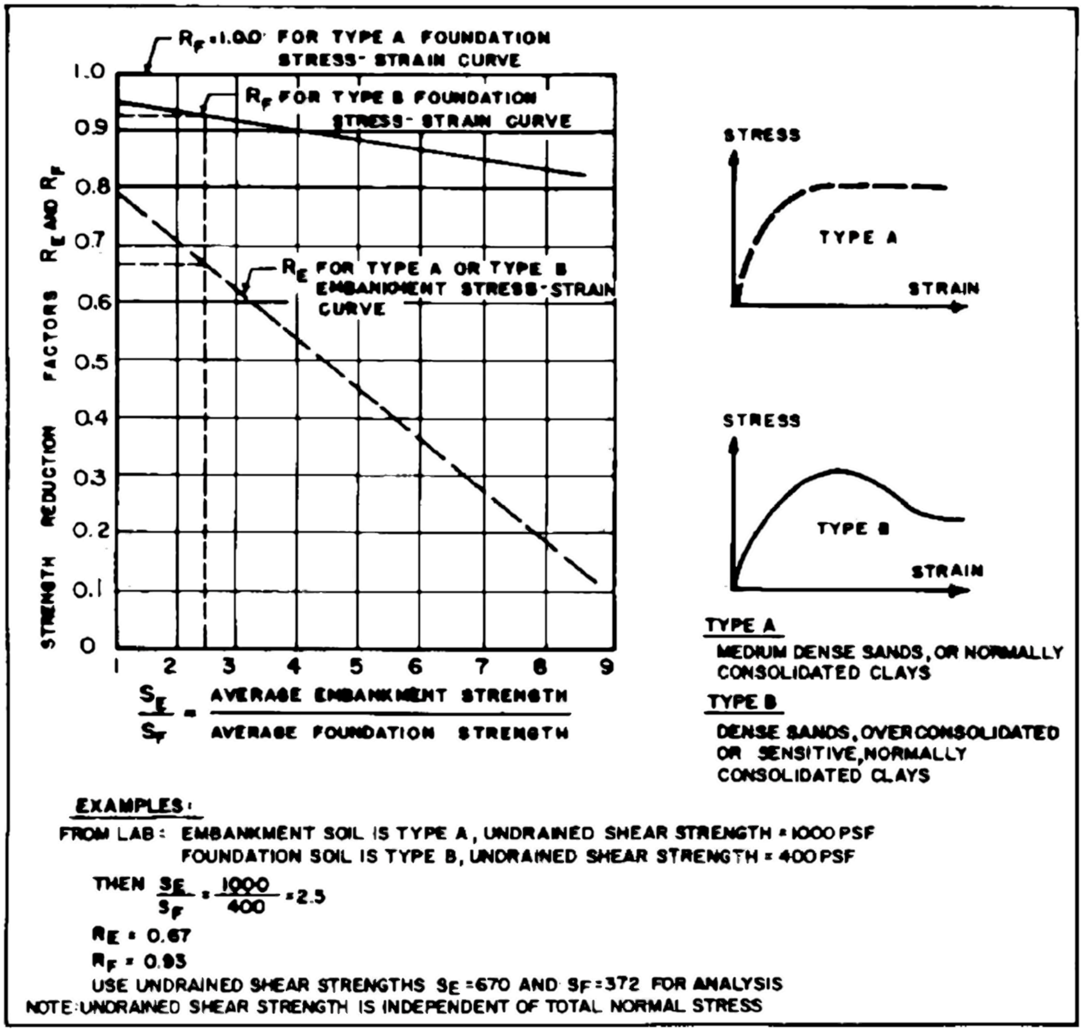
The choice of soil parameters and the methods of analyses are dictated by the types of materials encountered, the anticipated groundwater conditions, the time frame of construction, and climatic conditions. Soil strength parameters are selected either on the basis of total stress, ignoring the effect of the pore water pressure, or on the basis of effective stress where the analysis of the slope requires that the pore water pressures be treated separately.

8.4.1. Total vs. Effective Stress Analysis

The choice between total stress and effective stress parameters is governed by the drainage conditions that occur within the sliding mass and along its boundaries. Drainage is dependent upon soil permeability, boundary conditions, and time.

- Total Stress Analysis. Where effective drainage cannot occur during shear, use the undrained shear strength parameters such as vane shear, unconfined compression, and unconsolidated undrained (UU or Q) triaxial compression tests. Field vane shear and cone penetration tests may be used. Assume ϕ = 0. Examples where a total stress analysis are applicable include:
    - Analysis of cut slopes of normally consolidated or slightly preconsolidated clays. In this case little dissipation of pore water pressure occurs prior to critical stability conditions.
    - Analysis of embankments on a soft clay stratum. This is a special case as differences in the stress-strain characteristics of the embankment and the foundation may lead to progressive failure. The undrained strength of both the foundation soil and the embankment soil should be reduced in accordance with the strength reduction factors R_E and R_F in Figure 8-11.
    - Rapid drawdown of water level providing insufficient time for drainage. Use the undrained strength corresponding to the overburden condition within the structure prior to drawdown.
    - End-of-construction condition for fills built of cohesive soils. Use the undrained strength of samples compacted to field density and at water content representative of the embankment.

Figure 8-11 Correction Factors R_E and R_F to Account for Progressive Failure in Embankments on Soft Clay Foundations[244]



- Effective Stress Analysis. The effective shear strength parameters (c', ϕ') should be used for the following cases:
  - Long-term stability of clay fills. Use steady state seepage pressures where applicable.
  - Short-term or end-of-construction condition for fills built of free draining sand and gravel. Friction angle is usually approximated by correlation for this case.
  - Rapid drawdown condition of slopes in pervious, relatively incompressible, coarse-grained soils. Use pore pressures corresponding to new lower water level with steady state flow.
  - Long-term stability of cuts in saturated clays. Use steady state seepage pressures where applicable.
  - Cases of partial dissipation of pore pressure in the field. Here, pore water pressures must be measured by piezometers or estimated from consolidation data.

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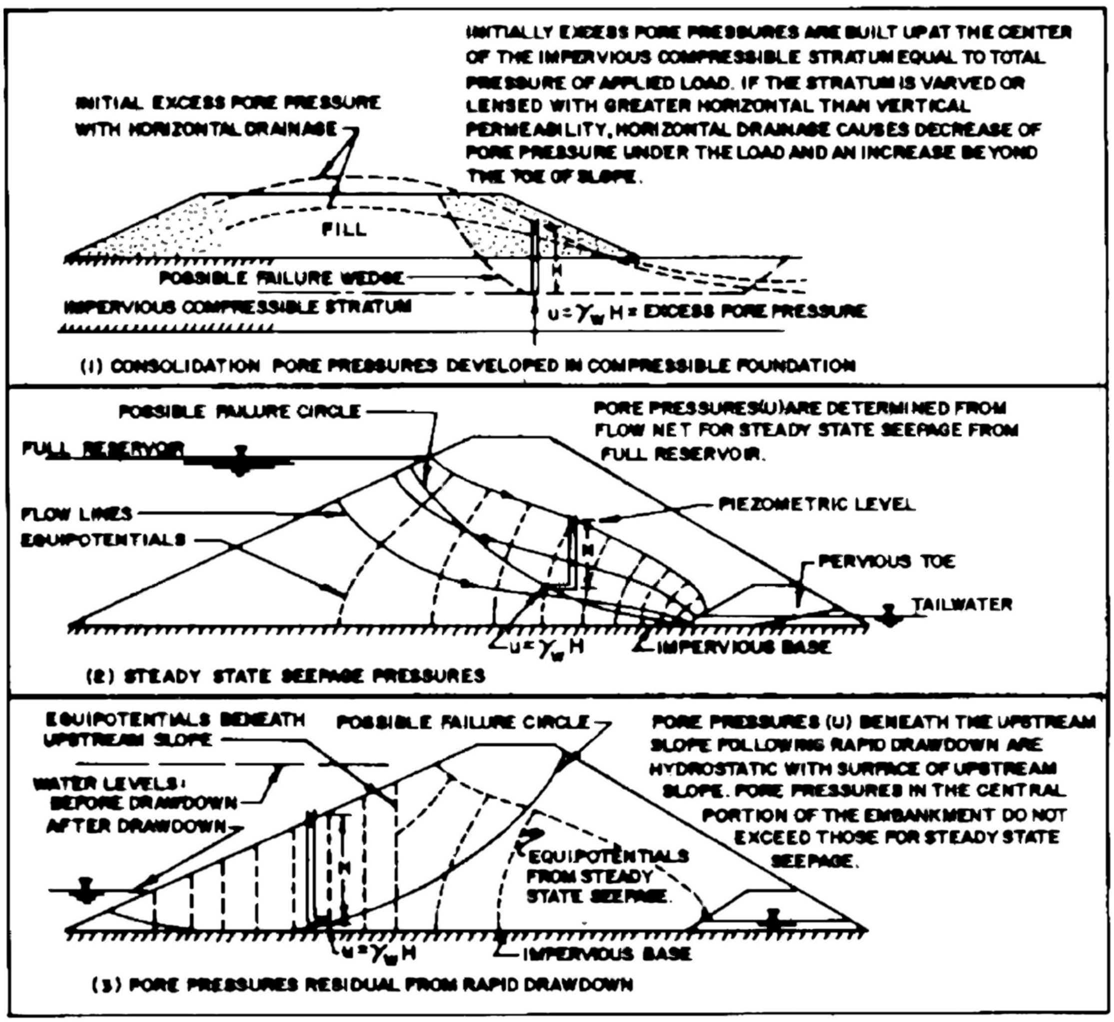
[244] Duncan, J.M. and Buchignani, A.L., An Engineering Manual for Slope Stability Studies, Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, March, 1975.

8.4.2. Effect of Groundwater and Excess Pore Pressure

Subsurface water movement and associated seepage pressures are the most frequent cause of slope instability. See Table 8-1 for illustrations of the effects of water on slope stability.

- Seepage Pressures. Subsurface water seeping toward the face or toe of a slope produces destabilizing forces that can be evaluated by flow net construction. The piezometric heads, which occur along the assumed failure surface, produce outward forces that must be considered in the stability analysis. See Table 8-4 and the example of Figure 8-2.

Table 8-4 Pore Pressure Conditions for Stability Analysis of Homogeneous Embankment



- Construction Pore Pressures. When compressible fill materials are used in embankment construction, excess pore pressure may develop and must be considered in the stability analysis. Normally, field piezometric measurements are required to evaluate the condition.
- Excess Pore Pressures in Embankment Foundations. Where embankments are constructed over compressible soils, the foundation pore pressures must be considered in the stability analysis. See top panel of Table 8-4.
- Artesian Pressures. Artesian pressures beneath slopes can have serious effects on the stability. Should such pressures be found to exist, they must be used to determine effective stresses and unit weights, and the slope and foundation stability should be evaluated by effective stress methods.

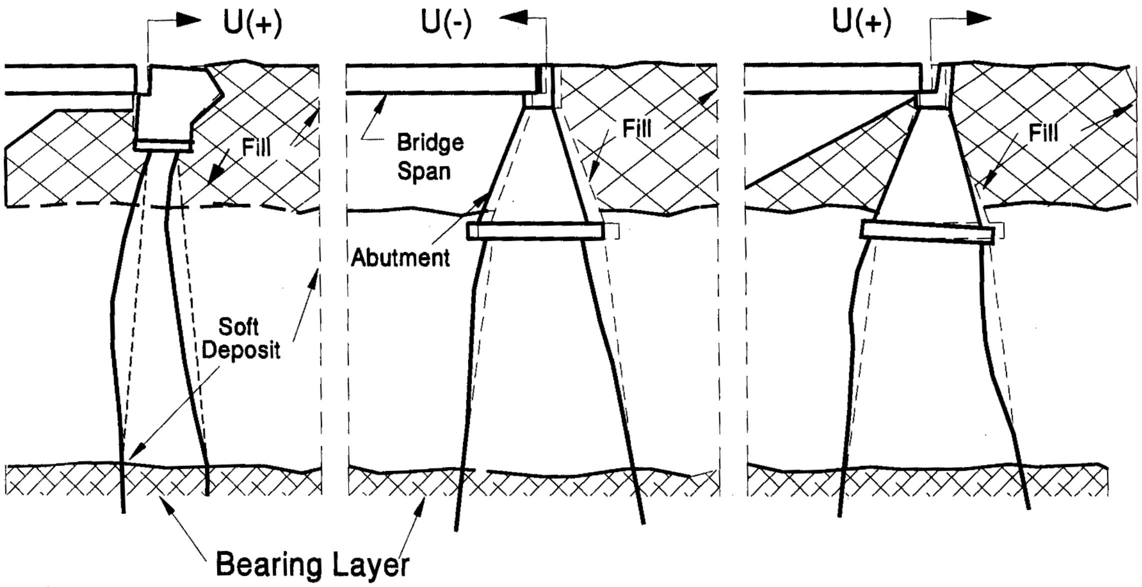
8.4.3. Stability Problems in Special Materials

- Overconsolidated, Fissured Clays and Clay Shales. See Table 8-2. Cuts in these materials cause opening of fissures and fractures with consequent softening and strength loss.
  - Analysis of Cut Slopes. For long-term stability of cut slopes use residual strength parameters c'_r and ϕ'_r from drained tests. The most reliable strength information for fissured clays is frequently obtained by back figuring the strength from local failures.
  - Old Slide Masses. Movements in old slide masses frequently occur on relatively flat slopes because of gradual creep at depth. Exploration may show the failure mass to be stiff or hard; but a narrow failure plane of low strength with slickensides or fractures may be undetected. In such locations, avoid construction which involves regrading or groundwater rise that may upset a delicate equilibrium.
- Saturated Granular Soils in Seismic Areas. Ground shaking may result in liquefaction and strength reduction of certain saturated granular soils. Empirical methods are available for estimating the liquefaction potential.
- Loess and Other Collapsible soils. Collapse of the structure of these soils can cause a reduction of cohesion and a rise in pore pressure.
  - Evaluate the saturation effects with unconsolidated undrained tests, saturating samples under low chamber pressure prior to shear. See above for evaluating collapse potential.
- Talus. For talus slopes composed of friable material, ϕ may range from 20º to 25º. If consisting of debris derived from slate or shale, ϕ may range from 20º to 29º, limestone about 32º, gneiss 34º, granite 35º to 40º. These are crude estimates of friction angles and should be supplemented by analysis of existing talus slopes in the area.

8.5. Lateral Squeeze of Foundation Soil

Field observations and measurements have shown that some bridge abutments supported on piling driven through thick deposits of soft compressible soils have tilted toward the backfill. Many of the structures have experienced large horizontal movements resulting in damage to the structure. The cause of this problem is the unbalanced fill load, which "squeezes" (consolidates) the soil laterally. This "lateral squeeze" of the soft foundation soil can transmit excessive lateral thrust which may bend or push the piles out, causing the abutment to rotate back toward the fill, as illustrated in Figure 8-12.

Figure 8-12 Lateral Squeeze of Foundations



Two questions that need to be considered in design are:

1. Can tilting occur?
2. If so, how can the amount of horizontal movement be estimated?

8.5.1. Tilting

Experience has shown that if the applied surface load imposed by the fill weight exceeds 3 times the cohesive shear strength of the soft soil, i.e.,

$$\textbf{Equation 8-7: } c < \frac{\gamma_{fill} H_{fill}}{3}$$

Then this lateral squeeze of the foundation soil and abutment tilting can occur.

Therefore, using the above relationship, the possibility of abutment tilting can be evaluated in design. For all practical purposes, the fill unit weight can be assumed at 125 pcf. The cohesive strength c of the soft soil must be determined either from *in situ* field vane shear tests, triaxial tests on high quality undisturbed Shelby tube samples.

8.5.2. Estimation of Amount of Horizontal Abutment Movement

The amount of horizontal movement the abutment may undergo toward the fill can also be estimated in design. This data provides a basis for estimating horizontal abutment movement for similar problems, providing a reasonable estimate of the post-construction fill settlement is made, using data from consolidation tests on high quality undisturbed Shelby tube samples. Note that the data for the structures listed in the previous summary showed horizontal abutment movement to range from 6 to 33% of the vertical fill settlement, with the average being 21%.

Therefore, if the fill load exceeds the 3c limit, then the horizontal abutment movement that may occur can reasonably be estimated as 25% of the vertical fill settlement, i.e.,

Equation 8-8: $$\text{Horizontal Abutment Movement} = \frac{\text{Fill Settlement}}{4}$$

8.5.3. Design Solutions to Prevent Abutment Tilting

The best way to handle the abutment tilting problem is to prevent it by getting the fill settlement out before the abutment piling are driven.

If the construction time schedule or other factors do not permit the settlement to be removed before the piling can be driven, then the problems resulting from abutment tilting can be provided for by the following design provisions:

- Use sliding plate expansion shoes large enough to accommodate the anticipated horizontal movement.
- Make provisions to fill in the bridge deck expansion joint over the abutment by inserting either metal plate fillers or larger neoprene joint fillers.
- Design piles for downdrag forces due to settlement.
- Use steel H-piles for the abutment piling since steel H-piles are capable of taking large tensile stresses without failing.
- Use backward battered piles at the abutment and particularly the wingwalls.

Movements should also be monitored so that predicted movement can be compared to actual.

8.6. Slope Stabilization

There are usually several solutions to a stability problem. The one chosen should be the most economical considering the following factors:

1. Available materials.
2. Quantity and cost of materials.
3. Construction time schedules.
4. Line and grade requirements.
5. Right-of-way.

Practical design solutions to stability problems are summarized below:

- Relocate structure. A line shift of a highway or just moving a structure, to a better soils area may be the most economical solution.[245]
- Reduce grade line. A reduction in grade line will decrease the weight of the embankment and may provide stability.
- Counterweight berms. The weight of counterberms, being on the outside of the centre of rotation, provides an increased moment that resists failure. This increases the factor of safety. Berms should be built concurrently with embankment. The embankment should never be completed prior to berm construction, since the critical time for shear failure is at the end of embankment work.
- Excavation of soft soil. The strength of swamp soil is often insufficient to support embankments. In such cases, soft soils are excavated and replaced with granular material.

[245] Always consider these simple solutions first to avoid more complicated, expensive solutions that follow.

- Displacement of soft soil. For deep swamp deposits, excavation is difficult. The swamp soil can be displaced by continuous shear failures along the advancing fill front until the embankment is on firm bottom. The mudwave forced up in front of the fill must be excavated to insure continuous displacement and prevent large pockets of swamp soil from being trapped under the fill.
- Slow rate or stage construction. Many weak subsoils will tend to gain strength during the loading process as consolidation occurs and pore water pressures dissipate. For soils that consolidate relatively fast, such as some silts and silt clays, this method is practical. Proper instrumentation is desirable to monitor the state of stressing the soil during the loading period to insure that loading does not proceeds o rapidly as to cause a shear failure. Typical instrumentation consists of slope inclinometers to monitor stability, piezometers to measure porewater pressure, and settlement devices to measure amount and rate of settlement. Planning of the instrumentation program and data interpretation should be done by a qualified geotechnical engineer.
- Lightweight embankment. In some areas of the U.S., lightweight blast furnace slag or expanded shale is available. This material weighs about 80 pounds per cubic foot.[246] Sawdust is also available in

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[246] New York DOT Embankment in Place Specification (Lightweight Fill)

Description: Under this Item, the Contractor shall furnish and place lightweight fill necessary to complete the embankments shown on the plans or as ordered by the Engineer.

Materials: The material shall be blast furnace slag, expanded shale or other materials as approved by the Deputy Chief Engineer, Technical Services. The material shall have a maximum particle size of 24 inches in greatest dimension and the compacted wet density shall not exceed the density specified in the proposal as measured in a test embankment.

Construction Details

The compacted wet density shall be determined in test embankments containing a minimum of 400 cubic yards of material constructed on firm flat surfaces. The Contractor shall construct each test embankment in an area bounded by 100 ft. by 50 ft. dimensions and he shall give th engineer at least one (1) week written notice prior to beginning each test in order for the location to be inspected and surveyed.

The lightweight fill material shall be stored in piles not exceeding 20,000 cubic yards prior to testing. Representative material from each storage pile shall be used to construct a test embankment to a minimum height of four (4) feet in accordance with this specification.

The Contractor shall weigh all the material prior to placement in the test embankment. The embankment shall be constructed in uniform layers not exceeding 24 inches in thickness prior to compaction. Each layer shall be rolled over its entire area by a vibratory steel drum roller, and the need for actually vibrating the roller will be as directed by the Engineer.

The Engineer shall determine the volume of the test embankment. If the compacted wet density of the material in the test embankment is greater than the specified density, both the material contained in the test embankment and the material from the storage pile it represents shall be rejected for use under this Item.

The design embankment shall be constructed using the same methods, equipment and procedures used to construct the test embankments. However, the following requirements contained in the earthwork section shall now apply:

a. The density requirements both in the embankment and in the subgrade area.

b. The maximum particle size in the subgrade area.

c. Proof rolling.

d. Compaction.

The top surface of the lightweight embankment lying directly beneath the subbase course materials shall be chinked to the satisfaction of the Engineer with lightweight material to prevent infiltration of the subbase materials.

timber producing areas such as the northwest U.S. Sawdust fill weighs about 60 pounds per cubic foot and has friction angle of 35º.[247] The overturning force is decreased by the lighter embankment weight.

- Seepage and Groundwater Control. Surface control of drainage decreases infiltration to potential slide area. Lowering of groundwater increases effective stresses and eliminates softening of fine-grained soils at fissures. Details on seepage and groundwater control are found earlier.
- Retaining Structures.
    - Application. Walls or large diameter piling can be used to stabilize slides of relatively small dimension in the direction of movement or to retain steep toe slopes so that failure will not extend back into a larger mass.
    - Analysis. Retaining structures are frequently misused where active forces on wall are computed from a failure wedge comprising only a small percentage of the total weight of the sliding mass. Such failures may pass entirely beneath the wall, or the driving forces may be large enough to shear through the retaining structure. Stability analysis should evaluate a possible increase of pressures applied to wall by active wedge extending far back into failing mass, and possible failure on sliding surface at any level beneath the base of the retaining structure.
    - Piles or Caissons. To be effective, the piles should extend sufficiently below the failure surface to develop the necessary lateral resistance. Figure 8-13 shows how the effect of the

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Method of Measurement: The quantity of lightweight fill to be paid for under this Item shall be the number of cubic yards of material computed in its final compacted position between the payment lines shown on the plans or between revised payment lines established by the Engineer prior to performing the work.

Basis of Payment: The unit price bid per cubic yard shall include the cost of furnishing all labor, material and equipment necessary to complete the work including the test embankments.

No payment will be made for any loss of material which may result from foundation settlement, erosion or any other cause. The cost of such losses shall be included in the price bid for this Item.

[247] The following is the special provision for lightweight sawdust fill used by the Washington State DOT as of March 1982.

Where shown in the plans or where directed by the Engineer, the Contractor shall furnish, load, haul, place, and compact sawdust borrow in place.

Materials: The sawdust borrow shall consist of 100% wood fibers, such as sawdust, hog fuel or wood chips. No composition wood products, such as particle or chip board, pressed hard board, or presto-log fragments shall be used in this embankment. Maximum size shall be 6 inches in the greatest dimension. Sufficient smaller sized material shall be used to produce a uniformly dense fill. Cedar sawdust borrow will not be allowed.

Construction: The sawdust borrow embankments may be constructed by dumping from trucks or by any other methods approved by the Engineer. Sawdust borrow shall be placed in lifts a maximum of 1 foot in depth of uncompacted material.

Compaction shall be obtained by covering the entire surface of each lift with a minimum of two passes with a D8-Caterpillar tractor or other similar compaction units as approved by the Engineer. Hauling units shall be routed over the entire fill for additional compaction.

Measurement: Sawdust borrow in place will be measured by the cubic yard of neat line volume in place.

Payment: The unit contract price per cubic yard for "Sawdust Borrow in Place" shall be full compensation for furnishing all labor, tools, equipment and materials necessary or incidental to complete the work as specified, including loading, hauling, placing, and compacting.

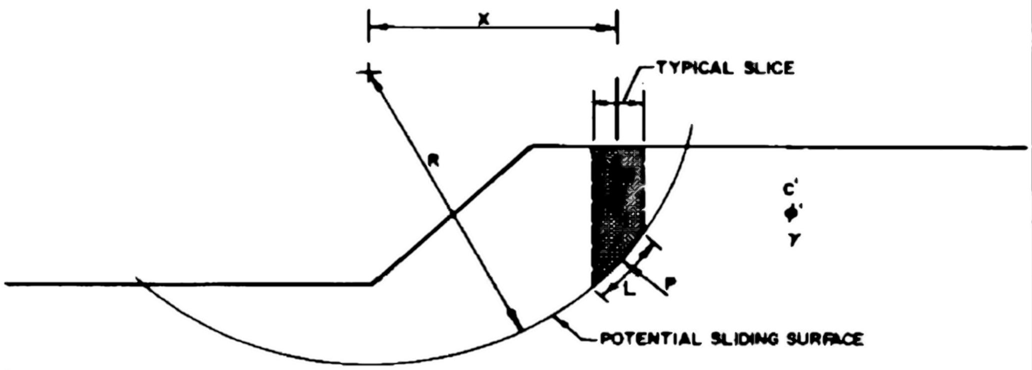
piles is considered in calculating the factor of safety. The distribution of pressure along the pile can be computed from charts shown in Figure 8-14. This assumes full mobilization of soil shear strength along the failure surface and should be used only when the safety factor without the piles is less than 1.4.[248]

- o See Figure 8-15 for example computations. Note the computations shown are for only one of the many possible slip surfaces.

∞ Other Methods.

- o Other potential procedures for stabilizing slopes include grouting, freezing, electro osmosis, vacuum pumping, and diaphragm walls.

Figure 8-13 Influence of Stabilizing Pile on Safety Factor

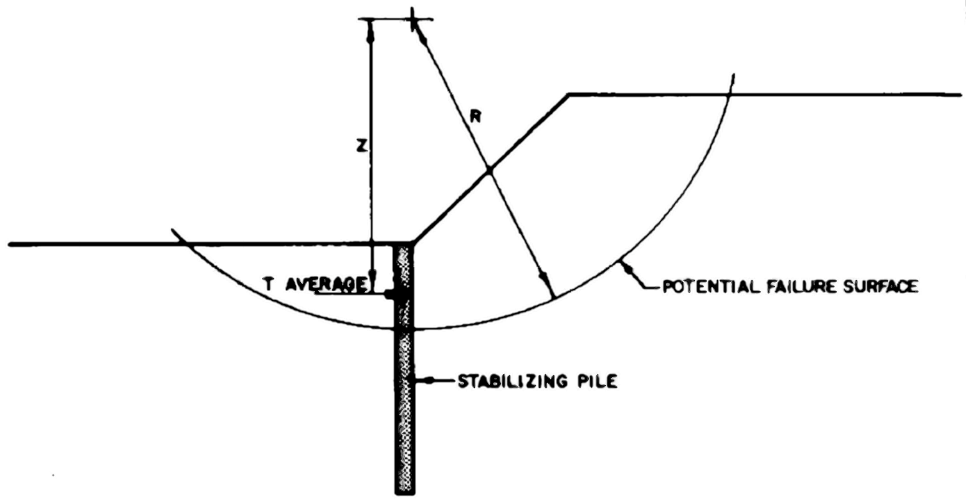


SAFETY FACTOR (F_s) FOR MOMENT EQUILIBRIUM FOR METHOD OF SLICES WITH CIRCULAR POTENTIAL SLIDING SURFACE (WITHOUT SEISMIC LOADS, SURCHARGE LOADS, OR PARTIAL SUBMERGENCE OF SLOPE) IS TYPICALLY DEFINED AS:

$$F_s = \frac{\Sigma c'LR + \Sigma(P - uL)R \tan\phi'}{\Sigma WX}$$

WHERE:
- c' = EFFECTIVE COHESION.
- ϕ' = EFFECTIVE FRICTION ANGLE.
- W = TOTAL WEIGHT OF SLICE.
- P = TOTAL NORMAL FORCE ON BASE OF SLICE.
- L = LENGTH OF POTENTIAL SLIDING SURFACE ACROSS SLICE.
- u = AVERAGE PORE WATER PRESSURE ON POTENTIAL SLIDING SURFACE ACROSS SLICE.
- R = RADIUS OF MOMENT ARM FOR POTENTIAL SLIDING SURFACE
- X = HORIZONTAL DISTANCE FROM CENTROID OF SLICE TO CENTER OF ROTATION.

[248] This criteria is based on results of analysis presented in DeBeer, E.E. and Wallays, M., Forces Induced in Piles by Unsymmetrical Surcharges on the Soil Around the Pile, Proceedings of Fifth European Conference Soil Mechanics and Foundation Engineering, Madrid, Volume 1, p. 335, 1972.



SAFETY FACTOR FOR MOMENT EQUILIBRIUM CONSIDERING THE SAME FORCES AS ABOVE PLUS THE EFFECT OF THE STABILIZING PILE IS EXPRESSED AS:

$$F_s = \frac{\Sigma\, c' L R + \Sigma (P - uL) R \tan \phi' + TZ}{\Sigma\, WX}$$

WHERE: T = AVERAGE TOTAL THRUST (PER LIN. FT., HORIZ.) RESISTING SOIL MOVEMENT.

Z = DISTANCE FROM CENTROID OF RESISTING PRESSURE (THRUST) TO CENTER OF ROTATION.

Figure 8-14 Pile Stabilized Slope

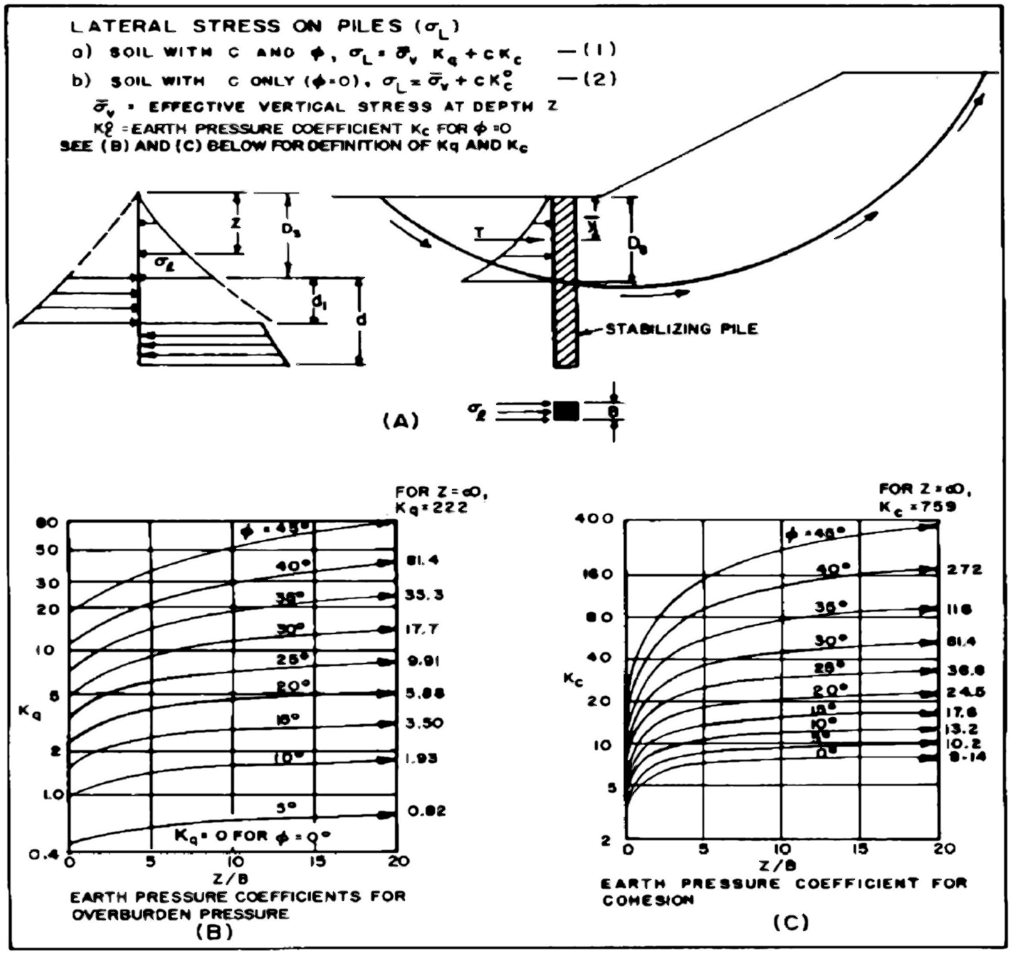
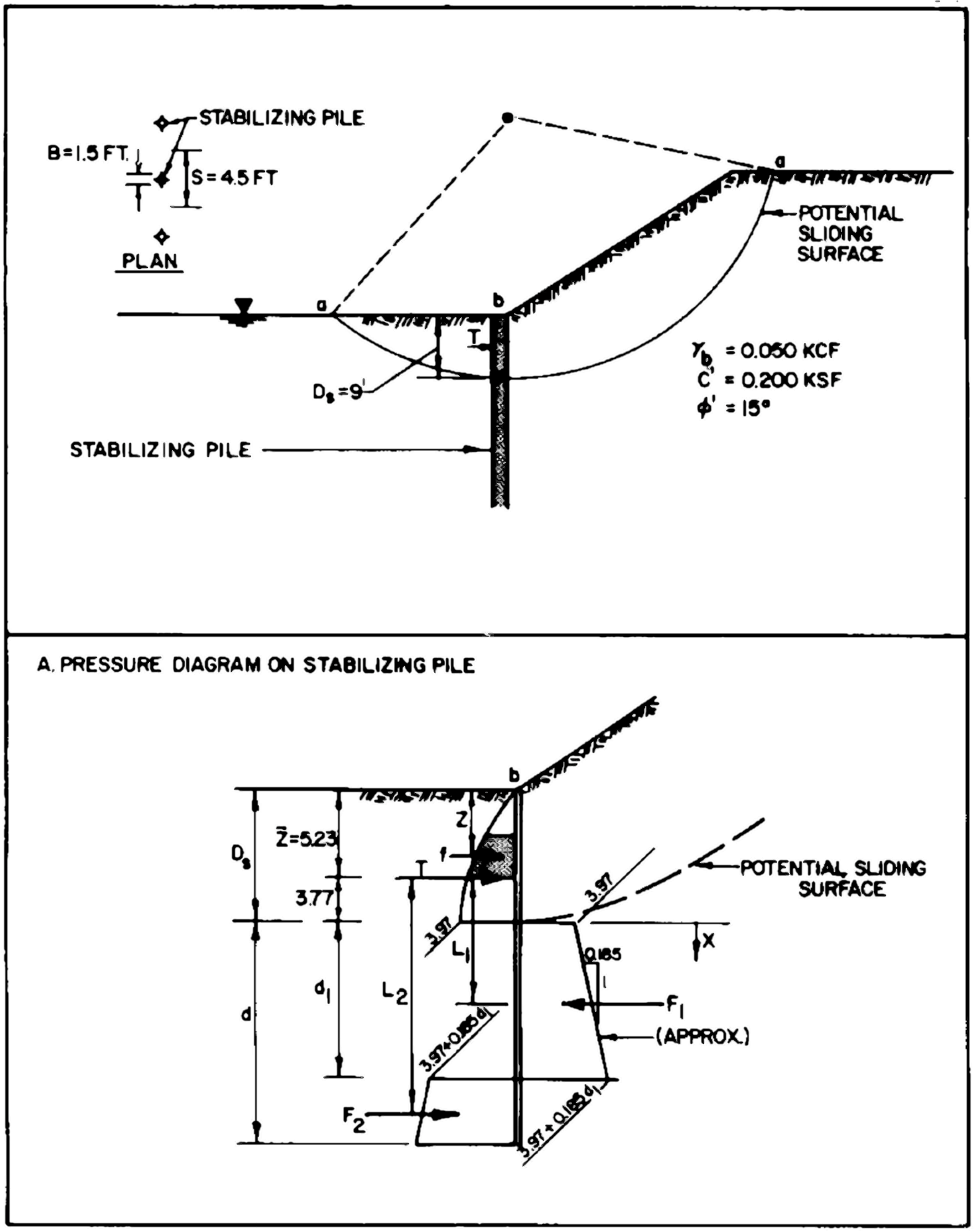


Figure 8-15 Example Calculation – Pile Stabilized Slopes



B. For trial slip surface a-a compute lateral resistance, generated by presence of pile if factor of safety without piles is less than 1.4. Compute pressures using Figure 12.

$\sigma_L = \bar{\sigma}_V K_q + cK_c$ } SEE FIGURE 12 FOR DEFINITIONS

Depth Below Top of Pile Z (ft)	Z/B	K_q	K_c	Vertical Effective Stress $\bar{\sigma}_V$ (kips/ft^2)	Lateral Resistance to Soil Movement σ_L KSF
0	0	1.5	4.0	0	0.8
3	2	2.1	10.8	0.15	2.48
6	4	2.4	12.8	0.30	3.28
9	6	2.6	14.0	0.45	3.97

C. Compute centroid of lateral resistance (i.e., location of force T)

Depth Range	Resultant Resistance (f) Over Depth Range	Z	fZ
0-3	$3\left(\frac{0.8 + 2.48}{2}\right) B = 4.92B$	1.5	7.38B
3-6	8.64B	4.5	38.88B
6-9	10.87B	7.5	81.53B
	$\Sigma T = 24.43B$		127.79B

$\bar{Z} = 127.79/24.43 = 5.23$ ft

D. Lateral resistance per linear foot of slope

$$T_1 = \Sigma T/S = 24.43 \times 1.5/4.5 = 8.14^k$$

Note that T accounts for three dimensional condition and need not be corrected.

E. Use T_1 in Step D and $\bar{Z}$ in Step C to compute additional stabilizing moment for evaluating safety factor including effect of piles (see Figure 11).

F. Compute +L, at depth corresponding to Z/B = 20 (Z = 30) in order to compute average Increase of positive resistance with depth:

$$K + q = 3.1, K + c = 16$$

$$\sigma + L = 3.1 \times 30 \times 0.05 + 16 \times 0.2 = 7.85 \text{ KSF}$$

Average increase in lateral resistance below D_s:

$$\sigma + L_{avg} = (7.85 - 3.97)/(30 - 9) = 0.185 \text{ KSF/ft}$$

Assume that the direction of lateral resistance changes at depth d+1, beneath failure surface, then:

G. Calculate depth of penetration d by solving the following equations and increase d by 30% for safety:

$$T + F_2 - F_1 = 0 \ (1)$$

$$F_1L_1 = F_2L_2 \ (2)$$

Compute forces per unit pile width:

$$T = 24.43.k-$$

$$F_1 = 3.97d_1 + 0.092d_1^2$$

$$F_2 = (3.97 + 0.185d_1)(d - d_1) + 0.092\ (d - d_1)^2$$

$$= 0.092d^2 + 3.97d - 3.97d_1 - 0.092d_1^2$$

H. Use Eq (1) in Step G to calculate d_1 for given values of d.

$$24.43 + 0.092d^2 + 3.97d - 7.94d_1 - 0.185d_1^2 = 0$$

$$d_1^2 + 42.9d_1 - (24.43 + 0.092d^2 + 3.97d)/\ 0.185 = 0$$

Let d = 15.8', then d_1 = 11.0'

From Eq (2) Step G (consider each section of pressure diagram broken down as a rectangle and triangle).

$$F_1 L_1 = \left[3.97 \times 11.0 \times \left(3.77 + \frac{11.0}{2}\right)\right] + \left[\frac{0.185}{2} \times 11.0^2 \times \left(3.77 + \frac{2 \times 11.0}{3}\right)\right]$$

$= 529.1$ FT-KPS

$d - d_1 = 4.8$

$$F_2 L_2 = \left[(3.97 + 0.185 \times 11.0) \times 4.8 \times \left(3.77 + 11.0 + \frac{4.8}{2}\right)\right]$$

$$+ \left[\frac{0.185}{2} \times 4.8^2 \times \left(3.77 + 11.0 + \frac{2 \times 4.8}{3}\right)\right]$$

$= 533.2$

$F_1 L_1 - F_2 L_2 = -4.1$

$d \approx 15.8'$ O.K.

I. Design

Increase d by 30% to obtain the practical driving depth

$d = 15.8 \times 1.3 = 20.5'$

LOCATE POINT OF ZERO SHEAR

$24.43 = 3.97X + 0.092X^2$

$X^2 + 43.15X - 265.54 = 0$

$$X = \frac{-43.15 \pm \sqrt{43.15^2 + 4 \times 265.54}}{2}$$

$= 5.46'$

COMPUTE MAXIMUM BENDING ON PILE ($B = 1.5'$)

$$M_{max} = \left[24.43 \times (3.77 + 5.46) - \left(\frac{3.97 \times 5.46^2}{2} + \frac{0.185 \times 5.46^3}{2 \times 3}\right)\right] \times 1.5$$

$= 241.9$ Kp-FT

CHECK PILE SECTION VS M_{max}

NOTES:

a. Higher embedment may be required to minimize slope movements.

b. Use residual shear strength parameters if appropriate.

c. Analysis applicable for safety factor $\leq$ 1.4 without piles. Soil movement assumed to be large enough to justify assumption on rupture conditions.

8.7. Slope Protection

8.7.1. Slope Erosion

Slopes that are susceptible to erosion by wind and rainfall should be protected. Protection is also required for slopes subjected to wave action as in the upstream slope of a dam, or the river and canal banks along navigational channels. In some cases, provision must be against burrowing animals.

8.7.2. Types of Protection Available

The usual protection against erosion by wind and rainfall is a layer of rock, cobbles, or sod. Protection from wave action may be provided by rock riprap (either dry dumped or hand placed), concrete pavement, precast concrete blocks, soil-cement, fabric, and wood. See Table 7-34 for additional guidance.

- Stone Cover. A rock or cobbles cover of 12" thickness is sufficient to protect against wind and rain.
- Sod. Grasses suitable for a given locality should be selected with provision for fertilizing and uniform watering.
- Dumped Rock Riprap. This provides the best protection against wave action. It consists of rock fragments dumped on a properly graded filter. Rock used should be hard, dense, and durable against weathering and heavy enough to resist displacement by wave action. See Table 8-5 for design guidelines. For additional design criteria see Figure 6-16.
- Hand-placed Riprap. Riprap is carefully laid with minimum amount of voids and a relatively smooth top surface. Thickness should be one-half of the dumped rock riprap but not less than 12". A filter blanket must be provided and enough openings should be left in the riprap facing to permit easy flow of water into or out of the riprap.
- Concrete Paving. As a successful protection against wave action concrete paving should be monolithic and of high durability. Underlying materials should be pervious to prevent development of uplift water pressure. Use a minimum thickness of 6".

  When monolithic construction is not possible, keep the joints to a minimum and sealed. Reinforce the slab at mid depth in both directions with continuous reinforcement through the construction joints. Use steel area in each direction equal to 0.5% of the concrete area.
- Gabions. Slopes can be protected by gabions.
- Geotextiles. Geotextiles are frequently used to protect slopes, both by facilitating water runoff without erosion and to enable slope surfaces to be seeded and grass to grow without washout taking place.

Table 8-5 Thickness and Gradation Limits of Dumped Riprap

Slope	*Nominal thickness, inches*	*Gradation, percentage of stones of various weights, pounds*[249]			
		Maximum Size	40 to 50% greater than	50 to 60% from – to	0 to 10 percent less than[250]
3:1	30	2,500	1,250	75-1,250	75
2:1	36	4,500	2,250	100-2,250	100

[249] Sand and rock dust shall be less than 5%, by weight, of the total riprap material.

[250] The percentage of this size material shall not exceed an amount which will fill the voids in larger rock.

8.8. Dynamic Slope Stability And Deformations

8.8.1. Slope Stability under Seismic Loading

Well compacted cohesionless embankments or reasonably flat slopes in insensitive clay that are safe under static conditions are unlikely to fail under moderate seismic shocks (up to 0.15 g or 0.20 g acceleration). Embankment slopes made up of insensitive cohesive soils founded on cohesive soil or rock can withstand higher seismic shocks. For earth embankments in seismic regions, provide internal drainage and select core material suited to resist cracking. In regions where embankments are made up of saturated cohesionless soil, the likelihood for liquefaction should be evaluated using detailed dynamic analysis[251].

8.8.2. Seismically Induced Displacement

Computation of slope displacement induced by earthquakes requires dynamic analysis. In 1965, Newmark pioneered a simplified computation procedure using acceleration data.

8.8.3. Slopes Vulnerable to Earthquakes

Slope materials vulnerable to earthquake shocks are:

a) Very steep slopes of weak, fractured, and brittle rocks or unsaturated loess are vulnerable to transient shocks that are likely to induce the opening of tension cracks.

b) Loose, saturated sand may be liquefied by shocks that may resist collapse of structure and flow slides.

c) Similar effects as b) are possible in sensitive cohesive soils with natural moisture exceeding the liquid limit.

d) Dry cohesionless material on a slope at the angle of repose will respond to seismic shock by shallow sloughing and slight flattening of the slope.

8.8.4. Analysis of Seismic Loading on Slopes

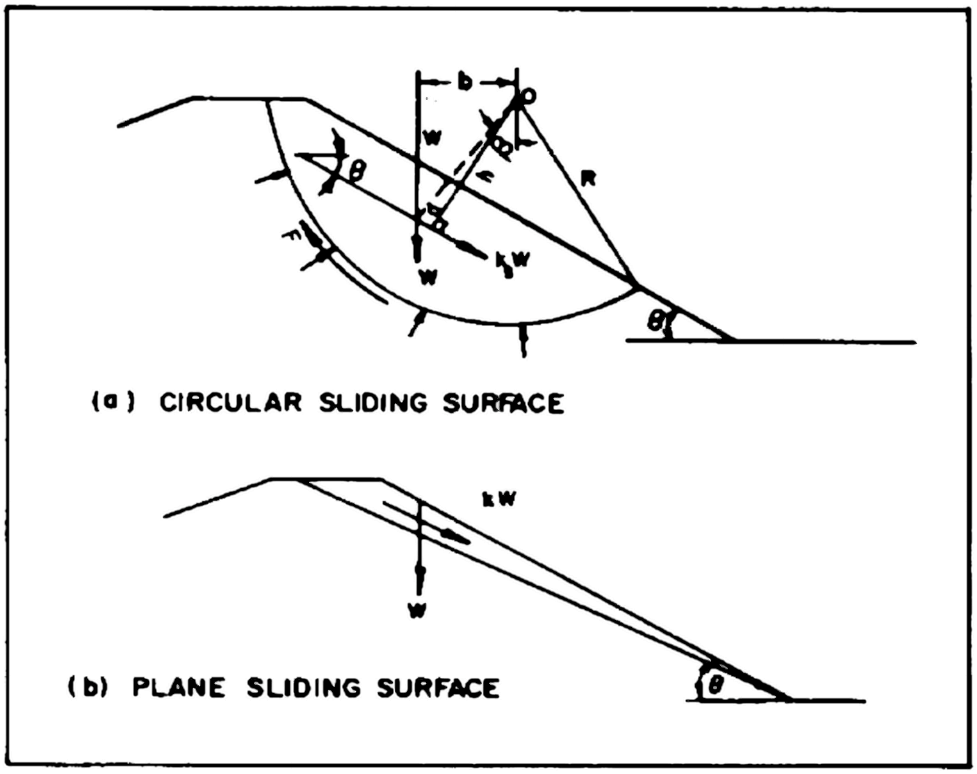
Seismically induced displacement of a slope in earth structures, such as dams or earth retaining structures can be computed using the Newmark method. The potential sliding blocks are identified using slope stability analysis.

8.8.4.1. Sliding Block Analysis

Earthquake effects can be introduced into the analysis by assigning a disturbing force on the sliding mass equal to kW where W is the weight of the sliding mass and k is the seismic coefficient. For the analysis of stability shown in Figure 8-16a, k_sW is assumed to act parallel to the slope and through the centre of mass of the sliding mass.

[251] Makdisi, F. I. and Seed, H. B., Simplified Procedures for Estimating Dam and Embankment Earthquake Induced Deformations, Journal of the Geotechnical Division, Vol. 104, No. GT7, 1978.

Figure 8-16 Earthquake Loading on Slopes



Thus, for a factor of safety of 1.0:

Equation 8-9: $FR = Wb + k_s Wh$

The factor of safety under an earthquake loading then becomes

Equation 8-10: $F_{Se} = \dfrac{FR}{Wb + k_s Wh}$

To determine the critical value of the seismic efficient (k_{cs}) that will reduce a given factor of safety for a stable static condition (F_{So}) to a factor of safety of 1.0 with an earthquake loading (F_{Se} = 1.0), use

Equation 8-11: $k_{cs} = \dfrac{b}{h}(F_{So} - 1) = (F_{So} - 1)\sin\theta$

- If the seismic force is in the horizontal direction and denoting such force as k_{ch} W, then $k_{ch} = (F_{So}-1)\tan\theta$.
- For granular, free-draining material with plane sliding surface (Figure 8-16b): $F_{So} = \tan\phi/\tan\theta$, and $k_{cs} = (F_{So}-1)\sin\theta$.
- Based on several numerical experiments, k_{ch} may be conservatively represented as $k_{ch} \approx (F_{So}-1)/4$.[252]

The down slope movement U may be conservatively predicted as:[253]

[252] Sarma, S.K. and Bhave, M.V., Critical Acceleration Versus Static Factor of Safety in Stability Analysis of Earth Dams and Embankments, Geotechnique, Vol. 24, No. 4, 1974.

Equation 8-12: $U = \frac{V^2}{2gk_{cs}}\frac{A}{k_{cs}}$

where

- A = peak ground acceleration, g's
- g = acceleration of gravity
- V = peak ground velocity

The above equations are based on several simplifying assumptions:

1) Failure occurs along well defined slip surface,
2) The sliding mass behaves as a rigid body;
3) Soils are not sensitive and would not collapse at small deformation; and
4) There is no reduction in soil strength due to ground shaking.

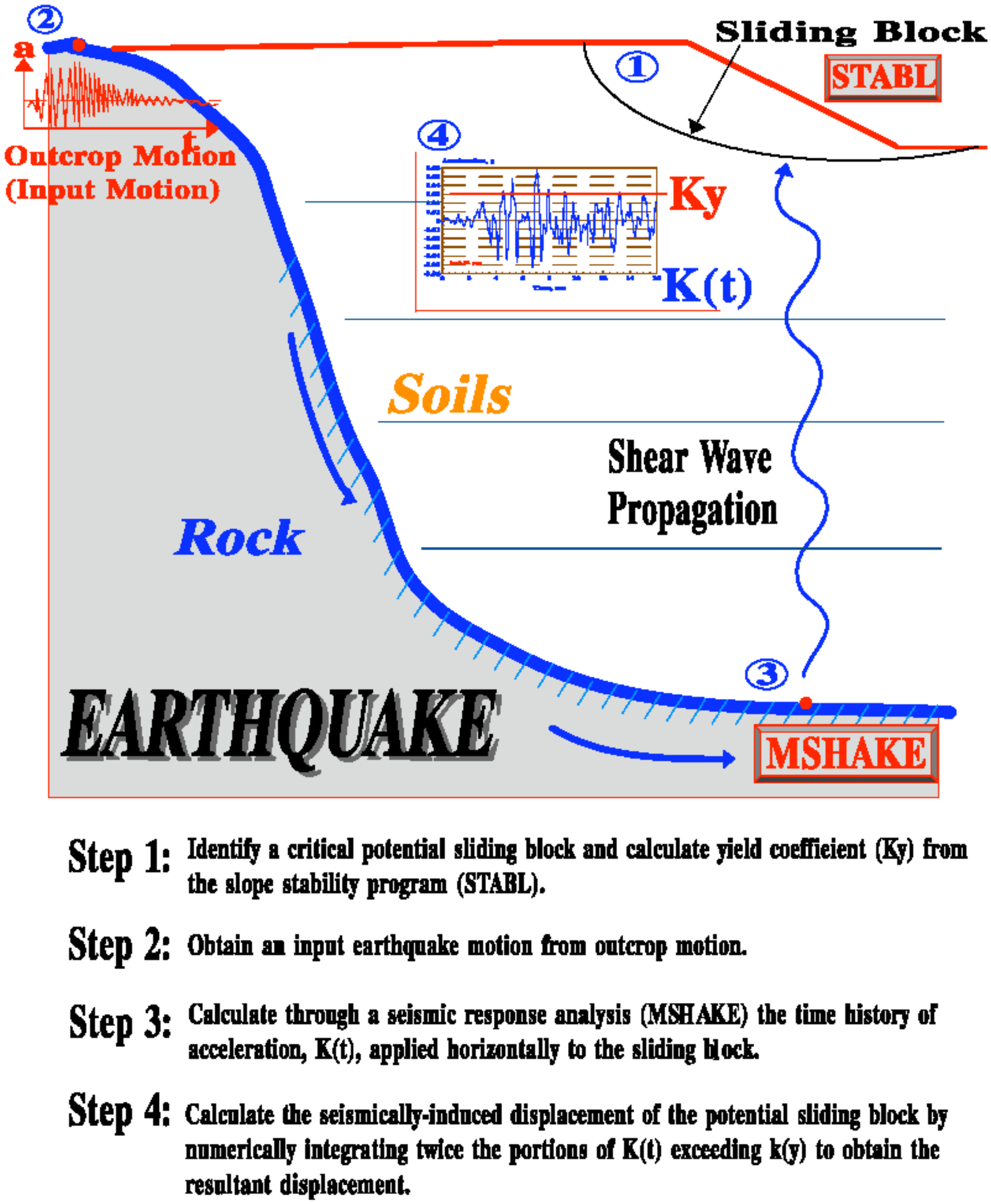
8.8.4.2. Newmark's Method

The earthquake-induced displacement of a potential slope-sliding block can be estimated from acceleration data using the Newmark method of prediction of embankment deformation induced by earthquake. Deformation of slope caused by earthquake can be estimated from the following four steps. These steps are shown in Figure 8-17.

a) Identify a critical potential sliding block, using slope stability program to find the yielding coefficient of earthquake loading, K_y, required to cause failure.
b) Obtain an input earthquake motion appropriate for the specific sites.
c) Find the average acceleration, K_t, from the acceleration time history of the site using seismic response analysis or other equivalent linear analysis. The yield coefficient is calculated from the average acceleration time history.
d) Calculate the seismically induced displacement of the potential sliding block by double integrating the potential of K_t exceeding K_y.

[253] Newmark, N.M., Effects of Earthquakes on Dams and Embankments, Geotechnique, Vol. 15, No. 2, 1965.

Figure 8-17 Prediction of Embankment Deformation Induced by Earthquake



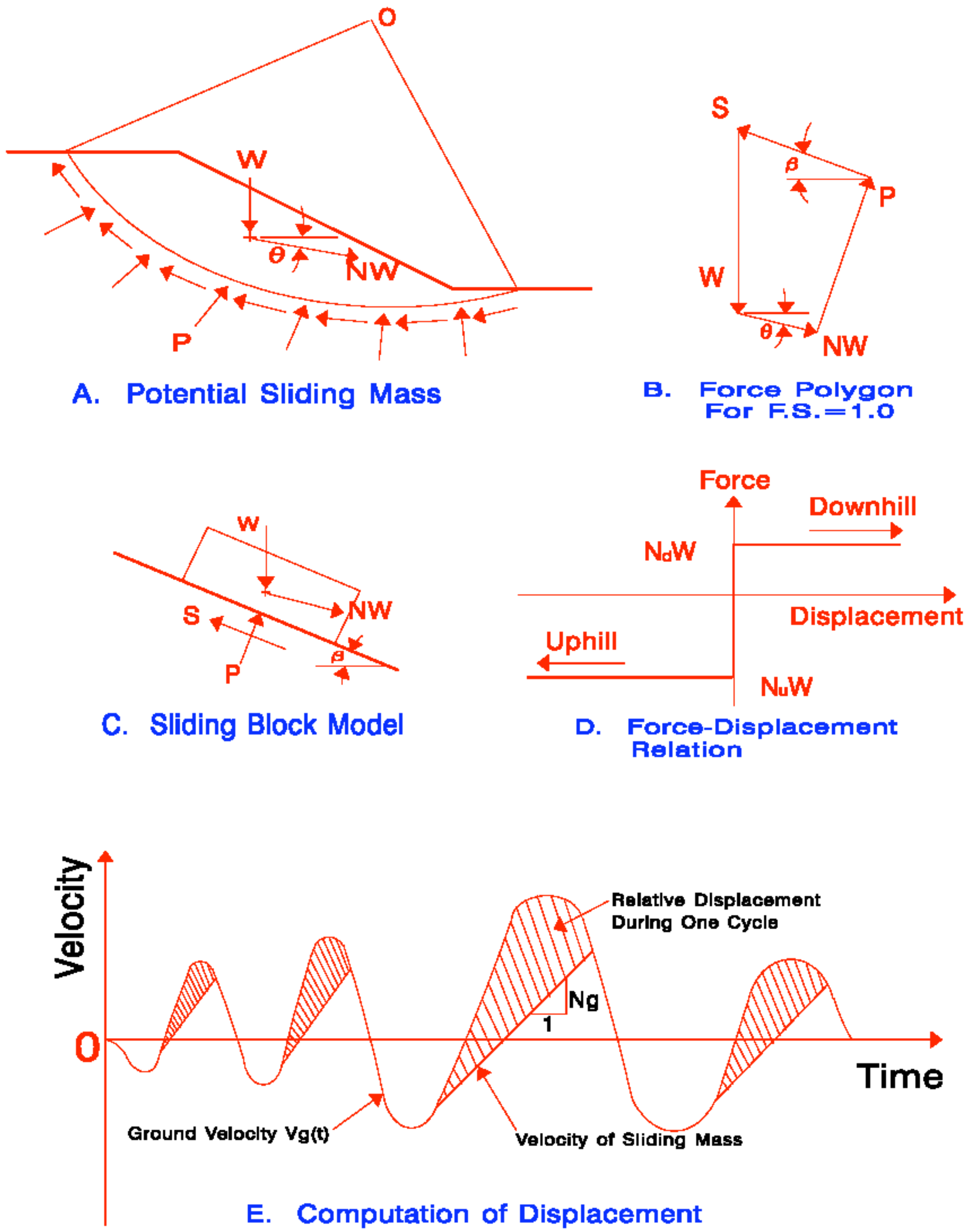
8.8.4.3. Sliding Block Analogy

Figure 8-18 depicts the principal components of the sliding block analysis[254]. The potential sliding mass in Figure 8-18A is assumed to be in a condition of impending (limiting equilibrium) failure, so that the

[254] Miscellaneous Earthquake Resistance of Earth and Paper S-171-17, Rock-Fill Dams Permanent Displacements Report 5 of Earth Embankments by Newmark Sliding Block Analysis, U.S. Army Engineer Waterway Experiment Station, EC, Vicksburg, MS, A. G. Franklin and F. K. Chang, 1977.

factor of safety equals unity. This condition is caused by acceleration of both the base and the mass toward the left of the sketch with an acceleration of N_g.

Figure 8-18 Principle Components of the Sliding Block Analysis



Acceleration of the mass is limited to this value by the limit of shear stresses that can be exerted across the idealized sliding contact, so that if base acceleration were to increase, the mass would move downhill relative to the base. By d'Alembert's principle, the limiting acceleration is represented by an inertia force NW applied pseudostatically to the mass in a direction opposite to acceleration and at the same angle θ. Figure 8-18B shows the balanced force polygon for the situation. The angle of inclination θ of the inertia force may be found as the angle that is most critical; i.e., the angle that minimizes N. Its value is usually within a few degrees of zero, and since the results of the analysis are not sensitive to it, the vertical component can generally be ignored or, equivalently, θ can be zero. The angle β is the direction of the resultant S of the shear stresses on the interface and is determined by the limit equilibrium stability analysis. The same force polygon applied to the model of a sliding block on a plane inclined at an angle model is used to represent the sliding mass at an angle β to the horizontal (Figure 8-18C). Hence, the sliding block model is used to represent the sliding mass in an embankment.

The force-displacement relation diagrammed in Figure 8-18D is assumed to apply to this sliding block system. The force in this diagram is the inertia force associated with the instantaneous acceleration of the block, and the displacement is the sliding displacement of the block relative to the base. It is usually assumed that resistance to uphill sliding is large enough that all displaced are downhill. If the base is subjected to a sequence of acceleration pulses (the earthquake) large enough to induce sliding of the block, the block will come to rest at some displaced position down the slope after the motion has ceased. The amount of permanent displacement, u, can be computed by using Newton's second law of motion, F = ma, to write the equation of motion for the sliding block relative to the base, and then numerically or graphically integrating (twice) to obtain the resultant displacement. During the time intervals when relative motion is occurring, the acceleration of the block relative to the base is given by:

Equation 8-13: $$u = a_{rel} = \left(a_{base} - N\right)\left(\frac{\cos(\beta - \theta - \phi)}{\cos\phi}\right) = \left(a_{base} - N\right)\alpha$$

Where

- a_{rel} = relative acceleration between the block and the inclined plane
- a_{base}= acceleration of the inclined plane, a function of time
- N = critical acceleration level at which sliding begins
- β = direction of the resultant shear force and displacement, and the inclination of the plane
- θ = direction of the acceleration, measured from the horizontal
- ϕ = friction angle between the block and the plane

The acceleration a_{base} is the earthquake acceleration acting at the level of the sliding mass in the embankment. It is assumed equal to the bedrock acceleration multiplied by an amplification factor that accounts for the quasi-elastic response of the embankment.

The amount of permanent displacement is determined by twice integrating the relative accelerations over the total duration of the earthquake record. It is assumed that ϕ, β, and θ do not change with time; thus, the coefficient α is constant and is not involved in the integration. In the final stage of analysis, the result of the integration is multiplied by the coefficient α, the determination of which requires knowledge of embankment properties and the results of the pseudostatic stability analysis. For most practical problems, the coefficient α may be assumed a value of unity, as it generally differs from unity by less than 15%.

The second step of acceleration integration is illustrated by the plot of base velocity versus time in Figure 8-18E. Since the slope of the velocity curve is the acceleration, the limiting acceleration Ng of the block defines the velocity curve for the block by straight lines in those parts of the plot where the critical acceleration has been exceeded in the base. The area between the curves gives the relative displacement. Note that the block continues to move relative to the underlying slope even when a_{base} has fallen below N. The absolute velocity of the block continues to change linearly with time until the velocities of the block and the ground are the same. In effect, the friction between the block and the ground continues to act on the block until the ground catches up with it.

Computer programs are available to compute the cumulative displacement of the sliding block. The work of Franklin and Chang and of others has demonstrated that the cumulative displacements calculated by the sliding block method increase as the ratio N/a_{base} decrease and tend to become significant when the ratio falls below 0.5.

§ 9. Compaction, Earthwork, and Hydraulic Fills

9.1. Introduction

This section concerns design and construction of compacted fills and performance of compacted materials. Compaction requirements are given for various applications and equipment. Earthwork control procedures and analysis of control test data are discussed. Guidance on hydraulic fills is also included.

9.1.1. Purpose of Compaction

1) Reduce material compressibility
2) Increase material strength.
3) Reduce permeability
4) Control expansion.
5) Control frost susceptibility.

9.1.2. Applications

The principal uses of compacted fill include support of structures or pavements, embankments for water retention or for lining reservoirs and canals, and backfill surrounding structures or buried utilities.

9.1.3. Types of Fill

a. Controlled Compacted Fills. Properly placed compacted fill will be more rigid and uniform and have greater strength than most natural soils.
b. Hydraulic Fills. Hydraulic fills cannot be compacted during placement and therefore it is important that the source materials be selected carefully.
c. Uncontrolled Fills. These consist of soils or industrial and domestic wastes, such as ashes, slag, chemical wastes, building rubble, and refuse. Use of ash, slag, and chemical waste is stringently controlled and current Environmental Protection Agency or other appropriate regulations must be considered.

9.2. Embankment Cross-Section Design

9.2.1. Influence of Material Type

Table 9-1 lists some typical properties of compacted soils that may be used for preliminary analysis. For final analysis, engineering property tests are necessary.

Table 9-1 Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression (Percent of Original Height)		Typical Strength Characteristics				Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
				At 1.4 tsf (20 psi)	At 3.6 tsf (50 psi)	Cohesion (as compacted) psf	Cohesion (saturated) psf	ϕ (Effective Stress Envelope Degrees)	Tan ϕ			
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	.....	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-7}$	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 - 150
OH	Organic clays and silty clays	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.
(..) indicates insufficient data available for an estimate.

a. Utilization. See Table 9-2 for relative desirability of various soil types in earth fill dams, canals, roadways and foundations. Although practically any nonorganic insoluble soil may be incorporated in an embankment when modern compaction equipment and control standards are employed, the following soils may be difficult to use economically:

1) Fine-grained soils may have insufficient shear strength or excessive compressibility.

2) Clays of medium to high plasticity may expand if placed under low confining pressures and/or at low moisture contents. See § 2 for identification of soils susceptible to volume expansion.

3) Plastic soils with high natural moisture are difficult to process for proper moisture for compaction.

4) Stratified soils may require extensive mixing of borrow.

Table 9-2 Relative Desirability of Soils as Compacted Fill

Group Symbol	Soil Type	RELATIVE DESIRABILITY FOR VARIOUS USES (No. 1 is Considered the Best, No. 14 Least Desirable)									
		Rolled Earth Fill Dams			Canal Sections		Foundations		Roadways		
									Fills		
		Homogeneous Embankment	Core	Shell	Erosion Resistance	Compacted Earth Lining	Seepage Important	Seepage Not Important	Frost Heave Not Possible	Frost Heave Possible	Surfacing
GW	Well graded gravels, gravel-sand mixtures, little or no fines	–	–	1	1	–	–	1	1	1	3
GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines	–	–	2	2	–	–	3	3	3	–
GM	Silty gravels, poorly graded gravel-sand-silt mixtures	2	4	–	4	4	1	4	4	9	5
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures	1	1	–	3	1	2	6	5	5	1
SW	Well-graded sands, gravelly sands, little or no fines	–	–	3 if gravelly	6	–	–	2	2	2	4
SP	Poorly-graded sands, gravelly sands, little or no fines	–	–	4 if gravelly	7 if gravelly	–	–	5	6	4	–
SM	Silty sands, poorly graded sand-silt mixtures	4	5	–	8 if gravelly	5 erosion critical	3	7	6	10	6
SC	Clayey sands, poorly graded sand-clay mixtures	3	2	–	5	2	4	8	7	6	2
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	6	6	–	–	6 erosion critical	6	9	10	11	–
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	5	3	–	9	3	5	10	9	7	7
OL	Organic Silts and organic silt-clays of low plasticity	8	8	–	–	7 erosion critical	7	11	11	12	–
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	9	9	–	–	–	8	12	12	13	–
CH	Inorganic clays of high plasticity, fat clays	7	7	–	10	8-vol change critical	9	13	13	8	–
OH	Organic clays of medium high plasticity	10	10	–	–	–	10	14	14	14	–

9.2.2. Embankments on Stable Foundation

The side slopes of fills not subjected to seepage forces ordinarily vary between 1 on 1-1/2 and 1 on 3. The geometry of the slope and berms are governed by requirements for erosion control and maintenance. See § 8 for procedures to calculate stability of embankments.

9.2.3. Embankments on Weak Foundations

Weak foundation soils may require partial or complete removal, flattening of embankment slopes, or densification. Analyse cross-section stability by methods of § 8.

9.2.4. Embankment Settlement

Settlement of an embankment is caused by foundation consolidation, consolidation of the embankment material itself, and secondary compression in the embankment after its completion.

a. Foundation Settlement. See 5.4 for procedures to decrease foundation settlement or to accelerate consolidation.

b. Embankment Consolidation. Significant excess pore pressures can develop during construction of fills exceeding about 80 feet in height or for lower fills of plastic materials placed wet of optimum moisture. Dissipation of these excess pore pressures after construction results in settlement. For earth dams and other high fills where settlement is critical, construction pore pressures should be monitored by the methods of § 3.

c. Secondary Compression. Even for well-compacted embankments, secondary compression and shear strain can cause slight settlements after completion. Normally this is only of significance in high embankments, and can amount to between 0.1 and 0.2% of fill height in three to four years or between 0.3 and 0.6% in 15 to 20 years. The larger values are for fine-grained plastic soils.

9.2.5. Earth Dam Embankments

Evaluate stability at three critical stages; the end of construction stage, steady state seepage stage, and rapid drawdown stage. See § 8 for pore pressure distribution at these stages. Seismic forces must be included in the evaluation. Requirements for seepage cut-off and stability dictate design of cross section and utilization of borrow materials.

a. Seepage Control. Normally the earthwork of an earth dam is zoned with the least pervious, fine-grained soils in the central zone and coarsest, most stable material in the shell. Analyse seepage by the methods of 7.7.

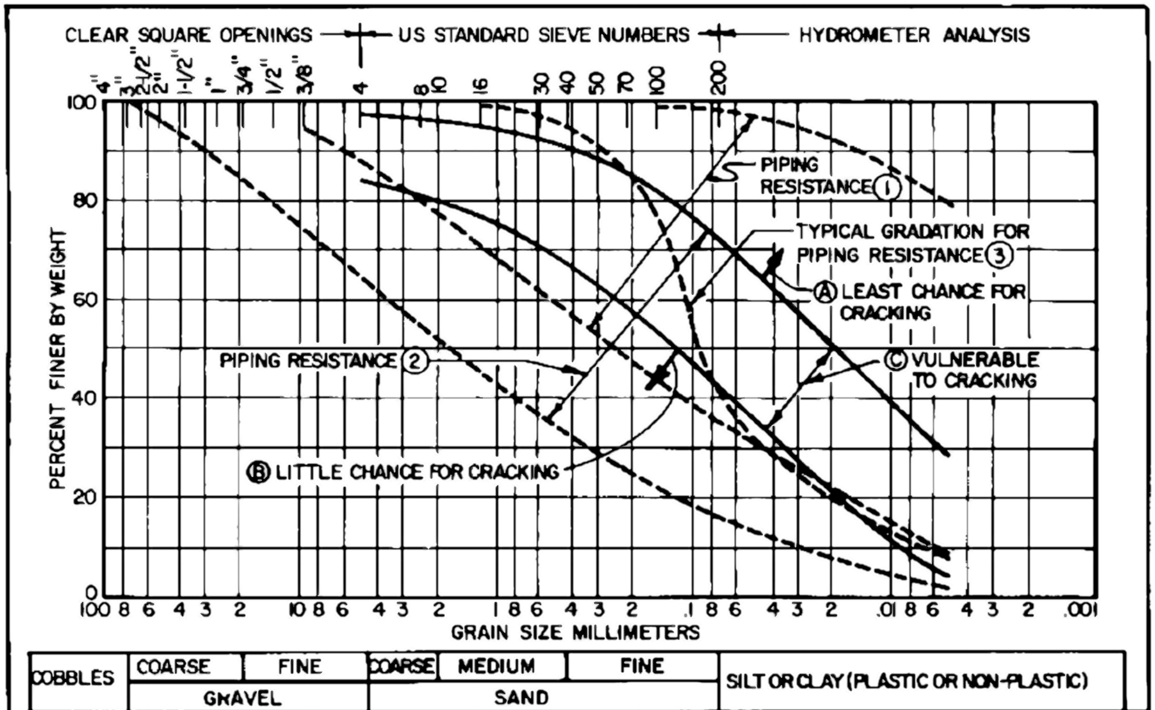
   1) Cutoff Trench. Consider the practicability of a positive cut-off trench extending to impervious strata beneath the embankment and into the abutments.

   2) Intercepting Seepage. For a properly designed and constructed zoned earth dam, there is little danger from seepage through the embankment. Drainage design generally is dictated by necessity for intercepting seepage through the foundation or abutments. Downstream seepage conditions are more critical for homogeneous fills. See 7.7 for drainage and filter requirements.

b. Piping and Cracking. A great danger to earth dams, particularly those of zoned construction, is the threat of cracking and piping. Serious cracking may result from tension zones caused by differences in stress-strain properties of zoned material. See Figure 9-1 for classification of materials according to resistance to piping or cracking. Analyse the embankment section for potential tension zone development. Place an internal drainage layer immediately downstream of the core to control seepage from possible cracking if foundation settlements are expected to be high.

c. Dispersive Soil. Dispersive clays should not be used in dam embankments. Determine the dispersion potential using Table 9-3. A hole through dispersive clay will increase in size as water flows through (due to the breakdown of the soil structure), whereas the size of a hole in a non-dispersive clay would remain essentially constant. Therefore, dams constructed with dispersive clays are extremely susceptible to piping.

Figure 9-1 Resistance of Earth Dam Embankment Materials to Piping and Cracking[255]



CATEGORY	MATERIAL	CHARACTERISTICS
PIPING RESISTANCE:		
①	CL AND CH WITH PI > 15, WELL GRADED SC WITH PI > 15.	GREATEST RESISTANCE TO PIPING, SMALL AND MEDIUM CONCENTRATED LEAKS WILL HEAL THEMSELVES. EMBANKMENT MAY FAIL AS A RESULT OF SLOWLY PROGRESSIVE PIPING CAUSED BY LEAK OF ABOUT 1/2 CFS.
②	CL AND ML WITH PI < 15, WELL GRADED SC AND GC WITH 15 > PI > 7.	INTERMEDIATE RESISTANCE TO PIPING. SAFELY RESISTS SATURATION OF LOWER PORTION OF DOWNSTREAM SLOPE INDEFINITELY. MAY FAIL EVENTUALLY AS A RESULT OF EROSION CAUSED BY A SMALL CONCENTRATED LEAK OR BY PROGRESSIVE SLOUGHING. IF A LARGE LEAK DEVELOPS, PIPING CAUSES FAILURE IN A SHORT TIME.
③	SP AND UNIFORM SM AND ML WITH PI < 7.	LEAST RESISTANCE TO PIPING. USUALLY FAILS IN A FEW YEARS AFTER FIRST RESERVOIR FILLING IF SEEPAGE IS ABLE TO BREAK OUT ON DOWNSTREAM SLOPE. SMALL CONCENTRATED LEAK ON DOWNSTREAM SLOPE CAN CAUSE FAILURE IN A SHORT PERIOD OF TIME. HIGH DENSITY FROM COMPACTION INCREASES RESISTANCE SIGNIFICANTLY.

255 Sherard, J.L., Influence of Soil Properties and Construction Methods on the Performance of Homogeneous Earth Dams, Technical Memorandum 645, U.S. Department of the Interior, Bureau of Reclamation.

CATEGORY	MATERIAL	CHARACTERISTICS
CRACKING RESISTANCE		
A	CH WITH D_{50} < 0.02 MM AND PI > 20	HIGH POSTCONSTRUCTION SETTLEMENT, PARTICULARLY IF COMPACTED DRY. HAS SUFFICIENT DEFORMABILITY TO UNDERGO LARGE SHEAR STRAINS FROM DIFFERENTIAL SETTLEMENT WITHOUT CRACKING.
B	GC, SC, SM, SP WITH D_{50} > 0.15 MM	SMALL POSTCONSTRUCTION SETTLEMENT. LITTLE CHANCE FOR CRACKING UNLESS POORLY COMPACTED AND LARGE SETTLEMENT IS IMPOSED ON EMBANKMENT BY CONSOLIDATION OF THE FOUNDATION.
C	CL, ML AND SM WITH PI < 20, 0.15 MM > D_{50} > 0.02 MM.	MEDIUM TO HIGH POSTCONSTRUCTION SETTLEMENT AND VULNERABLE TO CRACKING. SHOULD BE COMPACTED AS WET AS POSSIBLE CONSISTENT WITH STRENGTH REQUIREMENTS.

Table 9-3 Clay Dispersion Potential

Percent Dispersion[256]	*Dispersive Tendency*
Over 40	Highly Dispersive (do not use)
15 to 40	Moderately Dispersive
0 to 15	Resistant to Dispersion

9.3. Compaction Requirements and Procedures

9.3.1. Compaction Requirements

a. Summary. See Table 9-4 for a summary of compaction requirements of fills for various purposes. Modify these to meet conditions and materials for specific projects. "Modified Proctor Test" procedure referenced in the table can be found at 9.7.

b. Specification Provisions. Specify the desired compaction result. State the required density, moisture limits, and maximum lift thickness, allowing the contractor freedom in selection of compaction methods and equipment. Specify special equipment to be used if local experience and available materials so dictate.

[256] The ratio between the fraction finer than 0.005 mm in a soil-water suspension that has been subjected to a minimum fraction finer than 0.005 mm determined from a regular of mechanical agitation, and the total hydrometer test x 100.

Table 9-4 Compaction Requirements

Fill Utilized for:	Required Density, Percent of Modified Proctor	Tolerable Range of Moisture About Optimum, Percent	Maximum Permissible Lift Thickness, Compacted in.	Special Requirements
Support of structure	95	-2 to +2	12	Fill should be uniform. Blending or processing of borrow may be required. For plastic clays, investigate expansion under saturation for various compaction moisture and densities at loads equal to those applied by structure, to determine condition to minimize expansion. Clays that show expansive tendencies generally should be compacted at or above optimum moisture to a density consistent with strength and incompressibility required of the fill.
Lining for canal or small reservoir	90	-2 to +2	6	For thick linings, GW-GC, GC, SC are preferable for stability and to resist erosive forces. Single size silty sands with PI less than five generally are not suitable. Remove fragments larger than 6 inches before compaction.
Earth dam greater than 50 ft. high	95	-1 to +2	12(+)	Utilize least pervious materials as central core and coarest materials in outer shells. Core should be free of lenses, pockets, or layers of pervious material and successive lifts well bonded to each other. Amounts of oversize exceeding 1 percent of total material should be removed from the borrow prior to arrival on the embankment.
Earth dam less than 50 ft. high	92	-1 to +3	12(+)	In small dams that lack elaborate zoning, materials that are the most vulnerable to cracking and piping should be compacted to 98 percent density at moisture content from optimum to 3 percent in excess of optimum.
Support of pavements:				Place coarest borrow materials at top of fill. Investigate expansion of plastic clays placed near pavement subgrade to determine compaction moisture and density that will minimize expansion and provide required soaked CBR values.
Highways.....	See NAVFAC DM-5	-2 to +2	8(+)	
Airfields....	See NAVFAC DM-21	-2 to +2	8(+)	
Backfill surrounding structure	90	-2 to +2	8(+)	Where backfill is to be drained, provide pervious coarse-grained soils. For low walls, do not permit heavy rolling compaction equipment to operate closer to the wall than a distance equal to about 2/3 the unbalanced height of fill at any time. For highwalls or walls of special design, evaluate the surcharge produced by heavy compaction equipment by the methods of Chapter 3 and specify safe distances back of the wall for its operations.

Fill Utilized for:	Required Density, Percent of Modified Proctor	Tolerable Range of Moisture About Optimum, Percent	Maximum Permissible Lift Thickness, Compacted in.	Special Requirements
Backfill in pipe or utility trenches	90	-2 to +2	8(+)	Material excavated from trench generally is suitable for backfill if it does not contain organic matter or refuse. If backfill is fine grained, a cradle for the pipe is formed in natural soil and backfill placed by tamping to provide the proper bedding. Where free draining sand and gravel is utilized, the trench bottom may be finished flat and the granular material placed saturated under and around the pipe and compacted by vibration.
Drainage blanket or filter	90	Thoroughly wetted	8	Ordinarily vibratory compaction equipment is utilized. Blending of materials may be required for homogeneity. Segregation must be prevented in placing and compaction. For compaction adjacent to and above drainage pipe, use hand tamping or light travelling vibrators.
Subgrade of excavation for structure	95	-2 to +2	-	For uniform bearing or to break up pockets of frost susceptible material, scarify the upper 8 to 12 in. of the subgrade, dry or moisten as necessary and recompact. Certain materials, such as heavily preconsolidated clays which will not benefit by compaction, or saturated silts and silty fine sands that become quick during compaction, should be blanketed with a working mat of lean concrete or coarse grained material to prevent disturbance or softening. Depending on foundation conditions revealed in exploration, a substantial thickness of loose soils may have to be removed below subgrade and recompacted, or compacted in place by vibration, or pile driving.
Rock fill	-	Thoroughly wetted	2 to 3 ft.	For fill containing sizes no larger than ft., place in layers not exceeding 24 in., thoroughly wetted and compacted by travel or heavy crawler tractors in spreading. Material with sizes up to 2 ft. may be placed in 3 ft lifts. Placing should be such that the maximum size of rock increases toward the outer slopes. Rocks larger than 1 cu yd in volume should be embedded on the slope.

Notes:

1. Density and moisture content refer to "Modified Proctor" test values, (ASTM D 1557)
2. Generally, a fill compacted dry of OMC will have higher strength and a lower compressibility even after saturation.
3. Compaction of "Coarse-grained, granular soil" is not sensitive to moisture content so long as bulking moisture is avoided. Where practicable, they should be placed saturated and compacted by vibratory methods.

9.3.2. Compaction Methods and Equipment

Table 9-5 lists commonly used compaction equipment with typical sizes and weights and guidance on use and applicability.

Table 9-5 Compaction Methods and Equipment

Equipment Type	Applicability	Requirements for Compaction of 95 to 100 Percent Standard Proctor Maximum Density			Possible Variations in Equipment
		Compacted Lift Thickness, in.	Passes or Coverages	Dimensions and Weight of Equipment	
Sheepsfoot Rollers	For fine-grained soils or dirty coarse-grained soils with more than 20 percent passing No. 200 sieve. Not suitable for clean coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important.	6	4 to 6 passes for fine-grained soil. 6 to 8 passes for coarse-grained soil.	Soil Type / Foot Contact Area sq. ft. / Foot Contact Pressures psi Fine-grained soil PI>30: 5 to 12 / 250 to 500 Fine-grained soil PI<30: 7 to 14 / 200 to 400 Coarse-grained soil: 10 to 14 / 150 to 250 Efficient compaction of soils wet of optimum requires less contact pressure than the same soils at lower moisture contents.	For earth dam, highway and airfield work, articulated self propelled rollers are commonly used. For smaller projects, towed 40 to 60 inch drums are used. Foot contact pressure should be regulated so as to avoid shearing the soil on the third or fourth pass.
Rubber Tire Roller	For clean, coarse-grained soils with 4 to 8 percent passing the No. 200 sieve.	10	3 to 5 coverages	Tire inflation pressures of 35 to 130 psi for clean granular material or base course and subgrade compaction. Wheel load 18,000 to 25,000 lbs.	Wide variety of rubber tire compaction equipment is available. For cohesive soils, light-wheel loads, such as provided by wobble-wheel equipment, may be substituted for heavy-wheel load if lift thickness is decreased. For granular soils, large-size tires are desirable to avoid shear and rutting.
Do.......	For fine-grained soils or well graded, dirty coarse-grained soils with more than 8 percent passing the No. 200 sieve.	6 to 8	4 to 6 coverages	Tire inflation pressures in excess of 65 psi, for fine-grained soils of high plasticity. For uniform clean sands or silty fine sands, use large size tires with pressures of 40 to 50 psi.	
Smooth Wheel Rollers	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures.	8 to 12	4 coverages	Tandem type rollers for base course or subgrade compaction 10 to 15 ton weight, 300 to 500 lbs per lineal in. of width of rear roller.	3-wheel rollers obtainable in wide range of sizes. 2-wheel tandem rollers are availabe in the range of 1 to 20 ton weight. 3-Axle tandem rollers are generally used in the range of 10 to 20 tons weight. Very heavy rollers are used for proof rolling of subgrade or base course.
Do....	May be used for fine-grained soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands.	6 to 8	6 coverages	3-wheel roller for compaction of fine-grained soil; weights from 5 to 6 tons for materials of low plasticity to 10 tons for materials of high plasticity.	

Equipment Type	Applicability	Requirements for Compaction of 95 to 100 Percent Standard Proctor Maximum Density			Possible Variations in Equipment
		Compacted Lift Thickness, in.	Passes or Coverages	Dimensions and Weight of Equipment	
Vibrating Sheetsfoot Rollers	For coarse-grained soils sand-gravel mixtures	8 to 12	3 to 5	1 to 20 tons ballasted weight. Dynamic force up to 20 tons.	May have either fixed or variable cyclic frequency.
Vibrating Smooth Drum Rollers	For coarse-grained soils sand-gravel mixtures - rock fills	6 to 12 (soil) to 36 (rock)	3 to 5 4 to 6	- do -	- do -
Vibrating Baseplate Compactors	For coarse-grained soils with less than about 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	8 to 10	3 coverages	Single pads or plates should weigh no less than 200 lbs. May be used in tandem where working space is available. For clean coarse-grained soil, vibration frequency should be no less than 1,600 cycles per minute.	Vibrating pads or plates are available, hand-propelled, single or in gangs, with width of coverage from 1-1/2 to 15 ft. Various types of vibrating-drum equipment should be considered for compaction in large areas.
Crawler Tractor	Best suited for coarse-grained soils with less than 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	6 to 10	3 to 4 coverages	Vehicle with "Standard" tracks having contact pressure not less than 10 psi.	Tractor weight up to 85 tons.
Power Tamper or Rammer	For difficult access, trench backfill. Suitable for all inorganic soils.	4 to 6 in. for silt or clay, 6 in. for coarse-grained soils.	2 coverages	30-lb minimum weight. Considerable range is tolerable, depending on materials and conditions.	Weights up to 250 lbs., foot diameter 4 to 10 in.

9.3.3. Influence of Material Type

a. Soils Insensitive to Compaction Moisture. Coarse-grained, granular well-graded soils with less than 4% passing No. 200 sieve (8% for soil of uniform gradation) are insensitive to compaction moisture. (These soils have permeability greater than about 2 x 10^{-3} fpm.) Place these materials at the highest practical moisture content, preferably saturated. Vibratory compaction generally is the most effective procedure. In these materials, 70 to 75% relative density can be obtained by proper compaction procedures. If this is substantially higher than Standard Proctor maximum density, use relative density for control. Gravel, cobbles and boulders are insensitive to compaction moisture. Compaction with smooth wheel vibrating rollers is the most effective procedure. Use large-scale tests.[257]

b. Soils Sensitive to Compaction Moisture. Silts and some silty sands have steep moisture-density curves, and field moisture must be controlled within narrow limits for effective compaction. Clays are sensitive to moisture in that if they are too wet, they are difficult to dry to optimum moisture, and if they are dry, it is difficult to mix the water in uniformly. Sensitive clays do not respond to compaction because they lost strength upon remoulding or manipulation.

c. Effect of Oversize. Oversize refers to particles larger than the maximum size allowed using a given mould (i.e. No. 4 for 4” mould, 3/4 inch for 6” mould, 2” for a 12” mould). Large size particles interfere with compaction of the finer soil fraction. For normal embankment compaction, the maximum size cobble should not exceed 3 inches or 50% of the compacted layer thickness. Where economic borrow sources contain larger sizes, compaction trials should be run before approval.

d. There are two approaches to perform compaction tests with oversize particles in the fill:

   1. Adjust laboratory maximum standard density (from moisture-density relations test, see 9.7) to provide a reference density to which field density test results (with oversize) can be compared. Use the following equations to adjust the laboratory maximum dry density and optimum moisture content to values to which field test data (with oversize particles) may be compared.

Equation 9-1: $$\gamma_{max} = \frac{1 - 0.05F}{\frac{F}{162} + \frac{1-F}{\gamma_1}}$$

where:

- γ_{max} = adjusted maximum dry density pcf
- γ_1 = laboratory maximum dry density without oversize, pcf
- F = fraction of oversize particles by weight (from field density test)

Equation 9-2: $$w_j = F(w_g) + (1 - F)w_o$$

where:

- w_j = adjusted optimum moisture content
- w_g = moisture content of oversize (from field data)
- w_o = laboratory optimum moisture content without oversize

The density of oversize material is assumed as 162 pcf, obtained from bulk specific gravity 2.60, multiplied by 62.4.

[257] Gordon, B.B., Miller, R.K., Control of Earth and Rockfill for Oroville Dam, Journal of the geotechnical engineering Division, ASCE, Vol. 92, No. SM3, 1966.

This method is considered suitable when the weight of oversize is less than 60% by weight, for well-graded materials. For poorly graded materials, further adjustment may be appropriate.[258]

2. Perform the Compaction Test for Earth-Rock Mixtures as shown in 9.8 in lieu of the test shown in 9.7.

9.4. Embankment Compaction Control

9.4.1. Ground Preparation

1) Strip all organics and any other detrimental material from the surface. In prairie soils this may amount to removal of 2 or 3 inches of topsoil, and in forest covered land between 2 and 5 or more feet. (Only the heavy root mat and the stumps need be removed, not the hair-like roots.)
2) Remove subsurface structures or debris that will interfere with the compaction or the specified area use.
3) Scarify the soil, and bring it to optimum moisture content.
4) Compact the scarified soil to the specified density.

9.4.2. Field Test Section

By trial, develop a definite compaction procedure (equipment, lift thickness, moisture application, and number of passes) that will produce the specified density. Spot testing cannot control compaction adequately unless a well-defined procedure is followed.

9.4.3. Requirements for Control Tests

Perform in-place field density tests plus sufficient laboratory moisture-density tests to evaluate compaction. For high embankments involving seepage, settlement, or stability, perform periodic tests for engineering properties of density test samples, e.g., permeability tests, shear strength tests. See 9.7 for laboratory moisture density test procedures and § 3 for field density test methods.

a. Number of Field Density Tests. Specify the following minimum test schedule:
   1) One test for every 500 cu yd of material placed for embankment construction.
   2) One test for every 500 to 1,000 cu yd of material for canal or reservoir linings or other relatively thin fill sections.
   3) One test for every 100 to 200 cu yd of backfill in trenches or around structures, depending upon total quantity of material involved.
   4) At least one test for every full shift of compaction operations on mass earthwork.
   5) One test whenever there is a definite suspicion of a change in the quality of moisture control or effectiveness of compaction.
b. Field Density Test Methods. See § 3 for field density test methods. Proofrolling (spotting soft spots with a rubber-tired roller or any loaded earth-moving equipment) may be used in conjunction with density testing, but is practical only for extensive earthwork or pavement courses.

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[258] This method is modified after that described in McLeod, N.W., Suggested Method for Correcting Maximum Density and Optimum Moisture Content of Compacted Soils for Oversize Particles, Special Procedures for Testing Soil and Rock for Engineering Purposes, ASTM STP 479, ASTM, 1970.; also see Donaghe, R.T., and Townsend, F.C., Scalping and Replacement Effects on the Compaction Characteristics of Earth-Rock Mixtures, Soil Specimen Preparation for Laboratory Testing, ASTM STP 599, ASTM, 1976.

c. Laboratory Compaction Tests. Prior to important earthwork operations, obtain a family of compaction curves representing typical materials. Ideally, this family will form a group of parallel curves and each field density test will correspond to a specific compaction curve. During construction, obtain supplementary compaction curves on field density test samples, approximately one for every 10 or 20 field tests, depending on the variability of materials.

9.4.4. Analysis of Control Test Data

Compare each field determination of moisture and density with appropriate compaction curve to evaluate conformance to requirements.

a. Statistical Study. Overall analysis of control test data will reveal general trends in compaction and necessity for altering methods. Inevitably, a certain number of field determinations will fall below specified density or outside specified moisture range. Tabulate field tests, noting the percentage difference between field density and laboratory maximum density and between field moisture and optimum.

b. Moisture control. Close moisture control is evidenced if two-thirds of all field values fall in a range ± 1% about the median moisture content specified. Erratic moisture control is evidenced if approximately two-thirds of all field values fall in a range ± 3% about the median moisture content specified. To improve moisture control, blend materials from wet and dry sections of borrow area.

c. Compactive Effort. Suitable compaction methods are being utilized if approximately two-thirds of all field densities fall in a range of ± 3% about the percent maximum density required. Insufficient or erratic compaction is evidenced if approximately two-thirds of all field values fall in a range of ± 5% about the percent maximum density required. To improve compaction, consider methods for more uniform moisture control, alter the number of coverages, weights, or pressures of compaction equipment.

d. Overcompaction. A given compactive effort yields a maximum dry density and corresponding optimum moisture content. If the compactive effort is increased, the maximum dry density increases but the corresponding optimum moisture content decreases. Thus, if the compactive effort used in the field is higher than that used in the laboratory for establishing the moisture density relationship, the soil in the field may be compacted above its optimum moisture content, and the strength of the soil may be lower even though it has been compacted to higher density. This is of particular concern for high embankments and earth dams.[259]

9.4.5. Indirect Evaluation of Compaction in Deep Fills

The extent of compaction accomplished is determined by comparing the results from standard penetration tests and cone penetration tests before and after treatment (§ 3).

9.4.6. Problem Soils

The compaction of high volume change soils requires special treatment. See § 3.

[259] For further guidance, see Turnbull, W.J. and Foster, C.R., Stabilization of Materials by Compaction, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 82, No. SM2, 1956.

9.5. Borrow Excavation

9.5.1. Borrow Pit Exploration

The number and spacing of borings or test pits for borrow exploration must be sufficient to determine the approximate quantity and quality of construction materials within an economical haul distance from the project. For mass earthwork, initial exploration should be on a 200-foot grid. If variable conditions are found during the initial explorations, intermediate borings or test pits should be done. Explorations should develop the following information:

1. A reasonably accurate subsurface profile to the anticipated depth of excavation.
2. Engineering properties of each material considered for use.
3. Approximate volume of each material considered for use.
4. Water level.
5. Presence of salts, gypsums, or undesirable minerals.
6. Extent of organic or contaminated soils, if encountered.

9.5.2. Excavation Methods

a. Equipment. Design and efficiency of excavation equipment improves each year. Check various construction industry publications for specifications.

b. Ripping and Blasting. Determine rippability of soil or rock by borings (RQD and core recovery, see § 2 and § 3), geophysical exploration, and/or trial excavation.

9.5.3. Utilization of Excavated Materials

In the process of earthmoving there may be a reduction of the volume ("shrinkage") because of waste and densification, or an increase of volume ("swell") in the case of rock or dense soils, because the final density is less than its original density.

a. Borrow Volume. Determine total borrow volume, V_B required for compacted fill as follows:

Equation 9-3: $$V_B = \frac{\gamma_F}{\gamma_B} V_F + \frac{W_L}{\gamma_B}$$

Where:

- γ_F = dry unit weight of fill
- γ_B = dry unit weight of borrow
- V_F = required fill volume
- W_L = weight loss in stripping, waste, oversize and transportation

1. Compacted Volume. The volume of borrow soil required should be increased according to the volume change indicated above. A "shrinkage" factor of 10 to 15% may be used for estimating purposes.
2. Exclusions. A large percentage of cobble size material will increase the waste, because sizes larger than 3 inches are generally excluded from compacted fill.

b. Rock Fill

1. Maximum Expansion. Maximum expansion ("swell") from *in situ* conditions to fill occurs in dense, hard rock with fine fracture systems that breaks into uniform sizes. Unit volume in a quarry will produce approximately 1.5 volumes in fill.
2. Minimum Expansion. Minimum expansion occurs in porous, friable rock that breaks into broadly graded sizes with numerous spalls and fines. Unit volume in quarry will produce approximately 1.1 volumes in fill.

9.6. Hydraulic and Underwater Fills

9.6.1. General

Where large quantities of soil must be transported and ample water is available, hydraulic methods are economical. The choice of methods for placing hydraulic fill is governed by the type of equipment available, accessibility of borrow, and environmental regulations; see Table 9-6. Removal or placement of soil by hydraulic methods must conform to applicable water pollution control regulations.

Table 9-6 Methods of Fill Placement Underwater[260]

Methods	Characteristics
Bottom-dump scows	1. Limited to minimum depths of about 15 ft. because of scow and tug drafts. 2. Rapid; quick discharge entraps air and minimizes segregation.
Deck scows	1. Usable in shallow water. 2. Unloading is slow, by dozer, clamshell, or hydraulic jets. 3. Inspection of material being placed may be difficult.
Dumping at land edge of fill and pushing material into water by bulldozer.	1. Fines in material placed below water tend to separate and accumulate in front of advancing fill. 2. Work arrangement should result in central portion being in advance of side portions to displace sideways any soft bottom materials. 3. In shallow water, bulldozer blade can shove materials downward to assist displacement of soft materials.

9.6.2. Placement Methods

Placement, either under water or on land, should be done in a manner that produces a usable area with minimum environmental impact.

a. Deep Water Placement (over 75 feet). Most deep-water placement is by bottom dump scows and is unconfined, with no control on turbidity, except by the rate of dumping.

[260] Johnson, S.J., Compton, J.R., and Ling, S.C., Control for Underwater Construction, Underwater Sampling, Testing, and Construction Control, ASTM STP 501, ASTM, 1972.

b. Shallow Water Placement. Placement by pipeline, by mechanical equipment, or by side dumping from deck scows is the most common method in shallow water. Sheet pile containment, silt "curtains", or dikes are required to minimize lateral spreading and environmental impact. Where lateral spreading is not desired and steeper side slopes are needed, control the method of placement or use a mixed sand and gravel fill material. With borrow containing about equal amounts of sand and gravel, underwater slopes as steep as 1:3 or 1:2-3/4 may be achieved by careful placement. To confine the fill, provide berms or dikes of the coarsest available material or stone on the fill perimeter. Where rock is placed underwater, sluice voids with sand to reduce compressibility and possible loss of material into the rock.

c. Land Placement. On land, hydraulic fills are commonly placed by pipeline or by mechanical procedures (i.e. clam shell, dragline, etc.). Dikes with adjustable weirs or drop inlets to control the quality of return water are used for containment.

9.6.3. Performance of Hydraulic Fills

a. Coarse-Grained Fills. The most satisfactory hydraulically placed fills are those having less than 15% non-plastic fines or 10% plastic fines because they cause the least turbidity during placement, drain faster, and are more suitable for structural support than fine-grained material. Relative densities of 50 to 60% can be obtained without compaction. Bearing values are in the range of 500 to 2000 pounds per square foot depending on the level of permissible settlement. Density, bearing and resistance to seismic liquefaction may be increased substantially by vibroprobe methods.

b. Fine-Grained Fills. Hydraulically placed, bottom silts and clays such as produced by maintenance dredging will initially be at very high water contents. Depending on measures taken to induce surface drainage, it will take approximately 2 years before a crust sufficient to support light equipment is formed and the water content of the underlying materials approaches the liquid limit. Placing 1 to 3 feet of additional granular borrow will improve these areas rapidly so that they can support surcharge fills, with or without vertical sand drains to accelerate consolidation. Care must be exercised in applying the surcharge so that the shear strength of the soil is not exceeded.

9.6.4. Consolidation of Hydraulic Fills

If the coefficient of permeability of a hydraulic fill is less than 0.002 feet per minute, the consolidation time for the fill will be long and prediction of the behaviour of the completed fill will be difficult. For coarse-grained materials, fill consolidation and strength build-up will be rapid and reasonable strength estimates can be made. Where fill and/or foundation soils are fine-grained, it may be desirable to monitor settlement and pore water pressure dissipation if structures are planned. Settlement plates may be placed both on the underlying soil and within the fill to observe settlement rates and amounts.

9.7. Compaction Tests

9.7.1. Introduction

In the laboratory compaction test, a soil at a known water content is placed in a specified manner in a mould of given dimensions and subjected to a compactive effort of controlled magnitude after which the resulting unit weight of the soil is determined. The procedure is repeated at various water contents until a relation between water content and unit weight of the soil is established.

The laboratory compaction procedure is intended to simulate the compactive effort anticipated in the field. Generally, the *standard compaction test* shall be used to simulate field compaction for routine foundation and embankment design. In special cases, to suit anticipated construction procedures, it may be necessary to use higher or lower compactive efforts on the soil. For a higher compactive effort the *modified compaction test* and for a lower compactive effort the *15-blow compaction test* shall be used. Details of the standard, modified, and 15-blow compaction tests are given below.

9.7.2. Standard Compaction Test

9.7.2.1. Apparatus

The apparatus consists of the following:

(1) Moulds, cylindrical, metal. Moulds shall have a detachable base and a collar assembly extending approximately 2 1/2” above the top of the mould to retail soil during preparation of compacted specimens of the desired height and volume. Moulds having a slight taper to facilitate removal of the specimen after the compaction test are satisfactory provided the taper is no greater than 0.200” in diameter/foot of mould height. Capacities and dimensions of the moulds shall be as follows:

  a) Mould with an average inside diameter of 4.0” ± 0.016” and a capacity of 1/30 ± 0.0004 ft^3. A typical mould is shown in Figure 9-2 assembled and Figure 9-3.

  b) Mould with an average inside diameter of 6.0” ± 0.016” and a capacity of 3/40 ± 0.0004 ft^3. The 6.0” mould may be similar in construction to that shown in Figure 9-2, and shall be used for compacting samples containing material that would be retained on the No. 4 sieve but passing the 3/4” sieve.

  c) The exact volume of moulds should be determined before use and periodically thereafter, and this measured volume is used in calculations.

Figure 9-2 4” Diameter Compaction Mould

Figure 9-3 Disassembled Compaction Mould

(2) Rammer, manually or mechanically operated. The rammer shall consist of a drop weight that can be released to fall freely and strike the soil surface. The height of drop shall be controlled so that the weight falls from a height of 12" ± 1/16" above the surface of the soil. The mass of the free falling part of the rammer shall be 5.5 ± 0.02 lbs. and the striking face of the rammer shall be flat. Rammers must also meet the following requirements:

a) Manual rammer. The striking face shall be circular with a diameter of 2.0" ± 0.005". The rammer shall be equipped with a guide sleeve having sufficient clearance so that the free fall of the rammer shaft and head will not be restricted. The guide sleeve shall have at least four vent holes at each end (eight holes total) located with centres 3/4 ± 1/16" from each end and space 90º apart. The minimum diameter of the vent holes shall be 3/8". Additional vent holes or slots may be incorporated in the guide sleeve if desired. Figure 9-4 illustrates the two rammers, the left for the modified test and the right for the standard test.

b) Mechanical rammer. A mechanical rammer must operate in such a manner as to provide uniform and complete coverage of the specimen surface. The clearance between the rammer and the inside surface of the mould at its smallest diameter shall be 0.10" ± 0.03". When used with the 4" mould, the specimen contact face shall be circular with a diameter of 2.000" ± 0.005". When used with the 6.0" mould, the specimen contact face shall be either circular or sector shaped[261]; if sector shaped, it shall have a radius of 2.90" ± 0.02". The sector face

[261] The mechanical rammer equipped with a sector shaped foot should not be used for compacting specimens for the California Bearing Ratio (CBR) test described in MIL-STD-621A as CBR values may differ substantially from those obtained on specimens compacted with a rammer having a circular foot.

rammer shall operate in such a manner that the vertex of the sector is positioned at the centre of the specimen.

c) Calibration of mechanical rammer compactors. The mechanical rammer compactor must be calibrated periodically against the results obtained with the manual rammer. The compactor must be calibrated for the circular foot and, if used, the sector foot. The mechanical compactor shall be calibrated before initial use, near the end of each period during which the mould was filled 500 times before use after anything including repairs that may affect test results whenever test results are questionable, and before use after any 6-month period during which the rammer was not calibrated.

Figure 9-4 Manual Rammers for Standard Compaction Test

(3) Balance having a readability of 1 g an accuracy of 2 g, and having a capacity sufficient for weighing compacted samples.

(4) Oven (see 3.4.1.1).

(5) Sieves, US Standard 3/4" and No. 4 (0.187") conforming to ASTM Designation: E 11, Standard Specification for Wire-Cloth Sieves for Testing Purposes. Large sieves are generally more suitable for this purpose.

(6) Straightedge, steel, at least 1/8" x 1 3/8" x 10" and having a beveled edge.

(7) Mixing tools, such as mixing pan, spoon, trowel, spatula, etc. A suitable mechanical device may be used for mixing fine-grained soils with water.

(8) Specimen containers. Seamless metal containers with lids are recommended. The containers should be of a metal resistant to corrosion such as aluminum or stainless steel. Containers 2" high by 3-1/2" in diameter are adequate

(9) Sample splitter or riffle for dividing the samples.

(10) Glass jars, metal cans, or plastic buckets with airtight lids in which to store and cure soil prepared for compaction.

(11) Equipment for determining water content.

9.7.2.2. Preparation of Sample

The amount of soil required for the standard compaction test varies with the kind and gradation of the soil to be tested. For soils passing the No. 4 sieve that are to be tested in the 4.0" mould, 20 lb of soil is normally sufficient for the test. For samples containing gravel that are to be tested in the 6" mould, approximately 75 lb of processed material is required. Ordinarily, the soil to be tested shall be air-dried, or dried by means of drying apparatus provided the apparatus will not raise the temperature of the sample above 60° C (140º F). The requirement for fully air-drying soils in preparation for compaction is intended to facilitate soil processing and reduce variability in testing procedures. However, in some construction control operations, it may not be practical to completely air dry, rewet, and cure the soil in preparation for compaction. In these instances, the soil is air-dried to some water content near the driest point on the compaction curve and water for preparation of individual test specimens added as needed to obtain the desired range of water contents. Partial air-drying of some soils during preparation may lead to compaction results different from those that would be obtained if the soil had been completely air-dried during preparation. If a procedure other than the standard (fully air-dry, rewet, and cure) procedure is used, comparison tests must be performed for each of the soil types encountered at a given project to verify that there is no differences in results. If differences in results do appear, a procedure that reflects the actual field conditions must be adopted for both design and construction control testing.

Aggregations present in the sample shall be thoroughly broken, but care should be taken that the natural size of the individual particles is not reduced. The material shall then be screened through a _" and a No. 4 sieve. For some soils, it may be desirable to reduce aggregations before the sample is dried. If the entire material passes the No. 4 sieve, the sample shall be mixed thoroughly and a representative sample taken to determine the initial water content. The sample shall then be stored in an airtight container until ready for processing at different water contents for compaction in the 4.0" mould.

If the entire sample passes the 3/4" sieve and contains 5% or less material larger than the No. 4 sieve, the plus No. 4 fraction shall be discarded and the test performed using the 4.0" compaction mould. If the entire sample passes the 3/4" sieve but contains more than 5% material retained on the No. 4 sieve, it shall be tested in the 6" mould. The sample shall be mixed thoroughly after which its initial water content shall be determined. The sample shall then be stored in an airtight container until ready for processing at different water contents for compaction.

If the sample contains some material retained on the 3/4" sieve, but the amount is 5% or less, the plus 3/4" fraction shall be removed and discarded and the sample tested in the 6" mould. The initial water content of the sample shall be determined and the sample stored in an airtight container until ready for processing at different water contents for compaction.

If the sample contains more than 5% material retained on the 3/4" sieve, the test should be performed using the 12" compaction mould.

9.7.2.3. Procedure

(1) Material finer than No. 4 sieve. The procedure for soils finer than the No. 4 sieve shall consist of the following:

a. Record all identifying information for the sample such as project name or number, boring number and other pertinent data on a data sheet (see Figure 11-28 for suggested form). Record the compactive effort to be used, size of mould, and initial water content of processed sample.

b. From the previously prepared sample, weigh a quantity of air-dry soil equivalent to 2.5 kg oven-dry weight (see paragraph 2d(1)). Thoroughly mix the material with a measured quantity of water sufficient to produce a water content 4 to 6 percentage points below estimated optimum water content. At this water, nonplastic soils tightly squeezed in the palm of the hand will form a cast which will withstand only slight pressure applied by the thumb and fingertips without crumbling; plastic soils will ball noticeable. Store the soil in an airtight container for a sufficient length of time to permit it to absorb the moisture. The time required for complete absorption will vary depending on the type of soil. For nonplastic soils in which moisture is readily absorbed, storage is not necessary. For most other soils, a minimum curing time of 16 hours is usually adequate.

c. Repeat step (b) for at least four additional specimens. Increase the water content for each specimen by approximately 2 percentage points over that of the previous specimen.

d. Weigh the 4.0" compaction mould to the nearest gram, and record the weight on the data sheet.

e. Attach the mould, with collar, to the base plate and place the mould on a uniform, rigid foundation, such as a block or cylinder of concrete weighing not less than 200 lb.

f. Place an amount of the previously prepared sample in the 4.0" mould such that when three such layers have been compacted in the mould, the total compacted height is between 4 5/8"and 5"[262] Compact each layer by 25 uniformly distributed blows from the rammer, with the drop weight falling freely from a height of 12.0". In operating the manual rammer, take care to hold the rammer vertical and avoid rebounding the rammer drop weight from the top of the guide sleeve. Apply the blows at a uniform rate not exceeding 1.4 set/blow. The compaction procedure is illustrated in Figure 9-5.

g. Remove the extension collar from the mould. Remove the exposed compacted soil with a knife and carefully trim the surface even with the top of the mould by means of a straightedge. Any cavities formed by large particles being pulled out should be carefully patched with material from the trimmings.

h. Remove the mould with the compacted specimen therein from the base plate, weigh the mould plus wet soil to the nearest gram, and record the weight on the data sheet. When

[262] It is important-that the compacted soil just fill the mould with little excess to be struck off. As the amount of material to be struck off varies, the mass of soil to which a constant amount of energy is supplied varies. When the amount of material to be struck off is more than about 1/4", the test results become less accurate.

cohesionless soils are being tested, there is a possibility of losing the sample if the base plate is removed. For these soils, weigh the entire unit.

i. Remove the compacted specimen from the mould, and slice it vertically through the centre. Take a representative specimen of the material from each of the two parts and determine the water content of each. The water content specimens shall weigh not less than 100 g. Alternatively, the entire compaction specimen may be used for the water content determination. In this case, the wet weight of specimen for use in computing water content should be redetermined after the specimen is extruded from the compaction mould as some loss of material may occur during transfer of the specimen.

j. Repeat steps (d) through (i) for remaining specimens. Compact a sufficient number of test specimens over a range of water contents to establish definitely the optimum water content and maximum density. Generally, five compacted specimens prepared according to the above-described procedure should completely define a compaction curve. However, sometimes more specimens are necessary. To determine if the optimum water content has been reached, compare the wet weights of the various compacted specimens. The optimum water content and maximum density have been reached if the wettest specimens compacted indicate a decrease in weight in relation to drier specimens.

Figure 9-5 Compacting Soil Specimen

(2) Materials larger than 3/4" sieve. The procedure for determining the density and optimum water content of soils containing material retained on the 3/4" sieve is the same as that for the finer than 3/4" sieve material, except that the test is performed in the 6" diameter mould and the number of

blows of the compaction rammer is 56 per soil layer instead of 25. This results in equal compactive efforts for the two moulds. It is advisable to use the entire compacted specimen for the water content determination. The quantity of soil required for each compacted sample will be equivalent to about 5.5 kg of oven-dry material.

9.7.2.4. Computations

(1) Preparation of specimen. The required weight of soil, W_0' in grams necessary to produce 2.5 kg of oven-dry soil is computed as follows:

Equation 9-4: $$W_o' = W_s' + \frac{w_o}{100}$$

Where

- w_0 = initial water content of material (after air-drying)
- W'_s = desired weight of oven-dry soil = 2,500 g

The amount of water, W_w, in cm^3, to be added to the weight of soil, W'_0) to produce specimens at the desired test water contents is computed as follows:

Equation 9-5: $$W_w = \frac{W_s'(w' - w_o)}{100}$$

Where w' = desired test water content.

(2) Quantities obtained in compaction test. The following quantities are obtained for each specimen in the compaction test:

a. Weight of compaction mould plus wet soil. The weight of the compaction mould is subtracted from this value to obtain the weight of the soil, W.

b. The inside volume of the compaction mould. This volume is equal to the volume, V, of the wet soil specimen.

c. Weight of water content specimen plus tare before and after oven drying. The tare weight is subtracted from these values to obtain the weight of wet and dry soils for computing water content.

(3) Water content and density. The water content, w, of each compacted specimen shall be computed in accordance with 3.4.1.4. The weight of oven-dry soil, W_s' of each compacted specimen shall be computed according to the formula:

$$\text{Dry Weight Of Specimen} = \frac{\text{Weight Of Wet Soil}}{1 + \frac{\text{Water Content}}{100}}$$

Equation 9-6:

$$W_s = \frac{W}{1 + \frac{w}{100}}$$

The dry weight of the specimen is obtained directly if the entire compacted specimen is used for the water content determination and no loss of material occurs during removal of the specimen from the mould.

The wet unit weight, γ_m, (optional) and the dry unit weight, γ_d, expressed in pounds/cubic foot, shall be computed by the following formulas:

Equation 9-7:

$$\text{Wet Unit Weight} = 62.4\frac{\text{Weight In Grams Of Wet Specimen}}{\text{Volume In Cubic Centimetres Of Wet Specimen}}$$

$$\gamma_m = 62.4\frac{W}{V}$$

Equation 9-8:

$$\text{Dry Unit Weight} = 62.4\frac{\text{Weight In Grams Of Oven} - \text{Dry Specimen}}{\text{Volume In Cubic Centimetres Of Wet Specimen}}$$

$$\gamma_d = 62.4\frac{W_s}{V}$$

These computations may be simplified by the use of a mold constant, C, computed as follows:

Equation 9-9: $C = \dfrac{62.4}{V}$

In this case, Equation 9-7 and Equation 9-8 reduce to

Equation 9-10: $\gamma_m = CW$

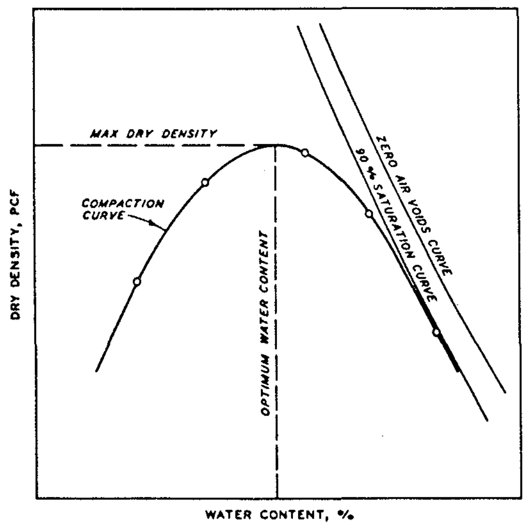
Equation 9-11: $\gamma_d = CW_s$

respectively.

9.7.2.5. Presentation of Results

(1) Compaction Curve. The results of the standard compaction test shall be presented in the form of a compaction curve on an arithmetic plot as shown in Figure 11-28. The dry densities in pounds/cubic foot are plotted as ordinates and the corresponding water contents in percentage of dry weight as abscissas. The plotted points shall be connected with a smooth curve; for most soils, the curve produced is generally parabolic in form. A typical compaction curve is shown in Figure 9-6. The water content corresponding to the peak of the compaction curve is the optimum water content, and this value shall be recorded to the nearest 0.1%. The dry unit weight of the soil in pounds/cubic foot at the optimum water content is the maximum dry density, and this value shall be recorded to the nearest 0.1 pcf.

Figure 9-6 Determination of Maximum Density and Optimum Water Content



(2) Air voids curve. The zero air voids curve (see example in Figure 9-6) represents the dry density and water content of a soil completely saturated with water. The zero air voids and 90% saturation curves shall be shown with the compaction curve in Figure 11-28. Data for plotting these curves for soils with different specific gravities are given in Table 9-7. The specific gravity of the soil used in the compaction test shall be determined in 3.4.3.

Table 9-7 Data for Zero Air Voids Curve

Specific Gravity of Soil	Water Content, w, in Per Cent of Dry Weight for Dry Unit Weight, γ_d, in pounds per cubic foot of																			
	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130	135	140	145
2.40	83.2	71.8	62.4	54.4	47.5	41.6	36.4	31.8	27.7	24.1	20.8	17.8	15.1	12.6	10.4	8.3	6.4	4.6	2.9	1.4
2.45	84.0	72.7	63.2	55.2	48.4	42.4	37.2	32.6	28.6	24.9	21.6	18.6	15.9	13.5	11.2	9.1	7.2	5.4	3.8	2.2
2.50	84.9	73.5	64.1	56.1	49.2	43.2	38.0	33.5	29.4	25.7	22.4	19.5	16.8	14.3	12.0	9.9	8.0	6.2	4.6	3.1
2.51	85.0	73.7	64.2	56.2	49.4	43.4	38.2	33.6	29.5	25.9	22.6	19.6	16.9	14.5	12.2	10.1	8.2	6.4	4.8	3.2
2.52	85.2	73.8	64.4	56.4	49.5	43.6	38.4	33.8	29.7	26.0	22.8	19.8	17.1	14.6	12.4	10.3	8.3	6.6	4.9	3.4
2.53	85.3	74.0	64.5	56.5	49.7	43.7	38.5	33.9	29.8	26.2	22.9	19.9	17.2	14.8	12.5	10.4	8.5	6.7	5.1	3.5
2.54	85.5	74.1	64.7	56.7	49.8	43.9	38.7	34.1	30.0	26.4	23.1	20.1	17.4	14.9	12.7	10.6	8.7	6.9	5.2	3.7
2.55	85.6	74.3	64.8	56.8	50.0	44.0	38.8	34.2	30.2	26.5	23.2	20.2	17.5	15.1	12.8	10.7	8.8	7.0	5.4	3.8
2.56	85.8	74.5	65.0	57.0	50.1	44.2	39.0	34.4	30.3	26.7	23.4	20.4	17.7	15.2	13.0	10.9	9.0	7.2	5.5	4.0
2.57	86.0	74.6	65.1	57.1	50.3	44.3	39.1	34.5	30.5	26.8	23.5	20.6	17.8	15.4	13.1	11.0	9.1	7.3	5.7	4.2
2.58	86.1	74.8	65.3	57.3	50.4	44.5	39.3	34.7	30.6	27.0	23.7	20.7	18.0	15.5	13.3	11.2	9.3	7.5	5.8	4.3
2.59	86.3	74.9	65.4	57.4	50.6	44.6	39.4	34.8	30.8	27.1	23.8	20.9	18.1	15.7	13.4	11.3	9.4	7.6	6.0	4.5
2.60	86.4	75.1	65.6	57.6	50.7	44.8	39.6	35.0	30.9	27.3	24.0	21.0	18.3	15.8	13.6	11.5	9.6	7.8	6.1	4.6
2.61	86.6	75.2	65.7	57.7	50.9	44.9	39.7	35.1	31.1	27.4	24.1	21.2	18.4	16.0	13.7	11.6	9.7	7.9	6.3	4.8
2.62	86.7	75.3	65.9	57.9	51.0	45.1	39.9	35.3	31.2	27.6	24.3	21.3	18.6	16.1	13.9	11.8	9.9	8.1	6.4	4.9
2.63	86.8	75.5	66.0	58.0	51.2	45.2	40.0	35.4	31.4	27.7	24.4	21.4	18.7	16.3	14.0	11.9	10.0	8.2	6.6	5.0
2.64	87.0	75.6	66.2	58.2	51.3	45.4	40.2	35.6	31.5	27.8	24.6	21.6	18.9	16.4	14.2	12.1	10.1	8.4	6.7	5.2
2.65	87.1	75.8	66.3	58.3	51.5	45.5	40.3	35.7	31.6	28.0	24.7	21.7	19.0	16.6	14.3	12.2	10.3	8.5	6.9	5.3
2.66	87.3	75.9	66.5	58.5	51.6	45.7	40.5	35.9	31.8	28.1	24.8	21.9	19.2	16.7	14.4	12.4	10.4	8.7	7.0	5.5
2.67	87.4	76.1	66.6	58.6	51.7	45.8	40.6	36.0	31.9	28.3	25.0	22.0	19.3	16.8	14.6	12.5	10.6	8.8	7.1	5.6
2.68	87.6	76.2	66.7	58.7	51.9	45.9	40.7	36.1	32.1	28.4	25.1	22.2	19.4	17.0	14.7	12.6	10.7	8.9	7.3	5.8
2.69	87.7	76.3	66.9	58.9	52.0	46.1	40.9	36.3	32.2	28.6	25.3	22.3	19.6	17.1	14.9	12.8	10.9	9.1	7.4	5.9
2.70	87.8	76.5	67.0	59.0	52.2	46.2	41.0	36.4	32.3	28.7	25.4	22.4	19.7	17.3	15.0	12.9	11.0	9.2	7.6	6.0
2.71	88.0	76.6	67.2	59.2	52.3	46.3	41.1	36.6	32.5	28.8	25.5	22.6	19.9	17.4	15.1	13.0	11.1	9.3	7.7	6.2
2.72	88.1	76.8	67.3	59.3	52.4	46.5	41.3	36.7	32.6	29.0	25.7	22.7	20.0	17.5	15.3	13.2	11.3	9.5	7.8	6.3
2.73	88.2	76.9	67.4	59.4	52.6	46.6	41.4	36.8	32.7	29.1	25.8	22.8	20.1	17.7	15.4	13.3	11.4	9.6	8.0	6.4
2.74	88.4	77.0	67.6	59.6	52.7	46.7	41.5	37.0	32.9	29.2	25.9	23.0	20.3	17.8	15.5	13.4	11.5	9.7	8.1	6.6
2.75	88.5	77.2	67.7	59.7	52.8	46.9	41.7	37.1	33.0	29.4	26.1	23.1	20.4	17.9	15.7	13.6	11.7	9.9	8.2	6.7
2.76	88.6	77.3	67.8	59.8	53.0	47.0	41.8	37.2	33.1	29.5	26.2	23.2	20.5	18.1	15.8	13.7	11.8	10.0	8.4	6.8
2.77	88.8	77.4	68.0	60.0	53.1	47.1	41.9	37.4	33.3	29.6	26.3	23.4	20.7	18.2	15.9	13.8	11.9	10.1	8.5	7.0
2.78	88.9	77.5	68.1	60.1	53.2	47.3	42.1	37.5	33.4	29.8	26.5	23.5	20.8	18.3	16.1	14.0	12.1	10.3	8.6	7.1
2.79	89.0	77.7	68.2	60.2	53.4	47.4	42.2	37.6	33.5	29.9	26.6	23.6	20.9	18.5	16.2	14.1	12.2	10.4	8.8	7.2
2.80	89.2	77.8	68.3	60.3	53.5	47.5	42.3	37.7	33.7	30.0	26.7	23.8	21.0	18.6	16.3	14.2	12.3	10.5	8.9	7.4
2.81	89.3	77.9	68.5	60.5	53.6	47.7	42.5	37.9	33.8	30.1	26.8	23.9	21.2	18.7	16.4	14.4	12.4	10.7	9.0	7.5
2.82	89.4	78.1	68.6	60.6	53.7	47.8	42.6	38.0	33.9	30.3	27.0	24.0	21.3	18.8	16.6	14.5	12.6	10.8	9.1	7.6
2.83	89.5	78.2	68.7	60.7	53.9	47.9	42.7	38.1	34.0	30.4	27.1	24.1	21.4	19.0	16.7	14.6	12.7	10.9	9.3	7.7
2.84	89.7	78.3	68.8	60.8	54.0	48.0	42.8	38.2	34.2	30.5	27.2	24.3	21.5	19.1	16.8	14.7	12.8	11.0	9.4	7.9
2.85	89.8	78.4	69.0	61.0	54.1	48.2	43.0	38.4	34.3	30.6	27.3	24.4	21.7	19.2	16.9	14.9	12.9	11.2	9.5	8.0
2.86	89.9	78.5	69.1	61.1	54.2	48.3	43.1	38.5	34.4	30.8	27.5	24.5	21.8	19.3	17.1	15.0	13.1	11.3	9.6	8.1
2.87	90.0	78.6	69.2	61.2	54.4	48.4	43.2	38.6	34.5	30.9	27.6	24.6	21.9	19.5	17.2	15.1	13.2	11.4	9.8	8.2
2.88	90.1	78.8	69.3	61.3	54.5	48.5	43.3	38.7	34.7	31.0	27.7	24.7	22.0	19.6	17.3	15.2	13.3	11.5	9.9	8.3
2.89	90.3	78.9	69.5	61.5	54.6	48.6	43.4	38.9	34.8	31.1	27.8	24.9	22.2	19.7	17.4	15.3	13.4	11.6	10.0	8.5
2.90	90.4	79.0	69.6	61.6	54.7	48.8	43.6	39.0	34.9	31.2	28.0	25.0	22.3	19.8	17.6	15.5	13.5	11.8	10.1	8.6
2.95	91.0	79.6	70.2	62.2	55.3	49.3	44.1	39.6	35.5	31.8	28.5	25.6	22.9	20.4	18.1	16.0	14.1	12.3	10.7	9.2
3.00	91.5	80.2	70.7	62.7	55.9	49.9	44.7	40.1	36.0	32.4	29.1	26.1	23.4	21.0	18.7	16.6	14.7	12.9	11.3	9.7
3.05	92.1	80.7	71.3	63.3	56.4	50.5	45.3	40.7	36.6	32.9	29.6	26.7	24.0	21.5	19.2	17.2	15.2	13.5	11.8	10.3
3.15	93.1	81.8	72.3	64.3	57.4	51.5	46.3	41.7	37.6	34.0	30.7	27.7	25.0	22.5	20.3	18.2	16.3	14.5	12.8	11.3

Note: Zero air voids curve equivalent to a degree of saturation, S , equal to 100 percent.

$$w = S\left(\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right)$$

where

w = water content, percent

S = degree of saturation, percent

γ_w = unit weight of water, lb per cu ft = 62.43

γ_d = dry unit weight of soil, lb per cu ft

G_s = specific gravity of soil solids

This equation may also be used to determine curves representing degrees of saturation other than 100 percent.

9.7.3. Modified Compaction Test

The modified compaction test differs from the standard test in that a greater compactive effort is used that results in higher maximum densities and lower optimum water contents. The apparatus, preparation of sample and procedure are the same as those used in the standard compaction test, with the following modifications:

9.7.3.1. Apparatus

The rammer shall consist of a 10.00-pound weight with an 18.0” free drop. If a mechanical rammer is used in performing these tests, the rammer must be calibrated separately for this test.

9.7.3.2. Procedure

The soil shall be compacted in five layers of equal thickness. The number of blows per layer shall be the same as for the standard compaction test: 25 blows per layer in the 4” diameter mould, and 56 blows per layer in the 6” diameter mould. The computations and presentation of results shall be the same as those used in the standard compaction test.

9.7.4. 15-Blow Compaction Test

The 15-blow compaction test differs from the standard compaction test in that a lesser compactive effort is used resulting in lower maximum densities and higher optimum water content.

The apparatus, preparation of samples and procedures shall be the same as those used in the standard compaction test (5.50 pound weight with a 12.0” free drop) with the following modifications:

a. The 6” mould shall not be used.

b. The number of blows per layer shall be 15. The computations and presentation of results shall be the same as those used in the standard compaction test.

9.7.5. Possible Errors

Following are possible errors that would cause inaccurate determinations of compaction curves for any compactive effort:

a. Aggregations of dried soil not completely broken.

b. Water not thoroughly absorbed into dried soil. Consistent results cannot be obtained unless the soil and water are complete mixed and sufficient time allowed for the soil to absorb the water uniformly.

c. Soil reused. Since some soils are affected by recompaction, fresh material must be used for each specimen. Recompaction tends to increase the maximum dry unit weight of some clays and, therefore, decrease the apparent optimum water content.

d. Insufficient number of range of water contents to define compaction curve accurately.

e. Improper foundation for compaction mould.

f. Incorrect volume of compaction mould used. The exact inside volume of each mould must be determined before being used.

g. Mechanical compactor not properly calibrated.

h. Human factors in the operation of hand rammer. Variations in results can be caused by not bringing the drop weight to a complete stop before releasing it to fall and compact the soil. If raising and releasing the rammer's drop weight is done too quickly, the drop weight will not be brought to rest before release. If the rammer is not held vertical during operation, the compactive effort will be reduced. The tendency to press the sleeve of the manual rammer into the soil specimen, the way the blows are distributed over the surface of the specimen, and other individual operator characteristics all tend to affect maintained within a laboratory; however, it is preferable that all specimens of a given test be compacted by the same person with the same rammer in one sitting.

i. Excessive variation in total depth of compacted specimen. The extension of the specimen into the collar of the mould should not exceed about 1/4", and care should be taken that each layer is nearly equal in weight.

j. Water content determination not representative of specimen. This error can be avoided by using the entire specimen for the water content determination.

9.8. Compaction Test for Earth-Rock Mixtures

9.8.1. Introduction

Previously, procedures are given for the standard effort compaction test using samples having particles finer than the 3/4" sieve size. The procedures outlined in this section are for the standard effort compaction test using material with particles larger than 3/4" and finer than 2" sieve sizes. This method should be used for testing material containing particles larger than 3/4" sieve sizes if these particles exceed 10% by weight of the total sample. If less than 5% by weight of the total sample is finer than the No. 200 sieve, maximum density should be determined by vibratory methods.

The test method outlined is comparable to the standard test in that a) the compactive effort applied is 12,300 ft-lb/ft^3 and (b) the equipment has been devised to maintain ratios between mould diameter, rammer diameter, and maximum particle size of the test specimen similar to those ratios used in the standard compaction test.

9.8.2. Apparatus

The apparatus shall consist of the following:

a. Cylindrical mold, with an ID of 12.0" ± 0.1", a height of 12.0" ± 0.1", and a detachable collar approximately 2 1/2" high. The mould and collar assembly should be constructed to fasten to a detachable base plate. Details of a typical assembly are shown in Figure 9-7.

b. Hand rammer, metal, of the sliding-weight, fixed-head type pith a 4" diameter face and a free-falling weight of 11.50 lb ± 0.05 lb. The rammer should be equipped with a guide such that the height of fall of the sliding weight is 24.00" ± 0.05". Details of a typical rammer are shown in Figure 9-8.

c. Balances sensitive to 0.1 lb with a capacity of 250 lb.

d. Oven, forced-draft type, 10 to 12 ft^3 capacity, automatically controlled to maintain a uniform temperature of 110º ± 5º C.

e. Pans, drying, of aluminum or other corrosion-resistant metal, with a capacity of at least 0.5 ft_. Roasting pans 18" by 24" by 4" are satisfactory.

f. Sieves, U. S. Standard, large diameter type, ranging from 4" openings to the No. 4 size, and a mechanical sieve shaker. Sieves with 3/8-, 1/2-, 3/4-, 1-, 1-1/2-, 2-, 3-, and 4" openings are normally required.

g. Containers, corrosion resistant, with a capacity of at least 1 ft_ and having airtight lids.

h. Shovel, hand, square-edged, and a mortar box having a capacity of at least 4 ft_.

i. Straightedge, steel, at least 16" long, 3/8" thick, and 1" wide with a beveled edge.

j. Graduates, hand scoop, trimming knife, wire brush, and rubber-head hammer.

Figure 9-7 12” Compactor Mold with Collar and Base Plate

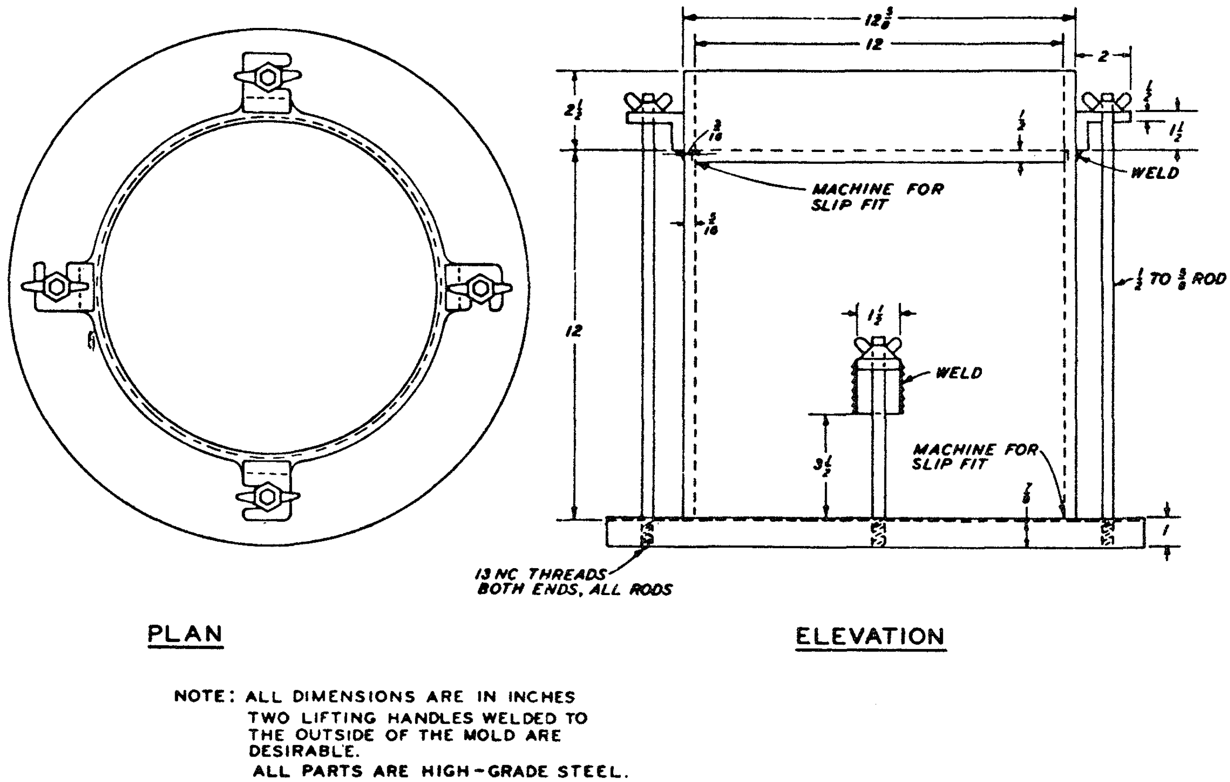
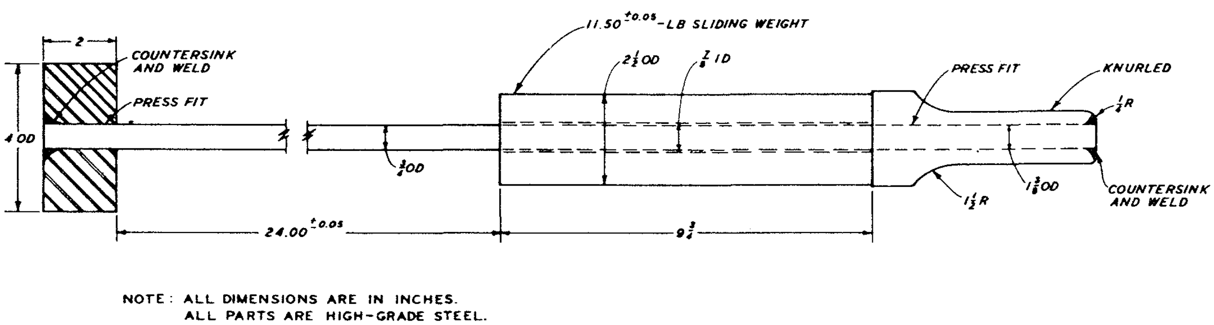


Figure 9-8 Compaction Rammer for 12” Mould



9.8.3. Quantity of Sample

At least 700 lb. of sample is required having particles finer than the 2” sieve sizes, If the field sample contains quantities of particles larger than the 2” sizes, the total sample weight required must be increased to permit removal of oversize particles.

9.8.4. Processing Of Sample

a. Record on a work sheet (Figure 11-29) identifying information for the sample, including visual classification.

b. Spread the material in flat pans and air-dry the entire sample. Other means, such as ovens and heat lamps, may be used to accelerate drying if the maximum drying temperature is 60º C.

c. Reduce all aggregates, or lumps formed during drying, of fine-grained material to particles finer than the No. 4 sieve.

d. With a wire brush or other means, remove all fine-grained material that may be clinging to rock sizes, taking care not to lose the fine-grained material. Separate all the material using a set of sieves ranging from the largest particle size in the sample to the No. 4 sieve. The total sample must be processed to determine the as-received gradation.

e. Place the material retained on each sieve and that passing the No. 4 sieve in separate containers, weigh the contents of each, and compute the percent of the total sample retained on each sieve as follows:

Equation 9-12: $$\text{Percent Retained} = 100\,\frac{\text{Dry Weight Of Material Retained On Sieve}}{\text{Dry Weight Of Total Sample}}$$

f. If 10% or less of a field sample is retained on the 2" sieve, the particles larger than this size should be discarded and replacement is not necessary.

g. If more than 10% of a field sample is retained on the 2" sieve, it will be necessary to remove the plus 2" sizes and replace them with an equal weight of material between the 2" and No. 4 sieve sizes. The gradation of the replacement material must be the same relative gradation as that of the total sample between the 2-in, and the No. 4 sieve sizes. The percent passing the No. 4 sieve remains constant and is equal to the percent passing the No. 4 sieve for the total as received sample. For each sieve between the 2" and the No. 4 sizes, the percent required to replace the plus 2-in, sizes is computed as follows:

Equation 9-13: $$\text{Replacement \%} = \text{Total \% Of} + 2\text{" Sizes}\,\frac{\text{\% Retained On One Sieve}}{\text{Total \% Between 2" And No. 4 Sieve}}$$ [263]

For each sieve, add the "Replacement %" to the "% Retained" on that sieve initially. This gives the percent by weight of a test specimen required for each sieve size in order to reconstitute a specimen with the +2" sizes replaced with sizes ranging from the 2" to the No. 4 sizes. Typical results are tabulated in Figure 11-29. A typical as-received gradation and test gradation is shown in Figure 11-30.

9.8.5. Special Considerations

In materials of a heterogeneous nature, such as mixtures of sandstones, siltstones, and shale, the large particles may be siltstone or sandstone, while the smaller size particles may be shale. For materials of this type, when particles larger than the 2" sieve sizes are removed for preparation of the test specimen, replacement must be made using the same types of materials "scalped off," or removed. For example, oversize sandstone particles must be removed and replaced, where applicable, with smaller particles of sandstone.

9.8.6. Preparation of Test Specimen

a. Prepare 130 lb of processed air-dried material for the test specimen by combining the weight of material required from each sieve size (refer to typical work sheet, Figure 11-29).

b. Thoroughly mix the material for the test specimen with a measured quantity of water sufficient to produce a water content 4 or 5 percentage points below the estimated optimum water content of the entire sample. This can be determined only by judgment and experience.

c. Store the moistened sample in an airtight container for a minimum of 16 hours.

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[263] Any sieve between 2" and No. 4 sieve sizes.

d. Prepare material for at least four additional test specimens by repeating steps in paragraphs 6a through 6c. Increase the water content of each specimen by approximately 2 percentage points over that of the previous specimen.

9.8.7. Compaction Procedure

a. Weigh the compaction mould to nearest 0.1 lb, determine its inside volume to the nearest 0.001 ft^3, record the data.

b. Attach the collar to the compactor mould, clamp the mould securely to the base plate and place the assembly on a level, rigid foundation made of a concrete cube having a total weight of at least 200 lb. This concrete foundation should not be covered with a metal plate.

c. Mix the cured material thoroughly to attain a uniform water distribution.

d. Place a sufficient quantity of the test specimen in the mould to give a compacted layer between 4.0 and 4.5" thick. Compact each layer by applying 140 uniformly distributed blows of the rammer, with the 11.5 pound weight falling freely from a height of 24.0" compact three equal layers in this manner, taking care to seat the rammer face flush with the soil surface before each blow and to keep the rammer assembly vertical during testing. Use just enough material to finish with less than 1" of sample protruding above the top of mould.

e. Detach the extension collar, taking care not to disturb the soil mass extending above the top of the mould. Trim the surface exactly even with the top of the mould. Fill any cavities formed by removal of particles during trimming with material from the trimmings and press this filling material firmly into place. Clean excess material from the lip of the mould.

f. Weigh the mould and compacted specimen to the nearest 0.1 lb and record the data.

g. Remove the entire test specimen from the mould, spread it in flat drying pans, and determine its water content. For most specimens, this requires at least 16 hours oven-drying time. Shorter drying times may be used if a constant weight is attained.

h. Repeat the steps in paragraphs c through g for a sufficient number of specimens over a range of water contents to establish the optimum water content and dry density. Five specimens will usually define the compaction curve accurately. Fresh material, not previously compacted, should be used for all tests.

i. For tests in which degradation of particles due to compaction is significant, determine the after-compaction gradation of at least two total specimens from each test series.

9.8.8. Computations

The computations shall consist of the following:

g. Compute the water content of each compacted specimen as follows:

Equation 9-14: $$\text{Water Content, W, \%} = 100\frac{W_w}{W_s}$$

Where

- W_w = wet weight of total specimen minus its oven-dry weight (lb)
- W_s = oven-dry weight of specimen (lb)

b. Compute the dry unit weight of each compacted specimen as follows:

Equation 9-15: $$\text{Dry Unit Weight}, \gamma, pcf = \frac{W_s}{V}$$

where V = volume of the compaction mould (ft^3).

9.8.9. Presentation of Results

Present the results of the test on Figure 11-28.

a. Compaction Curve. Plot the dry unit weight, in pounds/cubic foot, as the ordinate and the corresponding water contents, in percentages of dry weight, as the abscissa, on an arithmetic plot. Connect the plotted points with a smooth curve. The water content at the peak of the curve is the optimum water content and the corresponding dry unit weight is the maximum. Record the optimum water content to the nearest 0.1% and the maximum dry unit weight to the nearest 0.1 pcf.

b. Zero Air Voids and 90% Saturation Curves. Using the weighted average of the specific gravity of the plus No. 4 and the minus No. 4 material, compute and plot the zero air voids curve and the curve representing the line of 90% saturation.

9.8.10. Possible Errors

The following errors can cause inaccurate results:

a. Aggregations of air-dried soil not completely reduced to finer particles during processing.

b. Water not thoroughly absorbed into dried material due to insufficient mixing and curing time.

c. Material reused after compaction.

d. Insufficient number of tests to define compaction curve accurately.

e. Improper foundation for mould during compaction.

f. Incorrect volume or weight of compaction mould.

g. Incorrect rammer weight and height of fall.

h. Excessive material extending into the extension collar at the end of compaction.

i. Improper or insufficient distribution of blows over the soil surface.

j. Tendency to press the head of the rammer against the specimen before letting the weight fall.

k. Insufficient drying of sample for water content determination.[264]

[264] To eliminate the need for trimming away the excess material extending into the collar, an alternative may be used. Determine, using the sand-cone method, the volume of the empty collar above the top of the mould and the volume of the collar partially filled with the protruding specimen. The difference is the volume of the compacted excess material. Weigh the entire specimen without trimming the excess and determine the moist unit weight.

§ 10. Soil Dynamics

10.1. Introduction

10.1.1. Scope

This section is concerned with geotechnical problems associated with dynamic loads, and with earthquake related ground motion and soil response induced by earthquake loads. The dynamic response of foundations and structures depends on the magnitude, frequency, direction, and location of the dynamic loads; the geometry of the soil-foundation contact system; and the dynamic properties of the supporting soils and structures. Dynamic ground motions considered in this section are those generated from machine foundations and impact loading.

An example calculation of vertical, horizontal, and rocking motions induced by machinery vibration is included. Soil compaction resulting from dynamic impact and dynamic response induced by impact loading on piles are also included. An example calculation of dynamic compaction procedures for soils and an example of pile driving analysis are included.

Elements in a seismic response analysis are: input motions, site profile, static and dynamic soil properties, constitutive models of soil response to loading, and methods of analysis using computer programs. The contents include:

- Earthquake response spectra;
- Site seismicity;
- Soil response to seismic motion, design earthquake, seismic loads on structures, liquefaction potential, lateral spread from liquefaction, and foundation base isolation.

Some special problems in geotechnical engineering dealing with soil dynamics and earthquake aspects are discussed.

Its contents include: seismic design of anchored sheet pile walls, stone columns and displacement piles; and dynamic slope stability and deformations induced by earthquakes

10.2. Basic Dynamics

10.2.1. Vibratory Motions

Harmonic or sinusoidal motion is the simplest form of vibratory motion. An idealized simple harmonic motion may be described by the equation:

Equation 10-1: $z = A\sin(\omega t - \psi)$

where:

- z = displacement
- A = single amplitude
- _ = circular frequency
- t = time
- _ = phase angle

For simple harmonic motion the displacement amplitude, the phase angle, and the frequency are all that are needed to determine the complete history of motions. For motion other than harmonic motion, simple relationships usually do not exist between displacement, velocity, and acceleration, and the conversion from one quantity to the other must be accomplished by differentiation or integration of the equation of motion or by other mathematical manipulation.

The displacements described by the above equation will continue oscillating forever. In reality, the amplitude of the motions will decay over time due to the phenomenon called damping. If the damping is similar to that caused by a dashpot with constant viscosity, it is said to be linear viscous damping, and the amplitude decays exponentially with time. If the damping is similar to that caused by a constant coefficient of friction, it is said to be linear hysteric damping, and the amplitude decays linearly with time. All systems exhibit complicated combinations of various forms of damping, so any mathematical treatment is a convenient approximation to reality.

10.2.2. Mass, Stiffness, Damping

Dynamic analysis begins with a single-degree-of-freedom system illustrated in Figure 10-1 (A). A mass is attached to a linear spring and a linear dashpot. The sign convention is that displacements and forces are positive to the right.

If the mass M is accelerating to the right the force to cause this acceleration must be:

Equation 10-2: $$F_a = m\,a = m\,\frac{d^2u}{dt^2} = m\,\ddot{U}$$

The dots are used to indicate differentiation with respect to time; this simplifies writing the equations.

The linear dashpot has a restoring force that is proportional to the velocity of motion and acts in the opposite sense. This means that:

Equation 10-3: $$F_d = -c\,\frac{du}{dt} = -c\,\dot{U}$$

Finally, there may be some force P, which is a function of time, which is applied directly to the mass.

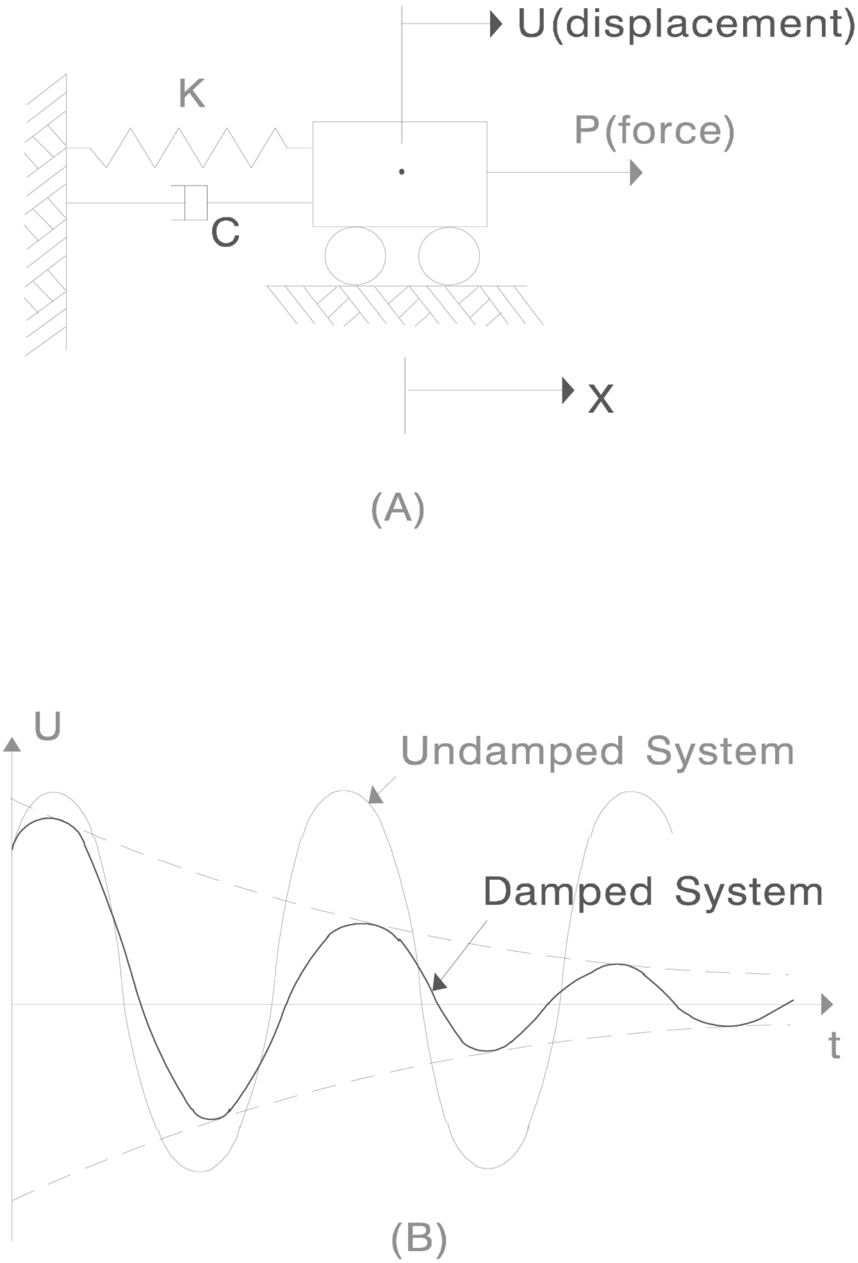
Adding the three forces together, setting the sum equal to mü, and rearranging terms gives the basic equation for an single degree of freedom system:

Equation 10-4: $$m\,\ddot{U} + c\,\dot{U} + k\,U = P$$

This equation applies to linear systems; for other types of systems, the equation has to be modified or the terms must be variable. In addition, when the motion involves rotation instead of translation, the displacements, velocities, and accelerations must be replaced with rotations, angular velocities, accelerations, and the other terms also modified appropriately.

In most practical cases, the mass m and the stiffness k can be determined physically. It is often possible to measure them directly. On the other hand, the damping is a mathematical abstraction used to represent the fact that the vibration energy does decay. It is difficult if not impossible to measure directly and, in some cases to be discussed below, it describes the effects of geometry and has nothing to do with the energy absorbing properties of the material.

Figure 10-1 Free Vibration of Simple System



In the case of no external force and no damping, the motion of the mass will be simple harmonic motion. The frequency _$_{o}$ will be:

$$\textbf{Equation 10-5: } \omega_o = \sqrt{\frac{k}{m}}$$

If the damping is not zero and the mass is simply released from an initial displacement U_o with no external force, the motion will be as shown in Figure 10-1 (B). The frequency of the oscillations will be ω_e:

$$\textbf{Equation 10-6: } \omega_e^2 = \omega_o^2 - \left(\frac{c}{2m}\right)^2$$

When c = $2(km)^{1/2}$, there will be no oscillations, but the mass will simply creep back to the at rest position at infinite time.

This is called critical damping, and it is written c_{cr}. The ratio of the actual damping to the critical damping is called the critical damping ratio D:

$$\textbf{Equation 10-7: } D = \frac{c}{c_{cr}}$$

If the basic equation is divided through by m, it can be written as:

$$\textbf{Equation 10-8: } \ddot{U} + 2D\,\omega_o\dot{U} + \omega_o^2 = \frac{P}{m}$$

The frequency of oscillations can be written:

$$\textbf{Equation 10-9: } \omega = \omega_o\sqrt{1 - D^2}$$

In almost all practical cases, D is much less than 1. For example, a heavily damped system might have a D of 0.2 or 20%. In that case, ω is 98% of ω_o, so little error is introduced by using the undamped frequency ω_o in place of the damped frequency ω.

10.2.3. Amplification Function

If now a sinusoidally varying force drives the single degree of freedom system, the right side of the basic equation becomes:

$$\textbf{Equation 10-10: } R = F\cos(\omega t)$$

For a very low frequency, this becomes a static load, and:

$$\textbf{Equation 10-11: } u = \frac{F}{k} = A_s$$

A_s is the static response.

In the dynamic case, after the transient portion of the response has damped out, the steady state response becomes:

$$\textbf{Equation 10-12: } u = M A_s \cos(\omega t - p)$$

In this equation, M is called the dynamic amplification factor and p is the phase angle. The dynamic amplification factor is the ratio of the amplitude of the dynamic steady-state response to the static response and describes how effectively the single degree of freedom amplifies or de-amplifies the input. The phase angle p indicates how much the response lags the input.

Mathematical manipulation reveals that:

Equation 10-13: $$M = \frac{1}{\sqrt{\left(1-\left(\frac{\omega}{\omega_o}\right)^2\right)^2 + \left(2D\frac{\omega}{\omega_o}\right)^2}}$$

And

Equation 10-14: $$P = \tan^{-1}\frac{2D\frac{\omega}{\omega_o}}{1-\left(\frac{\omega}{\omega_o}\right)^2}$$

The amplification factor M is plotted in Figure 10-7 (A). Note that the ratio of frequencies is the same regardless of whether they are expressed in radian/second or cycles/second.

When the problem involves rotating machinery, the amplitude of the driving force is proportional to the frequency of the rotating machinery. If e is the eccentricity of the rotating mass and m_e is its mass, then the amplitude of the driving force becomes:

Equation 10-15: $$F = m_e e\omega^2$$

In this case, the driving force vanishes when the frequency goes to zero, so it does not make sense to talk about a static response. However, at very high frequencies the acceleration dominates, so it is possible to define the high frequency response amplitude R:

Equation 10-16: $$R = m_e\left(\frac{e}{M}\right)$$

As in the case of the sinusoidal loading, the equations can be solved to give an amplification ratio. This is now the ratio of the amplitude of the response to the high-frequency response R. The curve is plotted in Figure 10-7 (B).

An important point is that the response ratio gives the amplitude of the displacement response for either case. To find the amplitude of the velocity response, the displacement response is multiplied by ω (or $2\pi f$). To find the amplitude of the acceleration response, the displacement response is multiplied by ω^2 (or $4\pi^2 f^2$).

10.2.4. Earthquake Ground Motions

Earthquake ground motions, which cause dynamic loads on the foundation and structures, are transient and may or may not occur several times during the design life of the structures.

10.3. Soil Properties

10.3.1. Soil Properties for Dynamic Loading

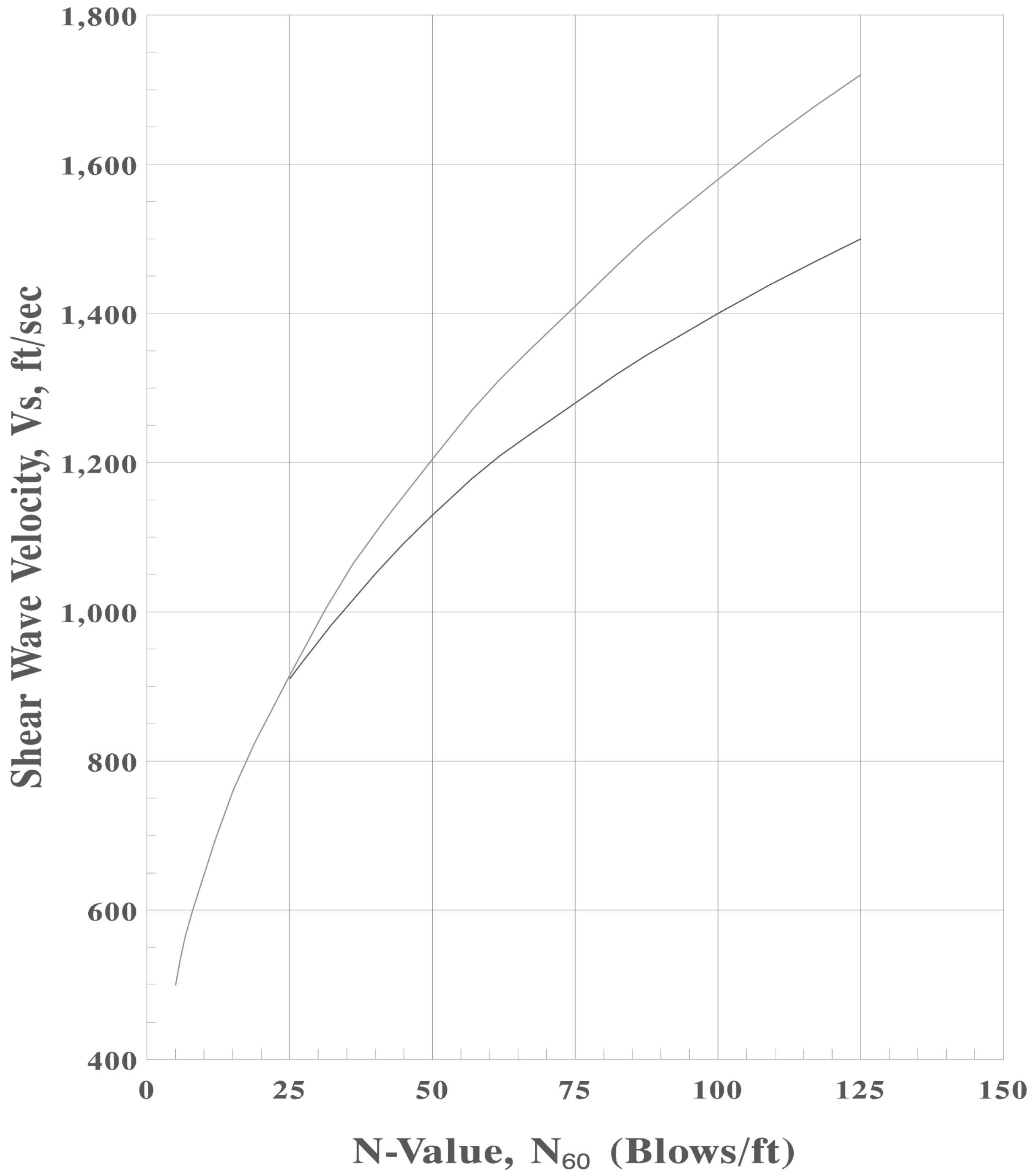
The properties that are most important for dynamic analyses are the stiffness, material damping, and unit weight. These enter directly into the computations of dynamic response. In addition, the location of the water table, degree of saturation, and grain size distribution may be important, especially when liquefaction is a potential problem.

One method of direct determination of dynamic soil properties in the field is to measure the velocity of shear waves in the soil. The waves are generated by impacts produced by a hammer or by detonating charges of explosives, and the travel times are recorded. This is usually done in or between boreholes. A rough correlation between the number of blows per foot in standard penetration tests and the velocity of shear waves is shown in Figure 10-2.

10.3.2. Types of Soils

As in other areas of soil mechanics, the type of the soil affects its response and determines the type of dynamic problems that must be analysed. The most significant factors separating different types of soils are the grain size distribution, the presence or absence of a clay fraction, and the degree of saturation. It is also important to know whether the dynamic loading is a transient phenomenon, such as a blast loading or earthquake, or is a long-term phenomenon, like a vibratory loading from rotating machinery. The distinction is important because a transient dynamic phenomenon occurs so rapidly that excess pore pressure does not have time to dissipate except in the case of very coarse, clean gravels. In this context the length of the drainage path is also important; even a clean, granular material may retain large excess pore pressure if the drainage path is so long that the pressures cannot dissipate during the dynamic loading. Thus, the engineer must categorize the soil by asking the following questions:

Figure 10-2 Relation between Number of Blows Per Foot in Standard Penetration Test and Velocity of Shear Waves



a) Is the material saturated? If it is saturated, a transient dynamic loading will usually last for such a short time that the soil's response will be essentially undrained. If it is not saturated, the response to dynamic loadings will probably include some volumetric component.

b) Are there fines present in the soil? The presence of fines, especially clays, not only inhibits the dissipation of excess pore pressure, it also decreases the tendency for liquefaction.

c) How dense is the soil? Dense soils are not likely to collapse under dynamic loads, but loose soils may. Loose soils may densify under vibratory loading and cause permanent settlements.

d) How are the grain sizes distributed? Well-graded materials are less susceptible to losing strength under dynamic loading than uniform soils. Loose, uniform soils are especially subject to collapse and failure.

10.3.2.1. Dry and Partially Saturated Cohesionless Soils

There are three types of dry or partially saturated cohesionless soils. The first type comprises soils that consist essentially of small-sized to medium-sized grains of sufficient strength or under sufficiently small stresses, so that grain breakage does not play a significant role in their behavior. The second type includes those soils made up essentially of large-sized grains, such as rockfills. Large-sized grains may break under large stresses and overall volume changes are significantly conditioned by grain breakage. The third type includes fine-grained materials, such as silt. The behavior of the first type of dry cohesionless soils can be described in terms of the critical void ratio. The behavior of the second type depends on the normal stresses and grain size. If the water or air cannot escape at a sufficiently fast rate when the third type of soil is contracting due to vibration, significant pore pressures may develop, with resulting liquefaction of the material.

10.3.2.2. Saturated Cohesionless Soils

If pore water can flow in and out of the material at a sufficiently high rate so that appreciable pore pressures do not develop, behaviour of these soils does not differ qualitatively from that of partially saturated cohesionless soils. If the pore water cannot flow in or out of the material, cyclic loads will usually generate increased pore pressure. If the soil is loose or contractive, the soil may liquefy.

10.3.2.3. Saturated Cohesive Soils

Alternating loads decrease the strength and stiffness of cohesive soils. The decrease depends on the number of repetitions, the relative values of sustained and cycling stresses, and the sensitivity of the soil.

Very sensitive clays may lose so much of their strength that there may be a sudden failure. The phenomenon is associated with a reduction in effective pressure as was the case with cohesionless soils.

10.3.2.4. Partially Saturated Cohesive Soils

The discussion in connection with saturated cohesive soils applies to insensitive soils when they are partially saturated, except that the possibility of liquefaction seems remote.

10.4. Measuring Dynamic Soil Properties

Soil properties to be used in dynamic analyses can be measured in the field or in the laboratory. In many important projects, a combination of field and laboratory measurements is used.

Laboratory measurements of soil properties can be used to supplement or confirm the results of field measurements. They can also be necessary to establish values of damping and modulus at strains larger than those that can be attained in the field or to measure the properties of materials that do not now exist in the field, such as soils to be compacted.

A large number of laboratory tests for dynamic purposes have been developed, and research continues in this area. These tests can generally be classified into two groups: those that apply dynamic loads and those that apply loads that are cyclic but slow enough that inertial effects do not occur.

10.4.1. Field Measurements of Dynamic Modulus

Direct measurement for soil or rock stiffness in the field has the advantage of minimal material disturbance. The modulus is measured where the soil exists. Furthermore, the measurements are not constrained by the size of a sample.

Moduli measured in the field correspond to very small strains. Although some procedures for measuring moduli at large strain have been proposed, none has been found fully satisfactory by the geotechnical engineering community. Since the dissipation of energy during strain, which is called material damping, requires significant strains to occur, field techniques have also failed to prove effective in measuring material damping.

In situ techniques are based on measurement of the velocity of propagation of stress waves through the soil. Because the P-waves or compression waves are dominated by the response of the pore fluid in saturated soils, most techniques measure the S-waves or shear waves. If the velocity of the shear wave through a soil deposit is determined to be V_s, the shear modulus G is:

Equation 10-17: $$G = \rho V_s^2 = \frac{\gamma}{g} V_s^2$$

where:

- ρ = mass density of the soil
- V_s = shear wave velocity
- γ = unit weight of the soil
- g = acceleration of gravity

The techniques for measuring shear wave velocity *in situ* fall into three categories: cross-hole, down-hole, and uphole. All require that borings be made in the soil.

In the cross-hole method, sensors are placed at one elevation in one or more borings and a source of energy is triggered in another boring at the same elevation. The waves travel horizontally from the source to the receiving holes. The arrivals of the S-waves are noted on the traces of the response of the sensors, and the velocity can be calculated by dividing the distance between borings by the time for a wave to travel between them. Because it can be difficult to establish the exact triggering time, the most accurate measurements are obtained from the difference of arrival times at two or more receiving holes rather than from the time between the triggering and the arrival at single hole.

Since P-waves travel faster than S-waves, the sensors will already be excited by the P-waves when the S-waves arrived. This can make it difficult to pick out the arrival of the S-wave. To alleviate this difficulty it is desirable to use an energy source that is rich in the vertical shear component of motion and relatively poor in compressive motion. Several devices are available that do this. The original cross-hole velocity measurement methods used explosives as the source of energy, and these were rich in compression energy and poor in shear energy. It is quite difficult to pick out the S-wave arrivals in this case, and explosives should not be used as energy sources for cross-hole S-wave velocity measurements today[265].

In the down-hole method, the sensors are placed at various depths in the boring and the source of energy is above the sensors - usually at the surface. A source rich in S-waves should be used. This technique does not require as many borings as the cross-hole method, but the waves travel through several layers from the source to the sensors. Thus, the measured travel time reflects the cumulative travel through layers with

[265] ASTM D 4428/D 4428M, *Cross-Hole Seismic Testing*, describes the details of this test.

different wave velocities, and interpreting the data requires sorting out the contribution of the layers. The seismocone version of the cone penetration test is one example of the down-hole method.

In the up-hole method the source of the energy is deep in the boring and the sensors are above it - usually at the surface.

A recently developed technique that does not require borings is the spectral analysis of surface waves (SASW). This technique uses sensors that are spread out along a line at the surface, and the source of energy is a hammer or tamper also at the surface. The surface excitation generates surface waves, in particular Rayleigh waves. These waves occur because of the difference in stiffness between the soil and the overlying air. The particles move in retrograde ellipses whose amplitudes decay from the surface. Recording the signals at each of the receiving stations and using a computer program to perform a spectral analysis of the data interpret the test results. Computer programs have been developed that will determine the shear wave velocities from the results of the spectral analysis.

The SASW method is most effective for determining properties near the surface. To increase the depth of the measurements the energy at the source must be increased. Measurements for the few feet below the surface, which may be adequate for evaluating pavements, can be accomplished with a sledge hammer as a source of energy, but measurements several tens of feet deep require track-mounted seismic "pingers." The SASW method works best in cases where the stiffness of the soils and rocks increases with depth. If there are soft layers lying under stiff ones, the interpretation may be ambiguous. A soft layer lying between stiff ones can cause problems for the crosshole method as well, because the waves will travel fastest through the stiff layers and the soft layer may be masked.

The cross-hole, down-hole, and up-hole methods may not work well very near the surface, where the complications due to surface effects may affect the readings. This is the region where the SASW method should provide the best result. The crosshole technique employs waves with horizontal particle motion, the down-hole and up-hole methods use waves whose particle motions are vertical or nearly so, and the surface waves in the SASW method have particle motions in all sensors. Therefore, a combination of these techniques can be expected to give a more reliable picture of the shear modulus than any one used alone.

10.4.2. Resonant Column Method

The most widely used of the laboratory tests that apply dynamic loads is the resonant-column method, described by ASTM D-4015, *Modulus and Damping of Soils by the Resonant-Column Method*. In this test, a column of soil is subjected to an oscillating longitudinal or torsional load. The frequency is varied until resonance occurs. From the frequency and amplitude at resonance, the modulus and damping of the soil can be calculated. A further measure of the damping can be obtained by observing the decay of oscillations when the load is cut off.

ASTM D 4015 describes only one type of resonant-column device, but several types have been developed. These devices provide measurements of both modulus and damping at low strain levels. Although the strains can sometimes be raised a few percent, they remain essentially low strain devices. The torsional devices give measurements on shear behavior, and the longitudinal devices give measurements pertaining to extension and compression behavior.

10.4.3. Cyclic Triaxial Tests

10.4.3.1. Overview of ASTM D 3999

The most widely used of the cyclic loading laboratory tests is the cyclic triaxial test, described in ASTM D 3999, *Determination of the Modulus and Damping Properties of Soils Using the Cyclic Triaxial Apparatus*. In this test, a cyclic load is applied to a column of soil over a number of cycles slowly enough that inertial effects do not occur. The response at a single amplitude of load is observed, and the test is repeated at a higher load. Figure 10-3 (A) shows the typical pattern of stress and strain, expressed as shear stress and shear strain. The shear modulus is the slope of the secant line inside the loop.

The critical damping ratio, D, is:

Equation 10-18: $$D = \frac{A_i}{4\pi A_T}$$

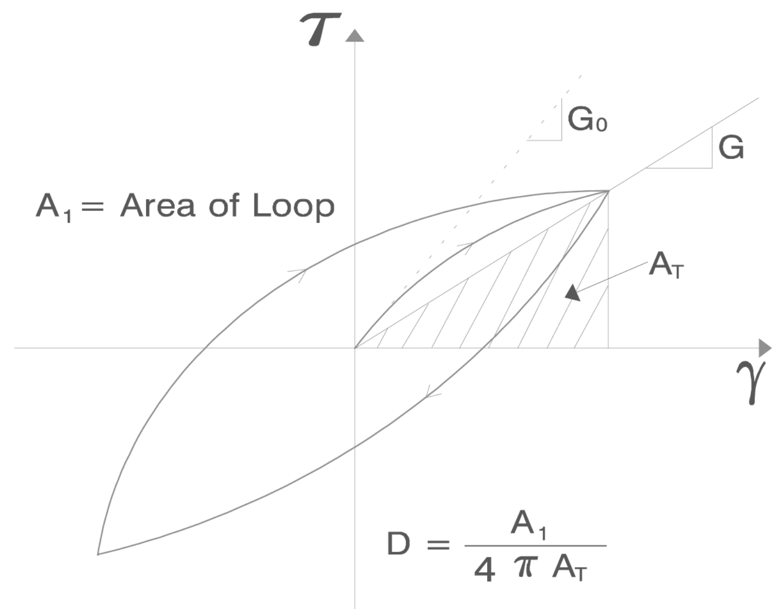
where:

- A_i = area of the loop
- A_T = shaded area

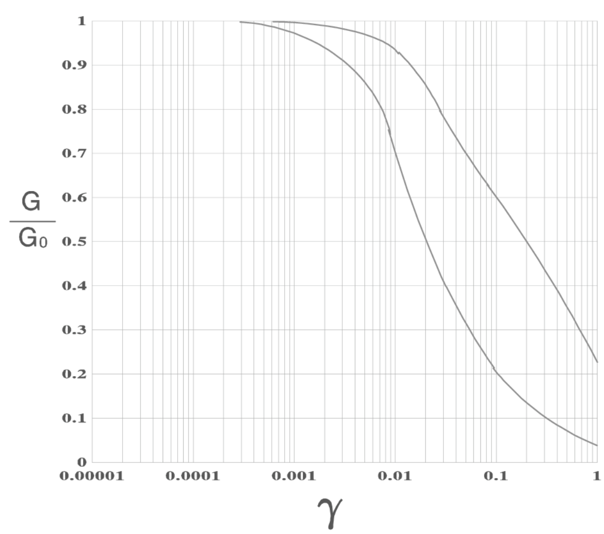
Other types of cyclic loading devices also exist, including cyclic simple shear devices. Their results are interpreted similarly. These devices load the sample to levels of strain much larger than those attainable in the resonant column devices. A major problem in both resonant-column and cyclic devices is the difficulty of obtaining undisturbed samples. This is especially true for small-strain data because the effects of sample disturbance are particularly apparent at small strains.

The results of laboratory tests are often presented in a form similar to Figure 10-3 (B-1 and B-2). In Figure 10-3 (B-1) the ordinate is the secant modulus divided by the modulus at small strains. In Figure 10-3 (B-2) the ordinate is the value of the initial damping ratio. Both are plotted against the logarithm of the cyclic strain level.

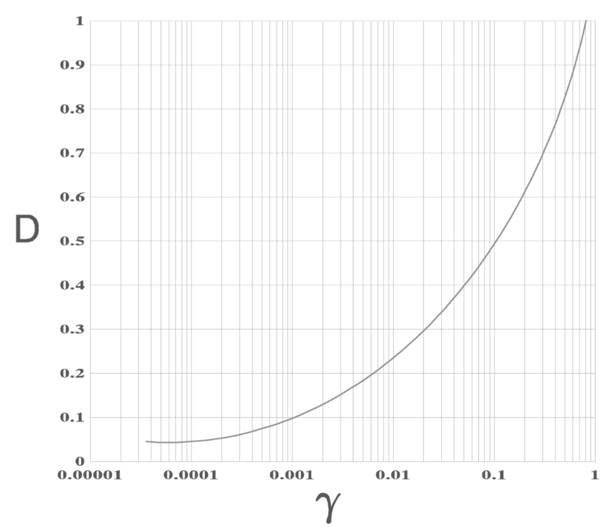
Figure 10-3 Laboratory Measurement of Dynamic Soil Properties



(A) Typical Pattern of Shear Stress and Shear Strain



(B-1) Cyclic Shear Strain vs. the Ratio of Secant Modulus and the Modulus of Small Strain



(B-2) Cyclic Shear Strain vs. the Value of Initial Damping Ratio

As stated above, ASTM D 3999 is primarily used to determine the damping and dynamic modulus of soils. Cyclic triaxial testing, however, can be used to determine other soil properties as well. The test given below is intended to determine the liquefaction potential of the soil; it thus differs from the description given above.[266]

[266] For this clarification we are grateful to Dr. Frank C. Townsend, University of Florida.

10.5. Machine Foundations

10.5.1. Analysis of Foundation Vibration

Types of foundation vibration are given below.

10.5.1.1. Machine Foundations

Operation of machinery can cause vibratory motions in the foundations and soils. The pattern of the applied load versus time will be repeated for many cycles.

Figure 4 shows waveforms of vibrations generated from rotating and impact machinery. The vibration may be irregular as shown in Figure 10-4 (A). In this case, it is often idealized into a simple form as shown in Figure 10-4 (B). These loads are generally assumed to persist during the design life of the structure.

10.5.1.2. Impact Loadings

Impact loading is generally transient. Typical examples are those generated by pile driving, heavy tamping, and blasting. Figure 10-4 (C) shows impact-generated waveform.

10.5.1.3. Characteristics of Oscillating Loads

Although there is a transient portion of the response as an oscillating load starts, the most important response to oscillating loads usually occurs when the load is maintained at steady state. There are two basic types of oscillating loads. In the first, the load is a sinusoidal function at constant amplitude with amplitude that is independent of frequency. In the second, the load is a sinusoidal function, but the amplitude depends on frequency. The latter is the case for rotating machinery, where the load is proportional to the eccentric mass, the moment arm of the eccentric mass, and the frequency of operation. Figure 10-5 shows an example of impact and rotating machinery vibration forces.

10.5.1.4. Method of Analysis

Machine induced foundation vibrations are analyzed as follows:

a) Simplify the actual foundation geometry and soil properties into an single degree of freedom system, involving a spring constant K and damping ratio D. Compute spring constant K and damping ratio D for anticipated modes of vibration. Figure 10-6 shows examples of modes of vibration.

Figure 10-4 Wave Forms of Vibrations Generated from Rotating and Impact Machinery

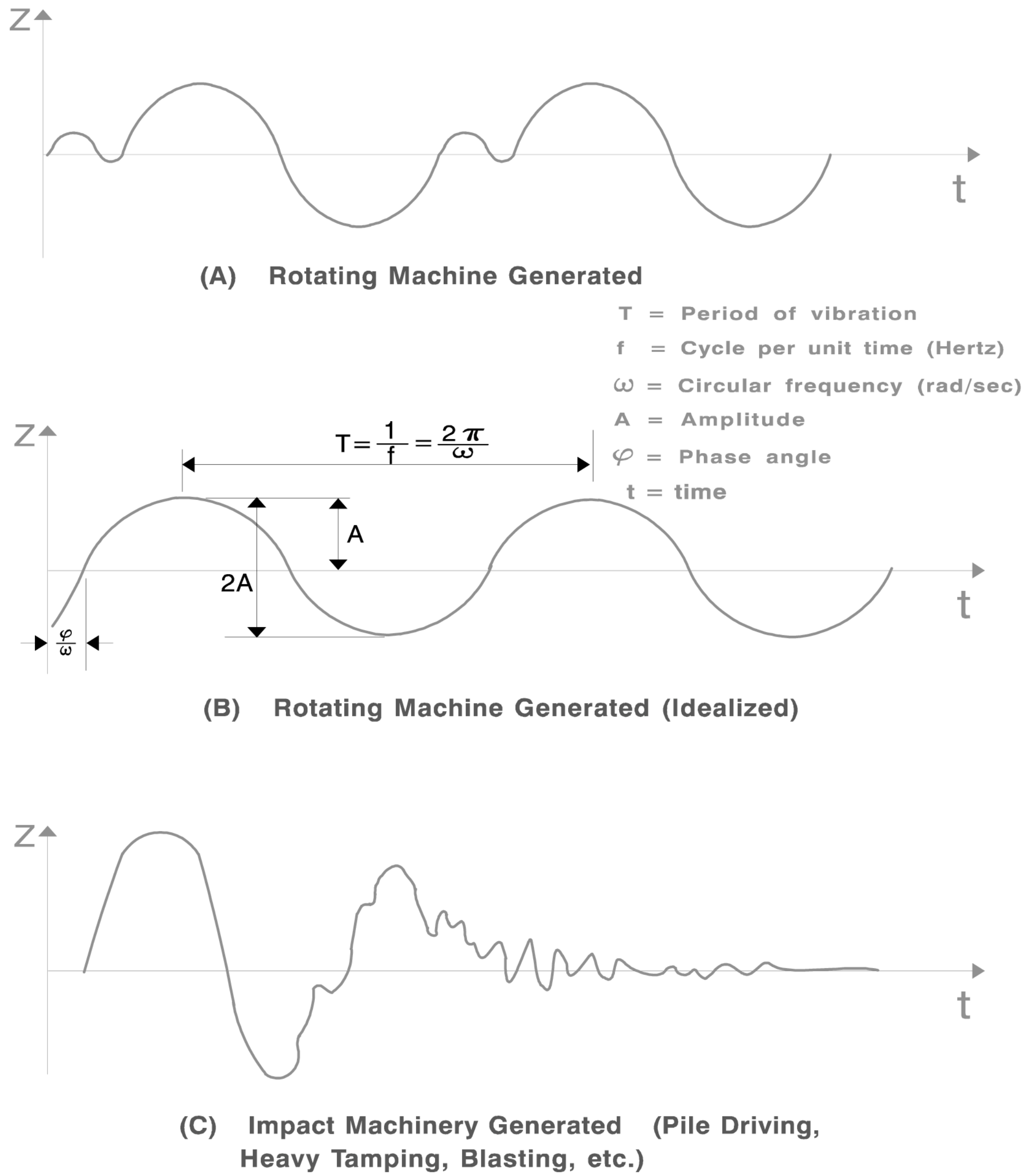
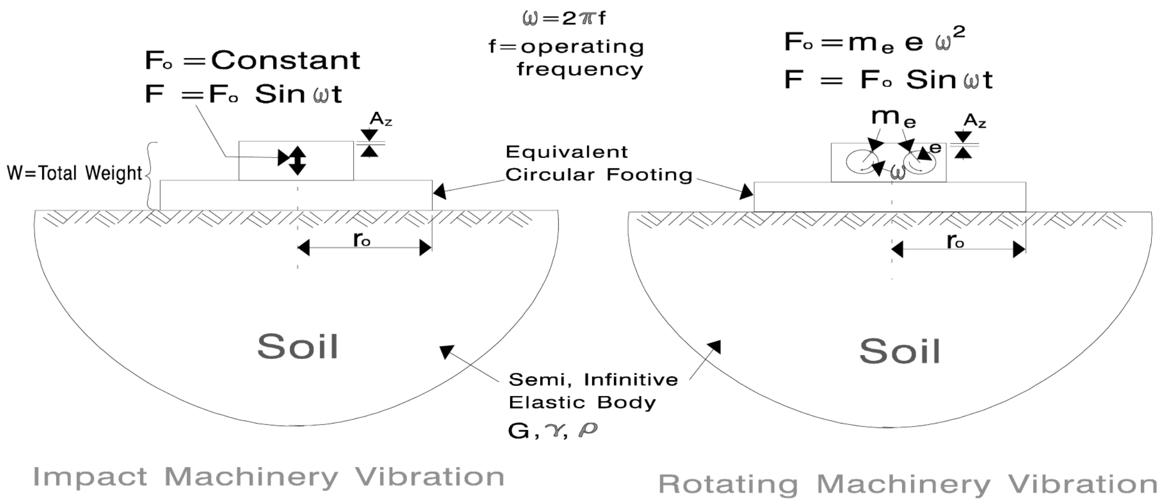


Figure 10-5 Frequency Dependent and Constant Amplitude Exciting Forces



<u>Definitions</u>:

Az = Vibration amplitude
ν = Poissons Ratio
m = Mass of Foundation and Machine

ρ = Foundation mass density = $\frac{\gamma_t}{g}$

r_o = Effective Radius = $\sqrt{B \cdot \frac{L}{\pi}}$ for vertical or horizontal translation

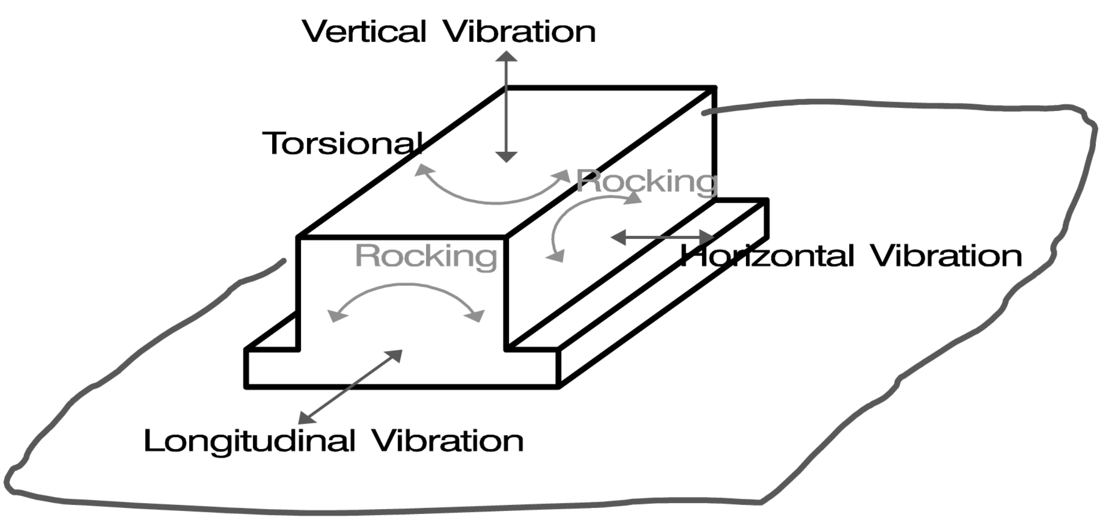
$$= \left(\sqrt{B \cdot \frac{L^3}{3 \cdot \pi}}\right)^{\frac{1}{2}} \quad \text{for rocking}$$

$$= \left(\sqrt{B \cdot L \cdot \frac{B^2 + L^2}{6 \cdot \pi}}\right)^{\frac{1}{2}} \quad \text{for torsion}$$

B = Width of foundation (along axis of rotation for case of rocking)
L = Length of foundation (in plane of rotation or rocking)
I_ψ = Mass moment of inertia around axis of rotation for rocking
I_θ = Mass moment of inertia around axis of rotation for torsion
G = Dynamic shear modulus
ω = Frequency of forced vibration (radians/sec)

Figure 10-6 Modes of Vibration[267]

Mode of Vibration	Mass (or Inertia) Ratio	Damping Coefficient	Damping Ratio $D := \frac{C}{\sqrt{Km}}$	Spring Constant K
Vertical	$B_z := \frac{1-\nu}{4} \cdot \frac{m}{\rho \cdot (ro)^3}$	$C_z := \frac{3.4 \cdot (ro)^2}{1-\nu} \cdot \sqrt{\rho \cdot G}$	$D_z := \frac{0.425}{\sqrt{Bz}}$	$K_z := \frac{4 \cdot G \cdot ro}{1-\nu}$
Horizontal Sliding	$Bx := \frac{(7-8 \cdot \nu)}{32 \cdot (1-\nu)} \cdot \frac{m}{\rho \cdot (ro)^3}$	$Cx := \frac{4.6 \cdot (ro)^2}{2-\nu} \cdot \sqrt{\rho \cdot G}$	$Dx := \frac{0.288}{\sqrt{Bx}}$	$Kx := \frac{32 \cdot (1-\nu) \cdot G \cdot ro}{7-8 \cdot \nu}$
Rocking	$B\psi := \frac{3 \cdot (1-\nu)}{4} \cdot \frac{I\psi}{\rho \cdot (ro)^5}$	$C\psi := \frac{0.8 \cdot (ro)^4}{(1-\nu) \cdot (1+B\psi)} \cdot \sqrt{\rho \cdot G}$	$D\psi := \frac{0.15}{(1+B\psi) \cdot \sqrt{B\psi}}$	$K\psi := \frac{8 \cdot G \cdot (ro)^3}{3 \cdot (1-\nu)}$
Torsional	$B\theta := \frac{I\theta}{\rho \cdot (ro)^5}$	$C\theta := \frac{4 \cdot \sqrt{B\theta}}{1-2 \cdot B\theta} \cdot \sqrt{\rho \cdot G}$	$D\theta := \frac{0.5}{1+2 \cdot B\theta}$	$K\psi := \frac{16 \cdot G \cdot (ro)^3}{3}$



b) Specify the type of exciting force. For constant amplitude exciting force, the load is expressed by:

[267] The classic work on vertical vibration is Lysmer, J., *Vertical Motion of Rigid Footings*. Contract Report 3-115. Vicksburg, MS: U.S. Army Corps of Engineers, Waterways Experiment Station, 1965, available at http://www.vulcanhammer.net. This report was also a doctoral dissertation at the University of Michigan, Ann Arbor.

Equation 10-19:

$$F = F_o \sin(\omega t)$$

or

$$\grave{I} = \grave{I}_o \sin(\omega t)$$

Where:

- ω = operating frequency (rad/sec) = $2\pi f$
- f = operating frequency (cycle/sec)
- F_o or M_o = amplitude of exciting force or moment (constant)
- F or M = exciting force or moment
- t = time

The exciting force F or moment M may depend on the frequency, ω, and the eccentric mass. In this case:

Equation 10-20:

$$F_o = m_o e\grave{u}^2$$

or

$$F_o = m_o e\omega^2 L$$

Where:

1. m_e = eccentric mass
2. e = eccentric radius from centre of rotation to centre of gravity
3. L = moment arm

c) Compute the undamped natural frequency, f_n, in cycles/second or ω_n in rad/second.

Equation 10-21:

$$f_n = \frac{\pi}{2}\sqrt{\frac{k}{m}}$$

or

$$f_n = \frac{\pi}{2}\sqrt{\frac{k}{I}}$$

or

$$\omega_n = \sqrt{\frac{k}{m}}$$

or

$$\omega_n = \sqrt{\frac{k}{I}}$$

Where:

- K = k_z for vertical mode, k_x for horizontal mode, k_y for rocking mode and k_q for torsional mode
- M = mass of foundation and equipment for vertical and horizontal modes
- I_y = mass moment of inertia around axis of rotation in rocking modes
- I_q = mass moment of inertia around axis of rotation in torsional modes.

Thus

Equation 10-22: $f_n = \frac{\pi}{2}\sqrt{\frac{k_z}{m}}$ **(for vertical mode)**

Equation 10-23: $f_n = \frac{\pi}{2}\sqrt{\frac{k_x}{m}}$ **(for horizontal mode)**

Equation 10-24: $f_n = \frac{\pi}{2}\sqrt{\frac{k_q}{I_q}}$ **(for torsional (yawing) mode)**

Equation 10-25: $f_n = \frac{\pi}{2}\sqrt{\frac{k_y}{I_y}}$ **(for rocking mode)**

d) Compute the mass ratio B and damping ratio D for modes analyzed using the formulas in Figure 10-6. Note that the damping terms are functions of mass and geometry - not of internal damping in the soil. This damping is called radiation damping and represents the fact that energy is transmitted away from the foundation toward the distant boundaries of the soil.

e) Calculate static displacement amplitude, A_s

Equation 10-26: $A_s = \frac{F_o}{k}$

or calculate the static relation as:

Equation 10-27: $\theta_s = \frac{M_o}{k}$

f) Compute the ratio f/f_n (same as ω/ω_n).

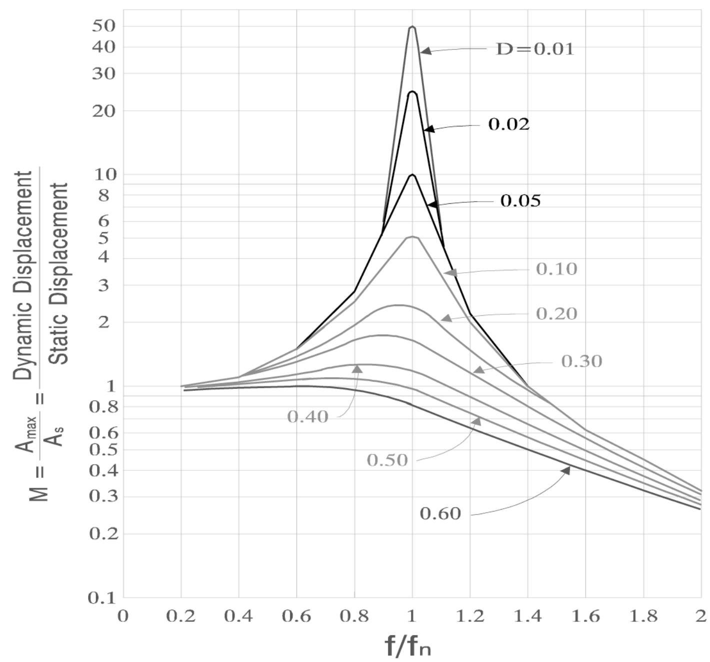
g) Calculate magnification factor M $=A_{max}/A_s$ or θ_{max}/θ_s from Figure 10-7.

h) Calculate maximum amplitude $A_{max} = M\ A_s$.

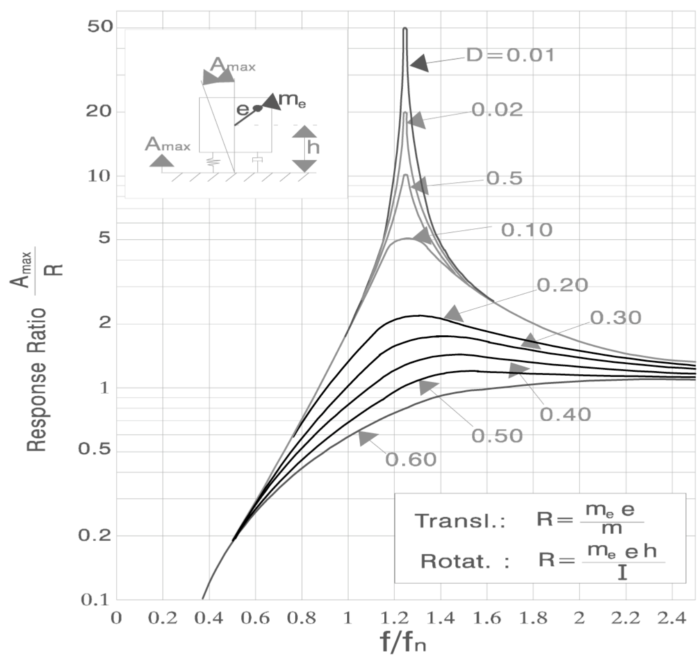
i) If the amplitudes are not acceptable, modify design and repeat Steps c) through h).

j) Figure 10-8 illustrates the calculation of vertical amplitude, horizontal amplitude alone and rocking amplitude alone. When these analyses are performed, particular attention must be paid to keeping track of the units.

Figure 10-7 Response Curves for Single-Degree-of-Freedom System with a Viscous Damping



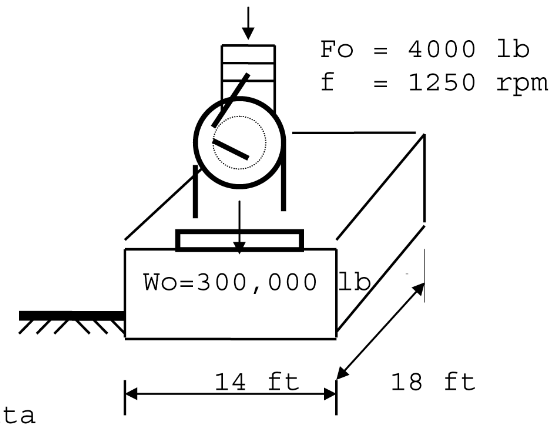
(A) $F = F_0 \text{ Sin } \omega t$



(B) $F = m_e e \omega^2 \text{ Sin } \omega t$

Figure 10-8 Example Calculations of Vertical, Horizontal, and Rocking Motions[268]

A. VIBRATION IN VERTICAL MODE



Equipment Data

Given a high speed generator with a frequency dependent amplitude

$F_o := 4000 \cdot lb$

Weight of vibrating equipment and foundation block Wo=300,000lb

Operating frequency f=1250 rpm $f := 20.83 \cdot sec^{-1}$ cycles/sec;

$\omega := f \cdot 2 \cdot \pi$ $\omega = 130.879 \cdot sec^{-1}$ rad/sec; Dimension: $B := 18 \cdot ft$ $L := 14 \cdot ft$

Soil Properties

Total unit weight $\gamma_t := 120 \cdot \frac{lb}{ft^3}$ Poisson's ratio $\nu := 0.35$;

Shear Modulus $G := 6700 \cdot \frac{lb}{in^2}$

Equivalent Radius

$r_0 := \sqrt{\frac{B \cdot L}{\pi}}$ $r_0 = 8.956$

Spring Constant

$K_z := \frac{4 \cdot G \cdot r_0}{1 - \nu}$ $K_z = 5.318$

Mass Ratio

$\rho := \frac{\gamma_t}{32.2 \cdot \frac{ft}{sec^2}}$ $m := \frac{W_o}{32.2 \cdot \frac{ft}{sec^2}}$ $B_z := \frac{(1 - \nu) \cdot m}{4 \cdot \rho \cdot r_0^3}$

$\rho = 3.727$ $m = 9.317$ $B_z = 0.565$

Damping Ratio

$D_z := \frac{0.425}{\sqrt{B_z}}$

$D_z = 0.565$

Static Amplitude

$A_s := \frac{F_o}{K_z}$

$A_s = 9.027 \cdot 10^{-4} \cdot in$

Natural Frequency

$\omega_n := \sqrt{\frac{K_z}{m}}$

$\omega_n = 75.548 \, sec^{-1}$ rad/sec

Dynamic Amplitude

$\frac{\omega}{\omega_n} = 1.732$ Then from Figure 7 (B) and for D = 0.56 $M := 1.1$

$A_{max} := A_s \cdot M$ $A_{max} = 9.929 \cdot 10^{-4} \cdot in$ Maximum dynamic amplitude

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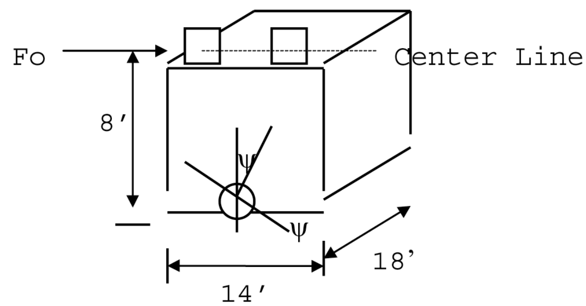
[268] Figure 7 cited in this figure is actually Figure 10-7.

B. EXAMPLE CALCULATION FOR HORIZONTAL TRANSLATION AND ROCKING



Equipment Data

Assume constant amplitude $F_o := 300 \cdot lb$ Dimensions $B := 18 \cdot ft$ $L := 14 \cdot ft$

Weight of foundation and machinery $W := 400000 \cdot lb$

Mass Moment of Inertia around axis of rotation $I_\psi := 400000 \cdot lb \cdot ft \cdot sec^2$

Operating Frequency f=350 rpm $f := 5.8 \cdot sec^{-1}$ cycles/sec $\omega := f \cdot 2 \cdot \pi$

$\omega = 36.442$ rad/sec

Soil Properties

Total Unit Weight $\gamma_t := 120 \cdot \frac{lb}{ft^3}$ Poisson's Ratio $\nu := 0.35$

Shear Modulus $G := 6700 \cdot \frac{lb}{in^2}$

HORIZONTAL TRANSLATION ONLY

Equivalent Radius

$$r_0 := \sqrt{\frac{B \cdot L}{\pi}}$$

$r_0 = 8.956$

Spring Constant

$$K_x := \frac{32 \cdot (1 - \nu) \cdot G \cdot r_0}{7 - 8 \cdot \nu}$$

$K_x = 4.279$

Mass Ratio

$$m := \frac{400000 \cdot lb}{32.2 \cdot \frac{ft}{sec^2}}$$

$m = 1.242$

$$\rho := \frac{120 \cdot \frac{lb}{ft^3}}{32.2 \cdot \frac{ft}{sec^2}}$$

$\rho = 3.727$

$$B_x := \frac{7 - 8 \cdot \nu}{32 \cdot (1 - \nu)} \cdot \frac{m}{\rho \cdot r_0^3}$$

$B_x = 0.937$

Damping Ratio

$$D_x := \frac{0.288}{\sqrt{B_x}}$$

$D_x = 0.298$

Static Displacement

$$A_s := \frac{F_o}{K_x}$$

$A_s = 8.413$

Natural Frequency

$$\omega_n := \sqrt{\frac{K_x}{m}}$$

$\omega_n = 58.693$ rad/sec

Dynamic Amplitude

$\frac{\omega}{\omega_n} = 0.621$

From Figure 7 (A), $M := 1.4$

$A_{max} := A_s \cdot M$

$A_{max} = 1.178$ Maximum Horizontal Movement

ROCKING ALONE

$r_0 := \left(\frac{B \cdot L^3}{3 \cdot \pi}\right)^{\left(\frac{1}{4}\right)}$

$r_0 = 8.508$

$B_\psi := \frac{3 \cdot (1 - \nu)}{8} \cdot \frac{I_\psi}{\rho \cdot r_0^5}$

$B_\psi = 0.454$

$F_0 := (300 \cdot lb) \cdot (8 \cdot ft)$

$F_0 = 2.88$

Spring Constant

$K_\psi := \frac{8 \cdot G \cdot r_0^3}{3 \cdot (1 - \nu)}$

$K_\psi = 2.926$ lb*in/rad

Damping Ratio

$D_\psi := \frac{0.15}{\left(1 + B_\psi\right) \cdot \sqrt{B_\psi}}$

$D_\psi = 0.153$

Natural Frequency

$\omega_n := \sqrt{\frac{K_\psi}{I_\psi}}$

$\omega_n = 78.07$

Static Rotation

$A_\psi := \frac{F_0}{K_\psi}$

$A_\psi = 9.844$ radians

Dynamic Amplitude

$\omega := 131 \cdot sec^{-1}$

From Figure 7 (A), $M := 0.4$

$\frac{\omega}{\omega_n} = 1.678$

Maximum Rocking Movement

$A_{\psi.max} := A_\psi \cdot M$

$A_{\psi.max} = 3.938$

Horizontal Motion At Machine Centerline

$Movement := A_{\psi.max} \cdot (8 \cdot ft)$

$Movement = 3.15$ radians

NOTE:

Above analysis is approximate since horizontal and rocking modes are coupled.

A lower bound estimate of first mode frequency may be calculated based on natural frequency w_n for rocking mode alone, and horizontal translation mode alone.

10.5.1.5. Dynamic Soil Properties

There are several interrelated criteria for design of foundations for machinery. The most fundamental is that the vibratory movement be held to a level below that which could damage the machinery or cause settlement of loose soils. In many cases, there are too many unknowns to solve this problem. The other criterion is to proportion the foundation such that resonance with the operating frequency of the machine is avoided.

For high frequency machines (say over 1000 RPM) it is common to "low tune" the foundation, so that the foundation frequency is less than half the operating frequency. For low frequency machines (say under 300 RPM) it is common to "high tune" the foundation, so that the fundamental frequency is at least twice the operating frequency.

10.5.2. Design to Avoid Resonance

Settlements from vibratory loads and displacements of the machinery itself in all directions are accentuated if imposed vibrations are resonant with the natural frequency of the foundation soil system. Both the amplitude of foundation motion and the unbalanced exciting force are increased at resonance. Compact cohesionless soils will be densified to some degree with accompanying settlement. Avoidance of resonance is particularly important in cohesionless materials, but should be considered for all soils. To avoid resonance, the following guidelines may be considered for initial design to be verified by the previous methods.

10.5.2.1. High-Speed Machinery

For machinery with operating speeds exceeding about 1000 RPM, provide a foundation with natural frequency no higher than one-half of the operating value, as follows:

a) Decrease natural frequency by increasing the weight of foundation block, analyse vibrations in accordance with the methods discussed.

b) During starting and stopping, the machine will operate briefly at the resonant frequency of the foundation. Compute probable amplitudes at both resonant and operating frequencies, and compare them with allowable values to determine if the foundation arrangement must be altered.

10.5.2.2. Low-Speed Machinery

For machinery operating at a speed less than about 300 RPM, provide a foundation with a natural frequency at least twice the operating speed, by one of the following:

a) For spread foundations, increase the natural frequency by increasing base area or reducing total static weight.

b) Increase modulus or shear rigidity of the foundation soil by compaction or other means of stabilization.

c) Consider the use of piles to provide the required foundation stiffness. See example in Figure 10-9.

10.5.2.3. Coupled Vibrations

Vibrations are coupled when their modes are not independent but influence one another. A mode of vibration is a characteristic pattern assumed by the system in which the motion of each particle is simple harmonic with the same frequency. In most practical problems, the vertical and torsional modes can be assumed to be uncoupled (i.e., independent of each other). However, coupling effects between the horizontal and rocking modes can be significant depending on the distance between the centre of gravity of the footing and the base of the footing. The analysis for this case is complicated and time consuming.

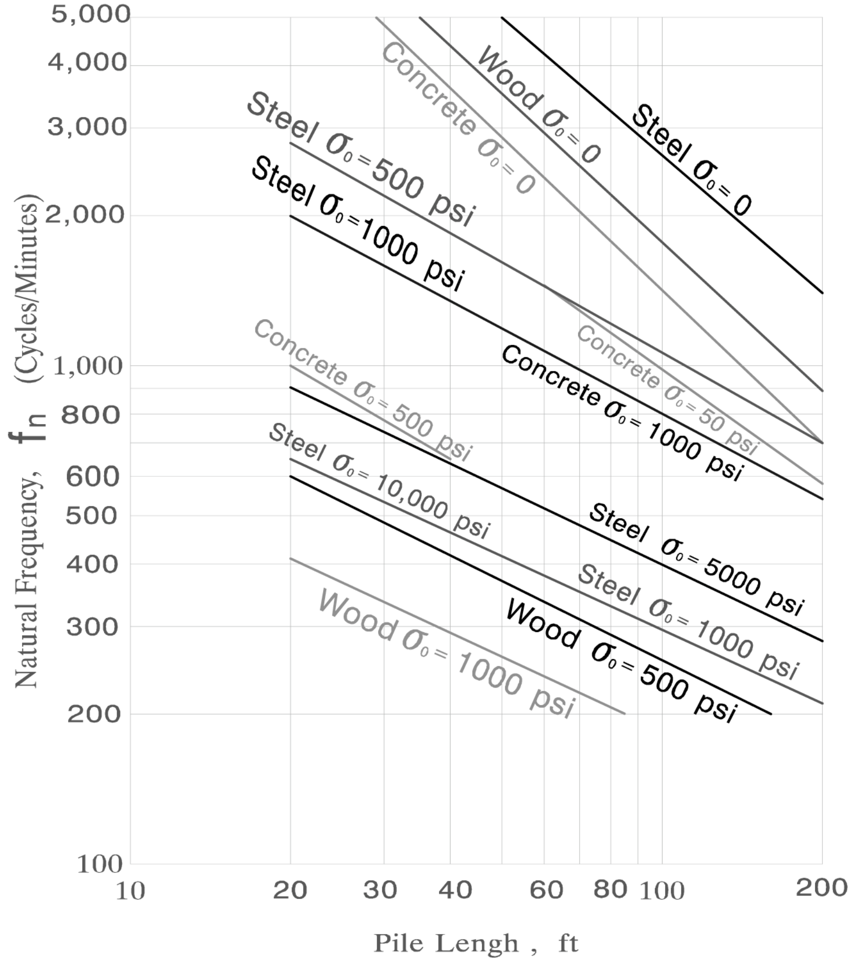
A lower bound estimate of the first mode, f_o, of coupled rocking and horizontal vibration can be obtained from:

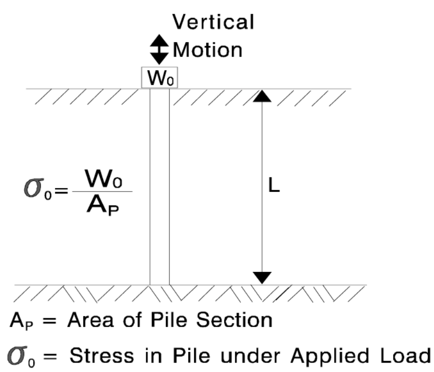
Equation 10-28: $$\frac{1}{f_o^2} = \frac{1}{f_x^2} + \frac{1}{f_y^2}$$

f_x and f_y are the undamped natural frequencies in the horizontal and rocking modes respectively.[269]

[269] For further guidance refer to Richart, F. E., Jr., Hall, S. R., and Woods, R. D., *Vibrations of Soils and Foundations*, Prentice-Hall, Inc., 1970, and Bereduqo, Y. O. and Novak, M., "Coupled Horizontal and Rocking Vibrations of Embedded Footings," Canadian Geotechnical Journal, Vol. 9, No. 4, 1972.

Figure 10-9 Natural Undamped Frequency of Point Bearing Piles on Rigid Rock





Example Design to Avoid Resonance:

For machinery operating at a low-speed (less than 300 rpm), provide a foundation with a natural frequency at least twice the operating speed.
This example considers the use of piles to provide the required foundation stiffness.

10.5.2.4. Effect of Embedment

Stiffness and damping are generally increased with embedment. However, analytical results (especially for damping) are sensitive to the conditions of the backfill (properties, contact with the footings, etc.). For footings embedded in a uniform soil with a Poisson's ratio of 0.4, the modified stiffness parameters are approximated as follows[270]:

Equation 10-29:

$$(k_z)_d \approx k_z\left[1+0.4\left(\frac{d}{r_o}\right)\right]$$

$$(k_x)_d \approx k_x\left[1+0.8\left(\frac{d}{r_o}\right)\right]$$

$$(k_y)_d \approx k_y\left[1+0.6\left(\frac{d}{r_o}\right)+0.3\left(\frac{d}{r_o}\right)^3\right]$$

$$(k_q)_d \approx k_q\left[1+2.4\left(\frac{d}{r_o}\right)\right]$$

Where $(k_z)_d$, $(k_x)_d$, $(k_y)_d$, and $(k_q)_d$, are spring constants for depth of embedment d.

Increases in embedment d will cause an increase in damping, but the increase in damping is believed to be sensitive to the condition of backfill. For footings embedded in a uniform soil, the approximate modifications for damping coefficient C (in Figure 10-6) are:

Equation 10-30:

$$(C_z)_d \approx C_z\left[1+1.2\left(\frac{d}{r_o}\right)\right]$$

$$(C_\theta)_d \approx r_o^4\sqrt{\rho G}\left[0.7+5.4\left(\frac{d}{r_o}\right)\right]$$

Where $(C_z)_d$ and $(C_\theta)_d$ are the damping coefficients in vertical and torsion modes for embedments d.

10.5.2.5. Proximity of a Rigid Layer

A relatively thin layer of soil over rigid bedrock may cause serious magnification of all amplitudes of vibration. In general, the spring constants increase with decreasing thickness of soil while damping coefficients decrease sharply for the vertical modes and to a lesser extent for horizontal and rocking modes. Use the following approximate relation for adjusting stiffness and damping to account for presence of a rigid layer[271]:

[270] Roesset, J. R., Stiffness and Damping Coefficients of Foundations, Dynamic Response of Pile Foundations, Analytical Aspects, 1980.

[271] Kausel, E. and Roesset, J. M., Dynamic Stiffness of Circular Foundations, Journal of the Engineering Mechanics Division, Vol. 101, No. EM6, 1975.

$$(K_Z)_L = K_Z\left(1+\frac{r_o}{H}\right)$$

Equation 10-31: $$(K_x)_L = K_x\left(1+\frac{r_o}{2H}\right),\ \frac{r_o}{H}<\frac{1}{2}$$

$$(K_y)_L = K_y\left(1+\frac{r_o}{6H}\right)$$

Where $(K_z)_L$,$(K_x)_L$,$(K_y)_L$ are stiffness parameters in case a rigid layer exists at depth H below a footing with radius r_o.

The damping ratio parameter D is reduced by the presence of a rigid layer at depth H. The modified damping coefficient (D_z) is 1.0 D_z for $H/r_o = \infty$, and approximately 0.31 D_z, 0.16 D_z, 0.09 D_z, and 0.044 D_z for $H/r_o = 4$, 3, 2 and 1 respectively[272].

10.5.2.6. Vibration for Pile Supported Machine Foundation

For piles bearing on rigid rock with negligible side friction, use Figure 9 for establishing the natural frequency of the pile soil system. Tip deflection and lateral stiffness can have a significant effect on natural frequency of the pile soil system[273]. Alternatively, and for important installations, such coefficients can be evaluated from field pile load tests.

10.5.3. Bearing Capacity and Settlements

Vibration tends to densify loose nonplastic soils, causing settlement. The greatest effect occurs in loose, coarse-grained sands and gravels. These materials must be stabilized by compaction or other means to support spread foundations for vibrating equipment. Shock or vibrations near a foundation on loose, saturated nonplastic silt, or silty fine sands, may produce a quick condition and partial loss of bearing capacity. In these cases, bearing intensities should be less than those normally used for static loads. For severe vibration conditions, reduce the bearing pressures to one-half allowable static values.

In most applications, a relative density of 70% to 75% in the foundation soil is satisfactory to preclude significant compaction settlement beneath the vibratory equipment. However, for heavy machinery, higher relative densities may be required. The following procedure may be used to estimate the compaction settlement under operating machinery.

The critical acceleration of machine foundations, $(a)_{crit}$, above which compaction is likely to occur, may be estimated based on[274]:

---

[272] Richart, F. E., Soil Structural Interaction, Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, Tokyo, 1977.

[273] Owies, I. S., Response of Piles to Vibratory Loads, Journal of the Geotechnical Division, Vol. 103, 1977. Solution for simple but practical cases for stiffness and damping coefficients are presented by Novak, M. and Aboul-Ella, M., Impedance Function of Piles in Layered Media, Journal of the Engineering Mechanics Division, Vol. 104, 1978.

[274] Barkan, D. D., *Dynamics of Bases of Foundation*, McGraw Hill Book Company, Inc., 1962.

Equation 10-32: $$(a)_{crit} = -\frac{\ln\left[1 - \frac{(D_r)_0}{100}\right]}{\beta}$$

Where:

- $(a)_{crit}$ = critical acceleration expressed in g's
- $(D_r)_o$ = initial (*in situ*) relative density at zero acceleration expressed in percent.
- β = coefficient of vibratory compaction, a parameter depending on moisture content; varies from about 0.8 for dry sand down to 0.2 for low moisture contents (about 5%). It increases to a maximum value of about 0.88 at about 18% moisture content. Thereafter, it decreases.

When densification occurs because of vibrations there will be an increase in relative density D_r, and for a sand layer with a thickness H, the settlement would be ΔH. The strain ΔH/H can be expressed in terms of D_r as:

Equation 10-33: $$\frac{\Delta H}{H} = 0.0025\gamma_{do}\left(\frac{\Delta D_r \%}{100}\right)$$

Where γ_{do} = the initial dry density of the sand layer (lb/cu. ft)

The above equation is based on the range of maximum and minimum dry densities for sands. The change in relative density D_r due to vibration is defined as:

Equation 10-34: $$\Delta D_r = (D_r)_i - (D_r)_o$$

Where:

- $(D_r)_o$ = initial in-situ relative density which may be estimated from the standard penetration resistance
- $(D_r)_f$ = final relative density, which may be conservatively estimated based on:

Equation 10-35: $$(D_r)_f = 100\left\{-e^{-\beta\left[(a_i)_{crit} + a_i\right]}\right\},\ a_i > (a_i)_{crit}$$

Or

Equation 10-36: $$(D_r)_f = (D_r)_o,\ a_i < (a_i)_{crit}$$

Where:

a_i = acceleration expressed in g's. The above equation is based on the work reported in Barkan, 1962.

In the above equation $(a_i)_{crit}$ and (a_i) are the critical acceleration and acceleration produced by equipment in each layer i. The acceleration a_i produced by equipment may be approximated using the following:

Equation 10-37: $$a_i = a_o\sqrt{\frac{r_o}{d}},\ d > r$$

Equation 10-38: $$a_i = a_o,\ d < r$$

Where:

- a_o = acceleration of vibration in g's at foundation level
- d = distance from base of foundation to mid point of soil layer

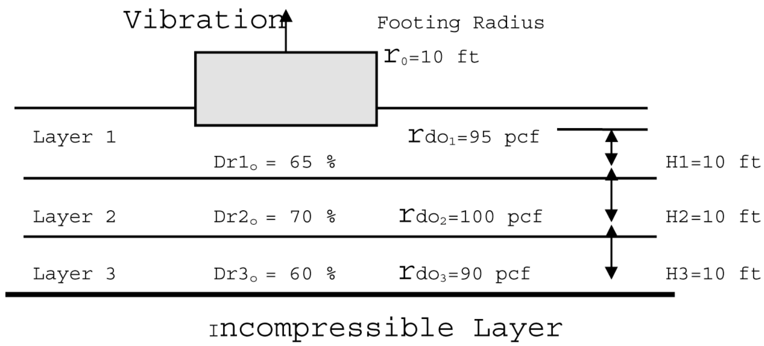
- r_o = equivalent radius of foundation

If maximum displacement, A_{max}, and frequency of vibration, ω rad/sec), are known at base of foundation then:

Equation 10-39: $a_o = \omega^2 A_{\max}$

An example illustrating the use of the above principles is shown in Figure 10-10.

Figure 10-10 Example Calculation for Vibration Induced Compaction Settlement Under Operating Machinery



GIVEN: Soil profile as shown:

Footing with radius $r_o := 10 \cdot ft$ subjected to a vibratory load causing a peak dynamic displacement $A_{max} := 0.007 \cdot in$

Operating frequency $f := 2500 min^{-1}$ (rev/min). Moisture content of soil is 16%. Use $\beta := 0.88$

$\omega := f \cdot 2 \cdot \pi$ $\omega = 26$ (rad/sec) $a_o := \dfrac{\omega^2 \cdot A_{max}}{32.2 \cdot \dfrac{ft}{sec^2}}$ $a_o = 1.2$ g

LAYER 1

Depth to mid layer $d := 5 \cdot ft$ $d < r_o$ Therefore use $a_i := a_o$

Critical Acceleration $a_{crit} := \dfrac{-\ln\left(1 - \dfrac{Dr1_o}{100}\right)}{\beta}$ $a_{crit} = 1.1$ g $a_i > a_{crit}$

$D_{rf} := 100 \left[1 - \exp\left[(-\beta) \cdot \left(a_{crit} + a_i\right)\right]\right]$ $\Delta Dr := D_{rf} - Dr1_o$ $\Delta H := \left(0.0025 \dfrac{ft^3}{lb} \cdot \dfrac{\Delta Dr}{100} \cdot \gamma_{do1}\right) \cdot H_1$

$D_{rf} = 88.26$ % $\Delta Dr = 23.2$ % $\Delta H = 6.63$

LAYER 2

$d_{midlayer} := 15 \cdot ft$

$a_i := a_o \cdot \sqrt{\dfrac{r_o}{d_{midlayer}}}$ $a_{crit} := \dfrac{-\ln\left(1 - \dfrac{Dr2_o}{100}\right)}{\beta}$

$a_i = 0.785$ g $a_{crit} = 1.368$ g

$a_i < a_{crit}$

$D_{rf} := Dr2_o$

No significant compaction settlement is likely.

LAYER 3

$d := 25 \cdot ft$

$a_i := a_o \cdot \sqrt{\dfrac{r_o}{d}}$ $a_{crit} := \dfrac{-\ln\left(1 - \dfrac{Dr3_o}{100}\right)}{\beta}$

$a_i = 0.785$ g $a_{crit} = 1.368$ g

$a_i < a_{crit}$

No significant compaction settlement is likely.

Anticipated Compaction Settlement = 6.6 in. Increase relative density of top layer to 70 percent or greater.

10.5.4. Vibration Transmission, Isolation, and Monitoring

For Vibration transmission, isolation, and monitoring the following guidance is provided.

10.5.4.1. Vibration Transmission

Transmission of vibrations from outside a structure or from machinery within the structure may be annoying to occupants and damaging to the structure.

Vibration transmission may also interfere with the operation of sensitive instruments. See Figure 10-11 for the effects of vibration amplitude and frequency. Tolerable vibration amplitude decreases as frequency increases. For approximate estimates of vibration amplitude transmitted away from the source, use the following relationship:

Equation 10-40: $$A_2 = A_1\sqrt{\frac{r_1}{r_2}}e^{-\alpha(r_2-r_1)}$$

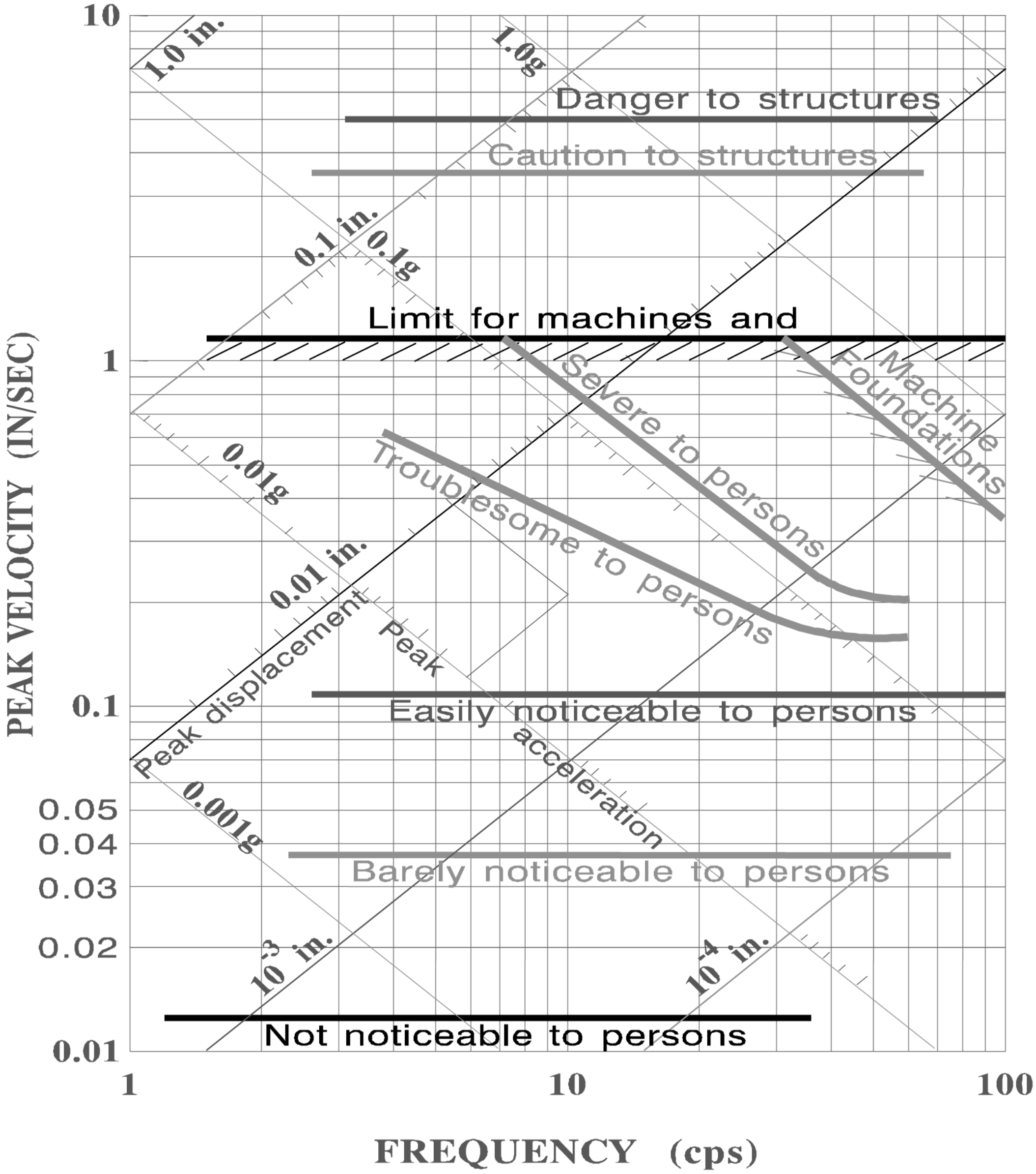
- Where: A_1 = computed or measured amplitude at distance r_1 from vibration source.
- A_2 = amplitudes at distance r_2, $r_2 > r_1$
- α = coefficient of attenuation depending on soil properties and frequency. Use Table 10-1.

Table 10-1 Attenuation Coefficient for Earth Materials

Materials	*α[275] (1/ft)*
Sand: Loose, fine	0.06
Sand: Dense, fine	0.02
Clay: Silty (loess)	0.06
Clay: Dense, dry	0.003
Rock: Weathered volcanic	0.002
Rock: Competent marble	0.00004

[275] The variable α is a function of frequency, and is in this table for 50 Hz. For other frequencies, $\alpha_f = (f/50)\ \alpha_{50}$

Figure 10-11 Allowable Amplitude of Vertical Vibrations



Given Velocity = 0.2 inch/sec.
Frequency = 10 cps
Then from Graph, Displacement = 0.003 inches
Acceleration = 0.03g
Motion is easy noticeable or troublesome to persons

10.5.4.2. Vibration and Shock Isolation

For vibration and shock isolation, see the following methods.

10.5.4.2.1. General Methods

For general methods of isolating vibrating equipment or insulating a structure from vibration transmission, refer to methods described later. These methods include physical separation of the vibrating unit from the structure, or interposition of an isolator between the vibrating equipment and foundation or between the structure foundation and an outside vibration source.

Vibration isolating mediums include resilient materials such as metal springs, or pads of rubber, or cork and felt in combination.

10.5.4.2.2. Other Methods

Additional methods available include the installation of open or slurry-filled trenches, sheet pile walls, or concrete walls. These techniques have been applied with mixed results. Analytical results suggest that for trenches to be effective, the depth of the trench should be 0.67L or larger, where L is wave length for a Rayleigh wave and is approximately equal to V_s/ω; when ω is the angular velocity of vibration in radian/sec, V_s is the shear wave velocity of the soil. Concrete walls may have isolating efficiency depending on the thickness, length, and rigidity[276].

10.5.4.3. Vibration Monitoring

Control of ground vibrations is necessary to ensure that the acceptable levels of amplitudes for structural safety are not exceeded. The sources of vibrations that may affect nearby structures are blasting, pile driving, or machinery. Acceptable vibration amplitudes are usually selected based on conditions of the structure, sensitivity of equipment within the structure, or human tolerance.

For structures which may be affected by nearby sources of vibrations (e.g., blasting, pile driving, etc.) seismographs are usually installed at one or more floors to monitor the effect and maintain records if site vibration limits are exceeded. A seismograph usually consists of one or more transducers that are embedded, attached, or resting on the vibrating structure, element, or soil and connected by a cable to the recording unit. The recording medium may be an oscilloscope or a magnetic tape. Most modern seismographs use digital technology, which provides records that can be processed readily. The actual details of installation depend on the type of equipment, nature of vibration surface, and expected amplitudes of motion.

10.6. Dynamic and Vibratory Compaction

10.6.1. Soil Densification

Dynamic and vibratory methods are often very effective in densifying soil to increase strength, reduce settlements, or lessen the potential for liquefaction. There are several different methods. Some are proprietary, and most contractors prefer to use one method because they have experience with it and have invested in the equipment. Each method works best in certain soils and poorly in others. Therefore, no one method can be used in all circumstances.

10.6.2. Vibro-Densification

Stabilization by densifying in-place soil with vibro-densification is used primarily for granular soils where excess pore water may drain rapidly. It is effective when the relative density is less than about 70%. At higher densities, additional compaction may not be needed and may even be difficult to achieve. Through proper treatment, the density of in-place soil can be increased considerably to a sufficient depth

[276] Haupt, W. A., Isolation of Vibrations by Concrete Core Walls, Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, Tokyo, 1977.

so that most types of structures can be supported safely without undergoing unexpected settlements. Figure 10-12 shows the range of grain size distribution for soils amendable to vibro densification. Effectiveness is greatly reduced in partly saturated soils in which 20% or more of the material passes a No. 200 sieve.

10.6.3. Dynamic Compaction

The use of high-energy impact to densify loose granular soils *in situ* has increased over the years.

This soil improvement technique, commonly known as dynamic compaction has become a well-established method for treating loose granular soils due to its simplicity and cost effectiveness.

A heavy weight (10 to 40 tons or more) is dropped from a height of 50 to 130 feet at points spaced 15 to 30 feet apart over the area to be densified to apply a total energy of 2 to 3 blows per square yard. In saturated granular soils, the impact energy will cause liquefaction followed by settlement as the water drains.

The method can be used both above and below the ground-water level. In granular soils, high-energy impact causes partial liquefaction. It generates low frequency vibrations that make this method less desirable in urban areas and near existing structures. It is not a proven technique in saturated fine-grained soils.

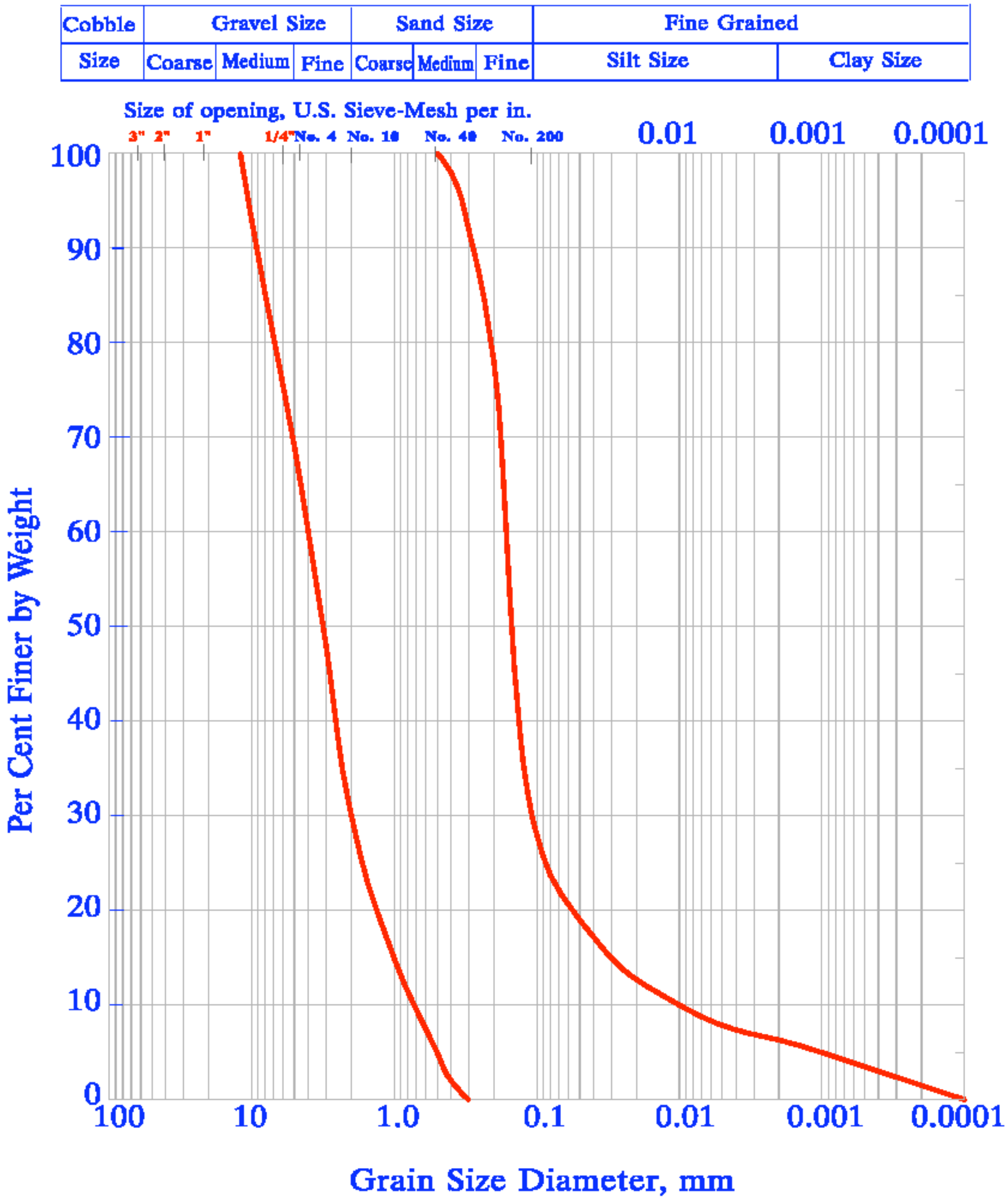
Radial fissures that form around the impact points will facilitate drainage. The method may be used to treat soils both above and below the water table. In granular soils, the effectiveness is controlled mainly by the energy per drop. Use the following relationship to estimate effective depth of influence on compaction:

Equation 10-41: $$D = \frac{Wh}{2}$$

Where:

- D = depth of influence, in feet
- W = falling weight in tons
- h = height of drop in feet

Figure 10-12 Grain Size Ranges Considered for Vibro-Densification



Relative density may be increased to 70 to 90% with relatively uniform increase in density throughout effective depth. Maximum depth of improvement is about 90 feet.

Bearing capacity increases of 200 to 400% can usually be obtained for sands. A minimum treatment area of 4 to 8 acres is necessary for economical use of this method. Currently this method is considered experimental for saturated clays. Because of the high-amplitude, low frequency vibrations (1 to 12 Hz), it is necessary to maintain minimum distances from adjacent facilities as follows:

Piles or bridge abutment	15 to 20 feet
Liquid storage tanks	30 feet
Reinforced concrete building	50 feet
Dwellings	100 feet
Computers (not isolated)	300 feet

10.6.4. Applications of Vibroflotation

Vibroflotation is used to densify granular soils. A crane-suspended cylindrical penetrator about 16 inches in diameter and 6 feet long, called a vibroflot, is attached to an adapter section containing lead wires and hoses. Electrically driven vibrators have RPM's in the order of 1800 to 3000. Hydraulically driven vibrators have variable frequencies. Total weight is generally about two tons.

Power ranges between 30 and 134 hp are available with corresponding centrifugal force ranging from 10 to 31 tons with peak-to-peak amplitudes ranging from 3 to 10 inches.

To sink the vibroflot to the desired treatment depth, a water jet at the tip is opened and acts in conjunction with the vibrations so that a hole can be advanced at a rate of 18 inches per minute. The bottom jet is then closed and the vibroflot is withdrawn at a rate of about one ft/min for 30 hp vibroflots and approximately twice that rate for vibroflots over 100 hp.

Concurrently, a sand or gravel backfill is dumped in from the ground surface and densified. Backfill consumption is at a rate of about 0.5 to 1.5 cubic yards per minute. In partially saturated sands, water jets at the top of the vibroflot can be opened to facilitate liquefaction and densification of the surrounding ground. Most of the compaction takes place within the first 2 to 5 minutes at any elevation.

See Figure 10-13 for guidance on the relationship between vibration centre spacing versus relative density.

Equilateral grid probe patterns are best for compacting large areas, while square and triangular patterns are used for compacting soils for isolated footings. See Table 10-2 as a guide for patterns and spacings required for an allowable pressure of 3 tsf under square footings using a 30 hp unit.

10.6.5. Compaction Grout

Compaction grouting, which is defined as the staged injection of low slump (less than 3 inches) mortar-type grout into soils at high pressures (500 to 600 pounds per square inch), is used to densify loose granular soils. At each grout location, a casing is drilled to the bottom of a previously specified soil target zone. Compaction grout is then pumped into the casing at increments of one lineal foot.

When previously determined criteria are met such as volume, pressure, and heave, pumping will be terminated and the casing will be withdrawn. The casing will be continuously withdrawn by one foot when it meets previously determined criteria until the hole is filled.

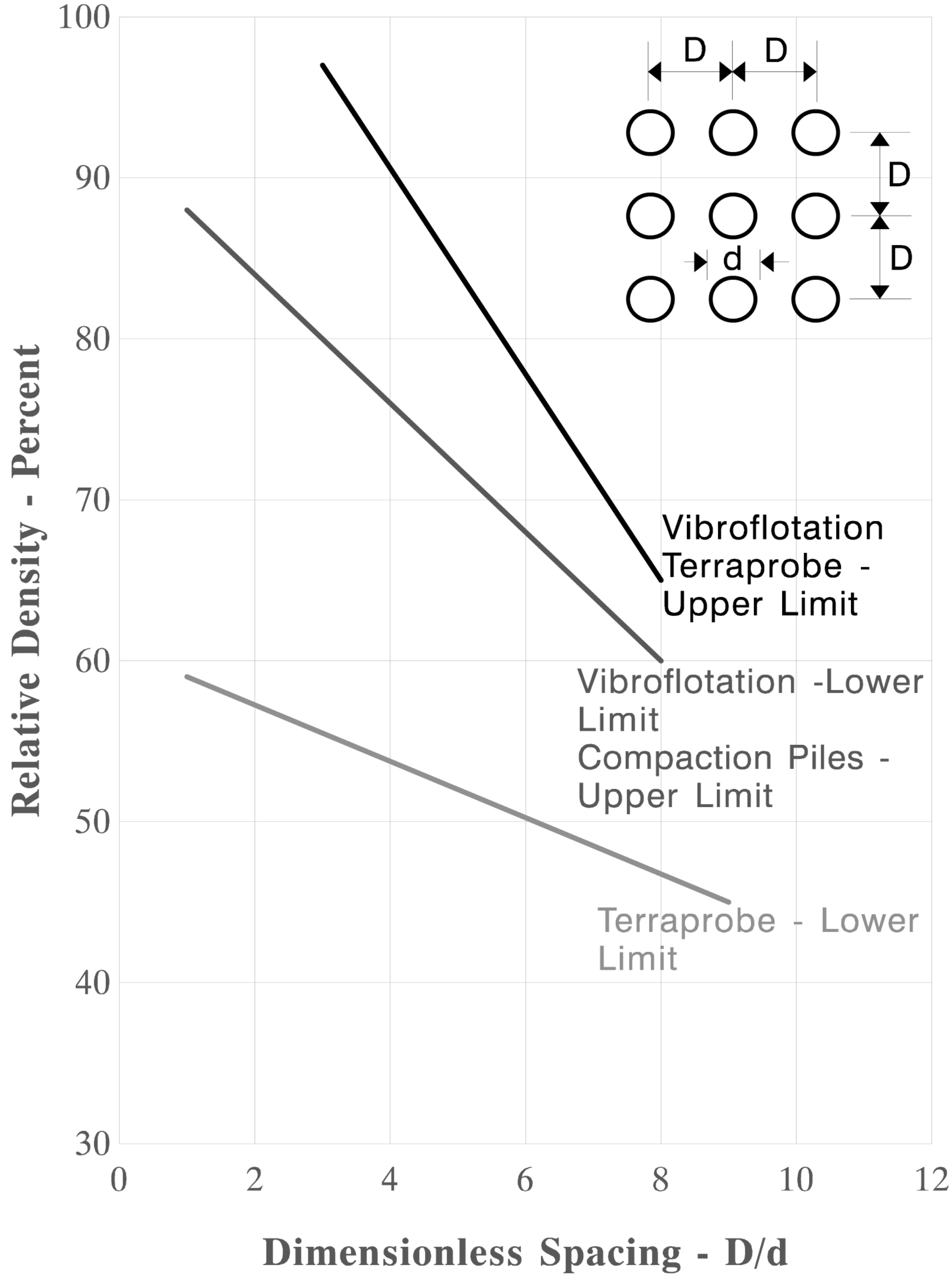
To detect when the grout criteria have been met it is useful to have a strip chart recorder attached to the grout line. A strip chart recorder produces a pressure versus time plot. Such a record, coupled with a known volume of grout delivered per pump piston stroke, can serve as a flow meter. In addition, the pressure versus time plot indicates a pattern in the development of the grout pressures.

Table 10-2 Examples of Vibroflotation Patterns and Spacings for Footings[277]

Square Footing (size - ft)	*Number of Vibroflotation Points*	*C-C Spacing (feet)*	*Pattern*
<4	1	---	---
4.5 - 5.5	2	6	Line
6 – 7	3	7.5	Triangle
7.5 - 9.5	4	6	Square
10 - 11.5	5	7.5	Square plus one @ center

[277] Desired Allowable Bearing Pressure = 3 TSF

Figure 10-13 Relative Density vs. Probe Spacing for Soil Densification



Based on evaluation of subsurface data and proposed structure stresses, areas requiring compaction grouting can be identified, and minimum ground improvement criteria can be established. Ground improvement criteria can be determined based on settlement and bearing capacity analyses. A grouting injection point grid and construction sequence can be formulated once the ground improvement criteria are known. Typically, the grouting points are injected from the perimeter of a grid towards the centre.

10.6.6. Selecting a Method

There are many combinations and variations of the vibratory compaction and vibro-replacement methods. These have been developed by different organizations using various configurations of equipment and procedures. Each method will work well in some circumstances and poorly in others.

In some cases, it may be possible to eliminate several techniques because of the soil type and conditions, but there will usually be several candidate methods remaining.

When selecting a method of dynamic or vibratory compaction, the engineer should bear in mind that, in addition to the usual factors of cost and time to complete the work, the success of the jobs will depend on how effective is the chosen method. In many cases, this will only become evident when an effective technique is employed at the actual site. In some cases, techniques that seemed to be suitable have proven ineffective in practice. The engineer should plan for this contingency. For a large project, a test section may be a wise investment.

§ 11.Glossary and Bibliography

11.1. Glossary

Activity of Clay	The ratio of plasticity index to percent by weight of the total sample that is smaller than 0.002 mm in grain size. This property is correlated with the type of clay material.
Anistropic Soil	A soil mass having different properties in different directions at any given point referring primarily to stress-strain or permeability characteristics.
Capillary Stresses	Pore water pressures less than atmospheric values produced by surface tension of pore water acting on the meniscus formed in void spaces between soil particles.
Clay Size Fraction	That portion of the soil that is finer than 0.002 mm, not a positive measure of the plasticity of the material or its characteristics as clay.
Desiccation	The process of shrinkage or consolidation of the fine-grained soil produced by increase of effective stresses in the grain skeleton accompanying the development of capillary stresses in the pore water.
Dynamic Compaction	The use of high-energy impact to densify loose granular soils *in situ*.
Effective Stress	The net stress across points of contact of soil particles, generally considered as equivalent to the total stress minus the pore water pressure.
Equivalent Fluid Pressure	Horizontal pressures of soil, or soil and water, in combination, which increase linearly with depth and are equivalent to those that would be produced by a heavy fluid of a selected unit weight.
Excess Pore Pressures	That increment of pore water pressures greater than hydro-static values, produced by consolidation stresses in compressible materials or by shear strain.
Exit Gradient	The hydraulic gradient (difference in piezometric levels at two points divided by the distance between them) near to an exposed surface through which seepage is moving.
Flow Slide	Shear failure in which a soil mass moves over a relatively long distance in a fluid like manner, occurring rapidly on flat slopes in loose, saturated, uniform sands, or in highly sensitive clays.
Hydrostatic Pore Pressures	Pore water pressures or groundwater pressures exerted under conditions of no flow where the magnitude of pore pressures increase linearly with depth below the ground surface.
Isotropic Soil	A soil mass having essentially the same properties in all directions at any given point, referring directions at any given point, referring primarily to stress-strain or permeability characteristics.
Liquefaction	The sudden, large decrease of shear strength of a cohesionless soil caused by collapse of the soil structure, produced by shock or small shear strains, associated with sudden but temporary increase of pore water pressure.
Machine Foundation	A foundation that receive regular or irregular vibratory loads that are generated from rotating or impact machinery.
Normal Consolidation	The condition that exists if a soil deposit has never been subjected to an effective stress greater than the existing overburden pressure and if the deposit

	effective stress greater than the existing overburden pressure and if the deposit is completely consolidated under the existing overburden pressure.
Overconsolidation	The condition that exists if a soil deposit has been subjected to an effective stress greater than the existing overburden pressure.
Piezometer	A device installed for measuring the pressure head of pore water at a specific point within the soil mass.
Pile Driving Analyzer (PDA)	The PDA uses electronic measurements and the wave equation analysis method to immediately compute average pile force and velocity.
Piping	The movement of soil particles as the result of unbalanced seepage forces produced by percolating water, leading to the development of boils or erosion channels.
Plastic Equilibrium	The state of stress of a soil mass that has been loaded and deformed to such an extent that its ultimate shearing resistance is mobilized at one or more points.
Positive Cut-off	The provision of a line of tight sheeting or a barrier of impervious material extending downward to an essentially impervious lower boundary to intercept completely the path of subsurface seepage.
Primary Consolidation	The compression of the soil under load that occurs while excess pore pressures dissipate with time.
Response Spectrum	Useful information regarding frequency dependent energy distribution of an earthquake derived from Fourier analysis.
Rippability	The characteristic of dense and rocky soils that can be excavated without blasting after ripping with a rock rake or ripper.
Slickensides	Surfaces with a soil mass that have been smoothed and striated by shear movements on these surfaces.
Standard Penetration Resistance	The number of blows of a 140-pound hammer, falling 30 inches, required to advance a 2” O.D. split barrel sampler 12 inches through a soil mass.
State-of-the-Art	The scientific and technical level attained at a given time.
Tolerable Vibration	The level of vibration magnitude, ranging from “not noticeable to persons” to “danger to structures” that a structure is designed.
Total Stress	At a given point in a soil mass the sum of the net stress across contact points of soil particles (effective stress) plus the pore water pressure at the point.
Underconsolidation	The condition that exists if a soil deposit is not fully consolidated under the existing overburden pressure and excess hydrostatic pore pressures exist within the material.
Varved Silt or Clay	A fine-grained glacial lake deposit with alternating thin layers of silt or fine sand and clay, formed by variations in sedimentation from winter to summer during the year.
Vibrodensification	The densification or compaction of cohesionless soils by imparting wave energy to the soil mass to rearrange soil particles resulting in fewer voids in the overall mass.
Vibroflot	A crane-suspended cylindrical penetrator with a water jet at the tip that is opened and acts in conjunction with vibrations to dig a hole.

Vibroflotation	A method to densify granular soils using a vibroflot to dig a hole and then backfilled with sand or gravel that is dumped in from the surface and densified.

11.2. Data Sheets for Laboratory Tests

Figure 11-1 Water Content – General: Data Sheet

WATER CONTENT - GENERAL DATE ____________

PROJECT ____________

BORING NO. ____________

Sample or Specimen No.								
Tare No.								
Weight in grams	Tare plus wet soil							
	Tare plus dry soil							
	Water	W_w						
	Tare							
	Dry soil	W_s						
Water content		w	%	%	%	%	%	%

Sample or Specimen No.								
Tare No.								
Weight in grams	Tare plus wet soil							
	Tare plus dry soil							
	Water	W_w						
	Tare							
	Dry soil	W_s						
Water content		w	%	%	%	%	%	%

Sample or Specimen No.								
Tare No.								
Weight in grams	Tare plus wet soil							
	Tare plus dry soil							
	Water	W_w						
	Tare							
	Dry soil	W_s						
Water content		w	%	%	%	%	%	%

$$w\% = \frac{(\text{tare plus wet soil}) - (\text{tare plus dry soil})}{(\text{tare plus dry soil}) - (\text{tare})} \times 100 = \frac{W_w}{W_s} \times 100$$

Remarks ____________

____

Technician ____________ Computed by ____________ Checked by ____________

Figure 11-2 Data Sheet for Volumetric Method

UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION
(VOLUMETRIC METHOD)

DATE ____________

PROJECT ____________

BORING NO. ____________

WATER CONTENT

SAMPLE OR SPECIMEN NO.							
TARE NO.							
WEIGHT IN GRAMS	TARE PLUS WET SOIL						
	TARE PLUS DRY SOIL						
	WATER	W_w					
	TARE						
	DRY SOIL	W_s					
WATER CONTENT		w	%	%	%	%	

WEIGHT-VOLUME RELATIONS

SAMPLE OR SPECIMEN NO.						
CYLINDER NO.						
CENTI-METERS	HEIGHT OF CYLINDER	H				
	INSIDE DIAMETER OF CYLINDER	D				
WEIGHT IN GRAMS	WET SOIL AND TARE					
	TARE					
	WET SOIL	W				
	DRY SOIL†	W_s				
SPECIFITY GRAVITY OF SOIL		G_s				
VOLUME IN CC	WET SOIL (VOLUME OF CYLINDER)	V				
	DRY SOIL = W_s/G_s	V_s				
LB PER CU FT	WET UNIT WT = (W/V) 62.4	γ_m				
	DRY UNIT WT = (W_s/V) 62.4	γ_d				
VOID RATIO = $(V - V_s)/V_s$		e				
POROSITY, % = $[(V - V_s)/V] \times 100$		n	%	%	%	%
DEGREE OF SATURATION, % = $[V_w/(V - V_s)] \times 100$		S	%	%	%	%

VOLUME OF CYLINDER, $V = \frac{\pi D^2 H}{4}$

VOLUME OF WATER = $V_w = \frac{W_w}{\text{SPECIFIC GRAVITY OF WATER‡}}$

† IF NOT MEASURED ON ENTIRE SPECIMEN, DRY WEIGHT MAY BE COMPUTED AS FOLLOWS: $W_s = \frac{W}{1 + 0.01\,w}$

‡ SPECIFIC GRAVITY OF WATER IN METRIC SYSTEM = 1 (APPROX)

REMARKS ____________

TECHNICIAN ____________ COMPUTED BY ____________ CHECKED BY ____________

Figure 11-3 Data Sheet for Displacement Method

UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION
(DISPLACEMENT METHOD)

DATE__________

PROJECT__________

BORING NO.__________

WATER CONTENT

SAMPLE OR SPECIMEN NO						
TARE NO						
WEIGHT IN GRAMS	TARE PLUS WET SOIL					
	TARE PLUS DRY SOIL					
	WATER	W_w				
	TARE					
	DRY SOIL	W_s				
WATER CONTENT		w	%	%	%	%

WEIGHT-VOLUME RELATIONS

SAMPLE OR SPECIMEN NO.						
TEST TEMPERATURE OF WATER, T, C						
WEIGHT IN GRAMS	SOIL AND WAX IN AIR					
	WET SOIL	W				
	WAX					
	WET SOIL AND WAX IN WATER					
	DRY SOIL†	W_s				
SPECIFIC GRAVITY OF SOIL		G_s				
VOLUME IN CC	WET SOIL AND WAX‡					
	WAX					
	WET SOIL	V				
	DRY SOIL = W_s/G_s	V_s				
LB PER CU FT	WET UNIT WT = (W/V) 62.4	γ_m				
	DRY UNIT WT = (W_s/V) 62.4	γ_d				
VOID RATIO = $(V - V_s)/V_s$		e				
POROSITY, % = $[(V - V_s)/V] \times 100$		n	%	%	%	%
DEGREE OF SATURATION, % = $[V_w/(V - V_s)] \times 100$		S	%	%	%	%

VOLUME OF WAX = $\frac{\text{WEIGHT OF WAX}}{\text{SPECIFIC GRAVITY OF WAX}}$ = __________

† IF NOT MEASURED DIRECTLY, MAY BE COMPUTED AS FOLLOWS: $W_s = \frac{W}{1 + 0.01\,w}$

‡ VOLUME OF WET SOIL AND WAX = $\frac{\left(\text{WEIGHT OF WET SOIL AND WAX IN AIR}\right) - \left(\text{WEIGHT OF WET SOIL AND WAX IN WATER}\right)}{\text{DENSITY OF WATER AT TEST TEMPERATURE}}$

REMARKS__________

TECHNICIAN__________ COMPUTED BY__________ CHECKED BY__________

Figure 11-4 Specific Gravity Tests

SPECIFIC GRAVITY TESTS

Date ______

Project ______

Boring No. ______

SPECIFIC GRAVITY OF SOLIDS (G_s)

Sample or Specimen No.						
Flask No.						
Temperature of water and soil, T, °C						
Dish No.						
Weight in grams	Dish + dry soil					
	Dish					
	Dry soil	W_s				
	Flask + water at T, °C	W_{bw}				
	$W_s + W_{bw}$					
	Flask + water + immersed soil	W_{bws}				
	Displaced water, $W_s + W_{bw} - W_{bws}$					
Correction factor		K				
$(W_sK) \div (W_s + W_{bw} - W_{bws})$		G_s				

APPARENT (G_a) AND BULK (G_m) SPECIFIC GRAVITY

Sample or Specimen No.						
Temperature of water and soil, T, °C						
Weight in grams	Tare + saturated surface-dry soil					
	Tare					
	Saturated surface-dry soil	B				
	(Wire basket + soil) in water					
	Wire basket in water					
	Saturated soil in water	C				
	Tare + dry soil					
	Tare					
	Dry soil	A				
Correction factor		K				
$(AK) \div (A - C)$ (Apparent)		G_a				
$(AK) \div (B - C)$ (Bulk)		G_m				

Remarks ______

Technician ______ Computed by ______ Checked by ______

Figure 11-5 Form for Liquid and Plastic Limit Tests

LIQUID AND PLASTIC LIMIT TESTS

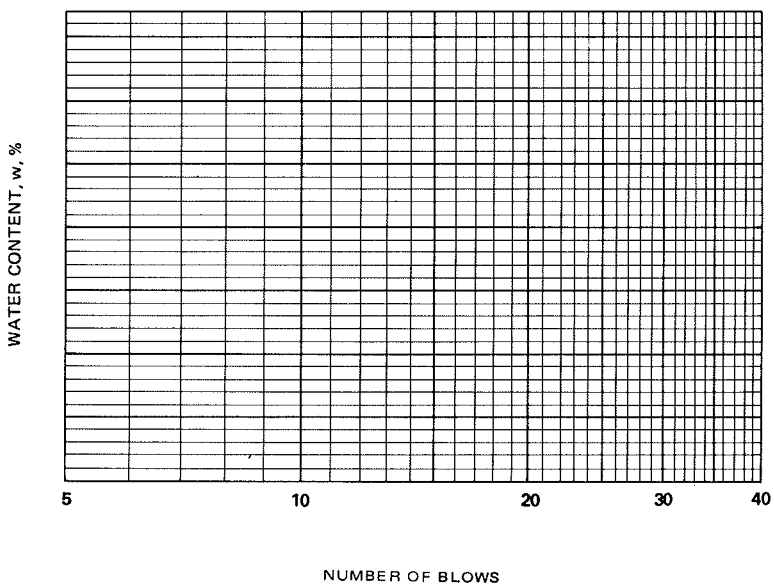
DATE ____________________

PROJECT ____

BORING NO. ______________________________ SAMPLE NO. ____________________

LIQUID LIMIT

			1	2	3	4	5	6
RUN NO.			1	2	3	4	5	6
TARE NO.								
WEIGHT IN GRAMS	TARE PLUS WET SOIL							
	TARE PLUS DRY SOIL							
	WATER	W_w						
	TARE							
	DRY SOIL	W_s						
WATER CONTENT, %		w						
NUMBER OF BLOWS								



LL ____________________

PL ____________________

PI ____________________

Symbol from plasticity chart

____

PLASTIC LIMIT

			1	2	3	4	5	NATURAL WATER CONTENT
RUN NO.			1	2	3	4	5	
TARE NO.								
WEIGHT IN GRAMS	TARE PLUS WET SOIL							
	TARE PLUS DRY SOIL							
	WATER	W_w						
	TARE							
	DRY SOIL	W_s						
WATER CONTENT, %		w						
PLASTIC LIMIT								

REMARKS ____

TECHNICIAN ____________________ COMPUTED BY ____________________ CHECKED BY ____________________

Figure 11-6 Plasticity Chart

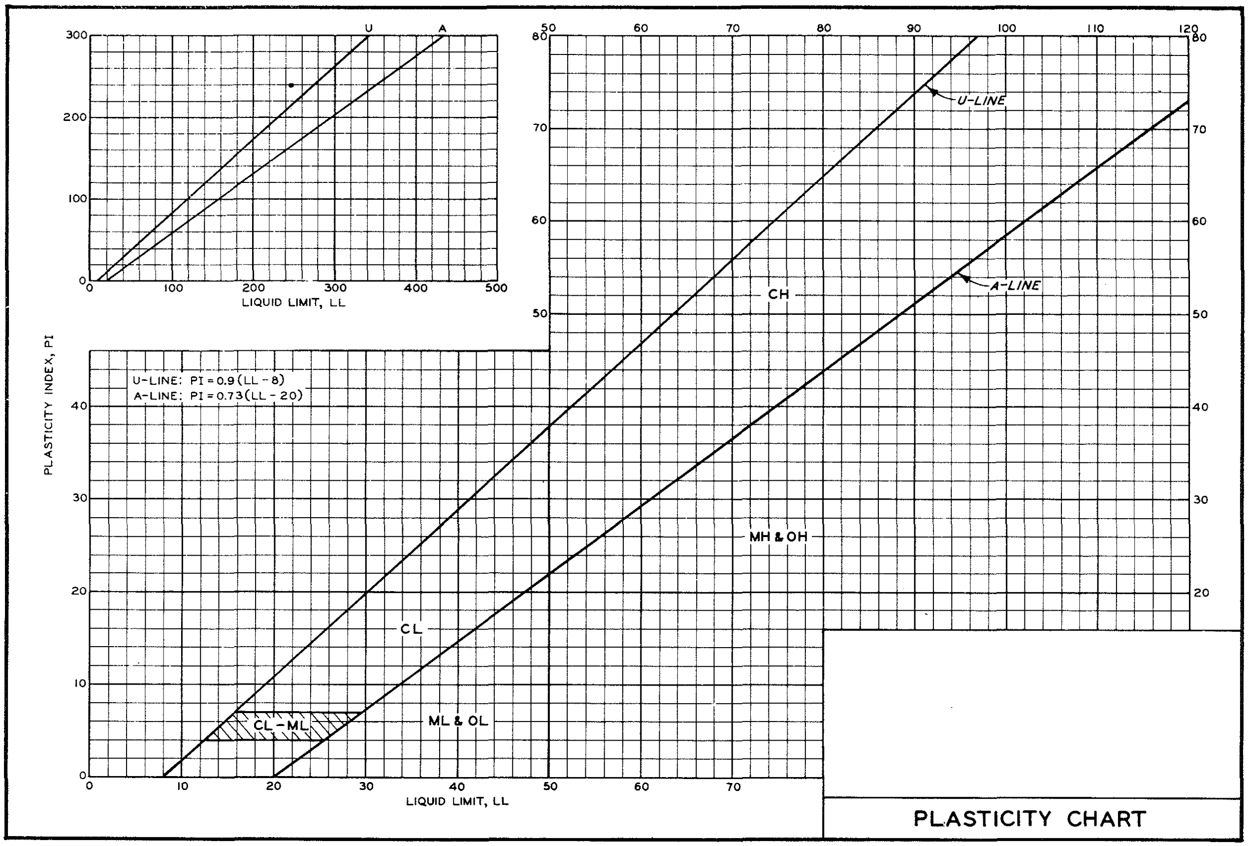


Figure 11-7 Sieve and Hydrometer Analysis

SIEVE AND HYDROMETER ANALYSIS

PART I – SIEVE ANALYSIS | DATE

PROJECT

BORING NO. | SAMPLING NO.

TOTAL WEIGHT IN GRAMS OF SAMPLE, W_S = ________ | WEIGHT IN GRAMS OF MATERIAL >NO. 4 SIEVE = ________

STEVE OPENINGS		U.S. STANDARD SIEVE SIZE OR NUMBER	WEIGHT RETAINED IN GRAMS	PERCENT RETAINED		PERCENT FINER BY WEIGHT
INCHES	MILLIMETERS			PARTIAL	TOTAL	
3.00		3-in.				
2.00		2-in.				
1.50		1-1/2-in.				
1.00	25.4	1-in.				
0.750	19.1	3/4-in.				
0.500	12.7	1/2-in.				
0.375	9.52	3/8-in.				
0.250	6.35	No. 3				
0.187	4.76	No. 4				
		Pan				
0.132	3.36	No. 6				
0.094	2.38	No. 8				
0.079	2.00	No. 10				
0.047	1.19	No. 16				
0.033	0.84	No. 20				
0.023	0.59	No. 30				
0.0165	0.42	No. 40				
0.0117	0.297	No. 50				
0.0083	0.210	No. 70				
0.0059	0.149	No. 100				
0.0041	0.105	No. 140				
0.0029	0.074	No. 200				
		Pan				
TOTAL WEIGHT IN GRAMS						

$$\text{Partial percent retained} = \frac{\text{wt in grams retained on a sieve}}{\text{wt in grams of sample used for a given series of sieves}} \times 100$$

$$\text{Total percent retained} = \frac{\text{wt in grams retained on a sieve}}{\text{total wt in grams of oven-dry sample}} \times 100$$

For an individual sieve, the percent finer by weight = percent finer than next larger sieve — percent retained on individual sieve

REMARKS

TECHNICIAN ________ COMPUTED BY ________ CHECKED BY ________

TECHNICIAN	COMPUTED BY	CHECKED BY

PART II – HYDROMETER ANALYSIS		DATE
PROJECT		
BORING NO.:		
SAMPLE OR SPECIMEN NO.	CLASSIFICATION	
DISH NO.	GRADUATE NO.	HYDROMETER NO.
DISPERSING AGENT USED		QUANTIY
DISPERSING AGENT CORRECTION, C_d =	MENISCUS CORRECTION, C_m =	

TIME	ELAPSED TIME MIN	TEMP °C	HYDRO. READING (R')	CORRECTED READING (R)	PARTICLE DIAMETER (D), MM	TEMP CORRECTION (m)	$R - C_d + m$	PERCENT FINER	
								PARTIAL	TOTAL

WEIGHT IN GRAMS			
	DISH PLUS DRY SOIL		
	DISH		
	DRY SOIL	W_o	

Specific gravity of solids, G_s =

Corrected hydrometer reading (R) = hydrometer reading (R') + C_m

The particle diamter (D) is calculated from Stoke's equation using corrected hydrometer reading. Use nomographic chart for solution of Stocke's equation.

Hydrometer graduated in specific gravity

$$\text{Partial percent finer} = \frac{G_s}{G_s - 1} \times \frac{100}{W_o}(R - C_d + m)$$

Hydrometer graduated in grams per liter

$$\text{Partial percent finer} = \frac{100}{W_o}(R - C_d + m)$$

$$\text{Total percent finer} = \text{partial percent finer} \times \frac{W_s - W_1}{W_s}$$

W_s = total oven-dry wt of sample used for combined analysis

W_o = oven-dry wt in grams of soil used for hydrometer analysis

W_1 = oven-dry wt of sample retained on No. 200 sieve

REMARKS ____

____

TECHNICIAN	COMPUTED BY	CHECKED BY

Figure 11-8 Relative Density Data Sheet

RELATIVE DENSITY

MINIMUM AND MAXIMUM DENSITY DETERMINATIONS

DATE ____________

PROJECT ____________

BORING NO. ____________ SAMPLE NO. ____________

CLASSIFICATION		
MOLD NO.	MOLD DIAMETER, IN. =	WEIGHT OF MOLD, W_m, LB =
VOLUME OF MOLD, V_m, CU FT =		END AREA OF MOLD, A_m, SQ FT =

MINIMUM DENSITY

TRIAL NO.			1	2	3	4
WEIGHT LB	MOLD (OR TARE) AND SOIL, DRY	W				
	MOLD (OR TARE)	W_m				
	SOIL, DRY	W_s				
MIN DRY DENSITY, LB/CU FT = W_s/V		γ_d				
MIN DRY DENSITY, AVERAGE			LB/CU FT			

MAXIMUM DENSITY

METHOD USED

TRIAL NO:			1	2	3	4
HEIGHT, IN.	LEFT DIAL READING	h_L				
	RIGHT DIAL READING	h_R				
	AVERAGE DIAL = $(h_L + h_R)/2$	h_{avg}				
	INITIAL DIAL READING	h_o				
	HEIGHT CHANGE = $h_o - h_{avg}$	Δh				
VOL CU FT	VOLUME CHANGE = $(\Delta h/12) \times A_m$	ΔV				
	VOLUME OF SOIL = $V_m - \Delta V$	V				
WEIGHT LB	MOLD (OR TARE) AND SOIL, DRY	W				
	MOLD (OR TARE)	W_m				
	SOIL, DRY	W_s				
MAX DRY DENSITY, LB/CU FT = W_s/V		γ_d				
MAX DRY DENSITY, AVERAGE			LB/CU FT			

REMARKS ____________

TECHNICIAN ____________ COMPUTED BY ____________ CHECKED BY ____________

Figure 11-9 Direct Shear Test Specimen Data

DIRECT SHEAR TEST
(Specimen Data)

Date ____________

Project ____

Boring No. ____________________ Sample No. ____________________

Shear Box No.	Normal Stress T/sq ft
Specimen No.	Classification

			Before Test				After Test	
			Specimen		Trimmings		Specimen	
Tare No.			Cutter and Glass Plates					
Weight in grams	Tare plus wet soil							
	Tare plus dry soil							
	Water	W_w			W_{wo}		W_{wf}	
	Tare							
	Wet soil	W						
	Dry soil	W_s						
Water content		w		%	w_o	%	w_f	%

Initial Condition of Specimen					
Area in sq cm	A		Volume of solids in cc	V_s	
Height in cm	H_o		Void ratio = $(V_o - V_s) \div V_s$	e_o	
Volume in cc = $A \times H_o$	V_o		Saturation, %	S_o	
Specific gravity of solids	G_s		Dry density in lb/cu ft	γ_d	

Condition of Specimen After Consolidation					
Change in height during consolidation, in.	ΔH_o		Volume in cc = $A \times H_c$	V_c	
Height in cm = $H_o - 2.54\Delta H_o$	H_c		Void ratio = $(V_c - V_s) \div V_s$	e_c	

Condition of Specimen After Test					
Change in height during shear test, in.	ΔH		Volume in cc = $A \times H_f$	V_f	
Height in cm = $H_c - 2.54\Delta H$	H_f		Void ratio = $(V_f - V_s) \div V_s$	e_f	
Saturation, %	S_f				

$$W_s = \frac{W}{1 + \frac{w_o}{100}} , \quad V_s = \frac{W_s}{G_s} , \quad S_o = \frac{\frac{w_o}{100} \times \frac{W_s}{\gamma_w}}{V_o - V_s} \times 100 , \quad S_f = \frac{\frac{w_f}{100} \times \frac{W_s}{\gamma_w}}{V_f - V_s} \times 100 , \quad \gamma_d = \frac{W_s}{V_o} \times 62.4$$

Remarks ____

____

Technician ____________ Computed by ____________ Checked by ____________

Figure 11-10 Direct Shear Test, Time-Stress Shear Data Sheet

DIRECT SHEAR TEST

(Time-Shear Stress Data) Date ______________

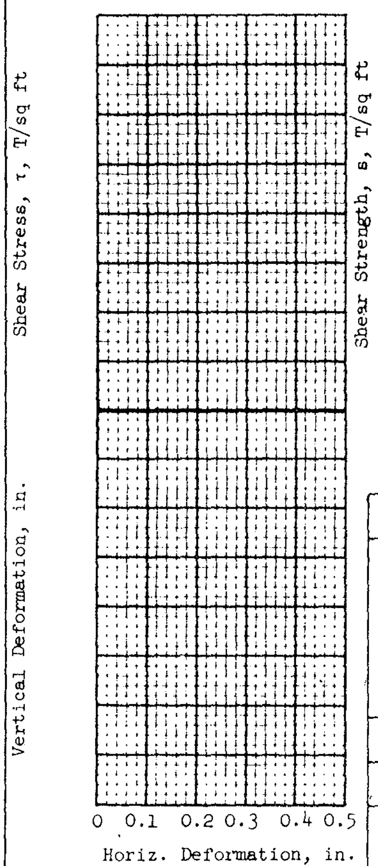
Project ____

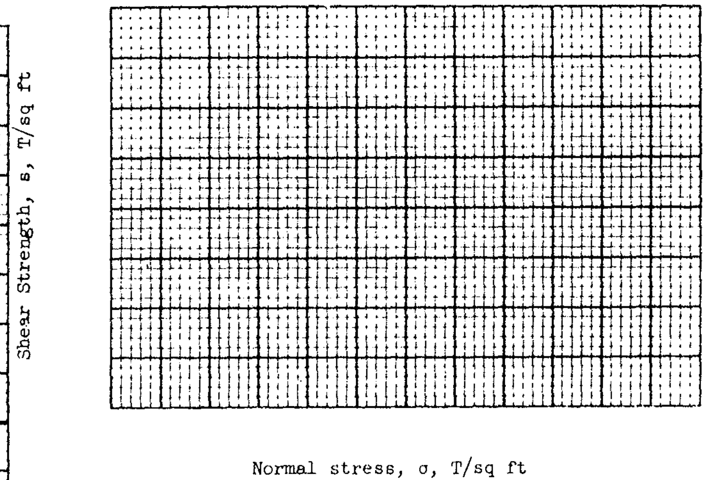
Boring No. ______________ **Sample No.** ______________ **Normal Stress** ______________

Time	Elapsed Time min	Time Intervals min	Proving Ring Dial Reading 10^{-} in.	Shear Stress T/sq ft	Vertical Dial Reading 10^{-} in.	Horizontal Dial Reading 10^{-} in.

Technician ______________

Figure 11-11 Direct Shear Test Report





Test No.						
Initial	Water content	w_o	%	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d				
Void ratio after consolidation		e_c				
Time for 50 percent consolidation, min		t_{50}				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
	Saturation	S_f	%	%	%	%
Normal stress, T/sq ft		σ				
Maximum shear stress, T/sq ft		τ_{max}				
Actual time to failure, min		t_f				
Rate of strain, in./min						
Ultimate shear stress, T/sq ft		τ_{ult}				

Shear Strength Parameters

ϕ' = ______ °

tan ϕ' = ______

= ______ T/sq ft

☐ Controlled stress

☐ Controlled strain

Type of specimen	in. square	in. thick
Classification		

LL	PL	PI		G_s

Remarks ______	Project	
______		
______	Area	
______	Boring No.	Sample No.
______	Depth El	Date
	DIRECT SHEAR TEST REPORT	

Figure 11-12 Triaxial Compression Test Specimen Data Sheet

TRIAXIAL COMPRESSION TEST
(SPECIMEN DATA)

Date ____________

Project ____

Boring No. ____________________ Sample No. ____________________

Type of Test	Confining Pressure tons/sq ft
Test No.	Classification

			Before Test			After Test	
			Specimen	Trimmings		Specimen	
Tare No.							
Weight, g	Tare plus wet soil						
	Tare plus dry soil						
	Water	W_w		W_{wo}		W_{wf}	
	Tare						
	Wet soil	W					
	Dry soil	W_s					
Water content		w	%	w_o	%	w_f	%

Initial Condition of Specimen

Diameter, cm	D_o	Top	Center	Bottom	Avg

Height, cm	H_o		Volume of solids, cc	V_s	
Area, sq cm = $0.7854\ D_o^2$	A_o		Void ratio = $(V_o - V_s) \div V_s$	e_o	
Volume, cc = H_oA_o	V_o		Saturation, %	S_o	
Specific gravity of solids	G_s		Dry density, lb/cu ft	γ_d	

Condition of Specimen After Consolidation (R and S Tests)

Change in height during consolidation, in.	ΔH_o		Volume, cc = A_cH_c	V_c	
Height, cm = $H_o - 2.54\Delta H_o$	H_c		Void ratio = $(V_c - V_s) \div V_s$	e_c	
Area, sq cm	A_c		Saturation, %	S_c	

Condition of Specimen After Test (R and S Tests)

Diameter, cm	D_f	Top	Center	Bottom	Avg

Change in height during shear test, in.	ΔH		Volume, cc = A_fH_f	V_f	
Height, cm = $H_c - 2.54\Delta H$	H_f		Void ratio = $(V_f - V_s) \div V_s$	e_f	
Area, sq cm	A_f		Saturation, %	S_f	

$$W_s = \frac{W}{1 + \frac{w_o}{100}}\ ,\quad V_s = \frac{W_s}{G_s}\ ,\quad S_o = \frac{\frac{w_o}{100} \times \frac{W_s}{\gamma_w}}{V_o - V_s} \times 100\ ,\quad S_c = \frac{\frac{w_c}{100} \times \frac{W_s}{\gamma_w}}{V_c - V_s} \times 100\ ,$$

$$S_f = \frac{\frac{w_f}{100} \times \frac{W_s}{\gamma_w}}{V_f - V_s} \times 100\ ,\quad \gamma_d = \frac{W_s}{V_o} \times 62.4\ ,\quad A_c = A_o\,\frac{H_o - 2\Delta H_o}{H_o}$$

Remarks ____

____

Technician ____________ Computed By ____________ Checked by ____________

Figure 11-13 Triaxial Compression (Q and R) Test Axial Loading Data Sheet

TRIAXIAL COMPRESSION (Q AND R) TEST
AXIAL LOADING DATA

Date ____________

Project ____________________________

Boring No. ____________ Sample No. ____________ Test No. ____________

Type Test ____________________ Confining Pressure ____________ tons/sq ft

Time	Elapsed Time min	Dial Reading 10^{-} in.	Cumulative Change (ΔH) 10^{-} in.	P Axial Load lb	Axial Strain* $\epsilon = \frac{\Delta H}{H}$	$1 - \epsilon$	$A_{corr} = \frac{A}{1-\epsilon}$ ** sq cm	Deviator Stress = $\frac{P}{A_{corr}} \times 0.465$ tons/sq ft

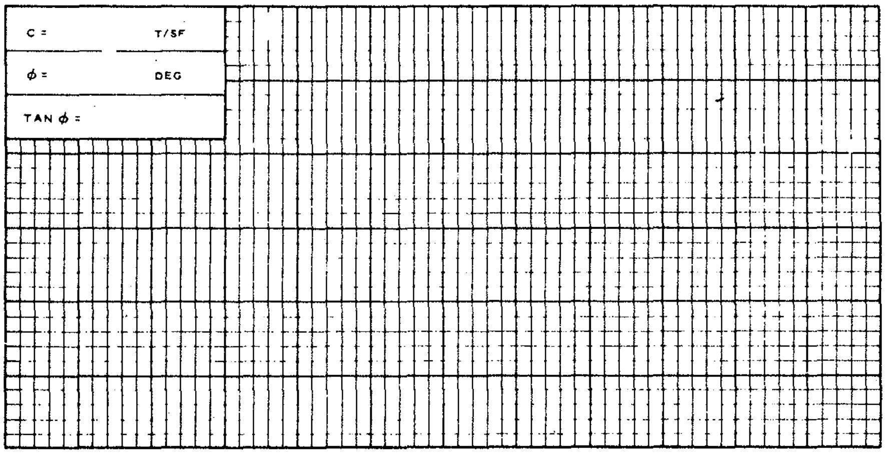
* Use H_o for Q tests and H_c for R tests.
** Use A_o for Q tests and A_c for R tests.

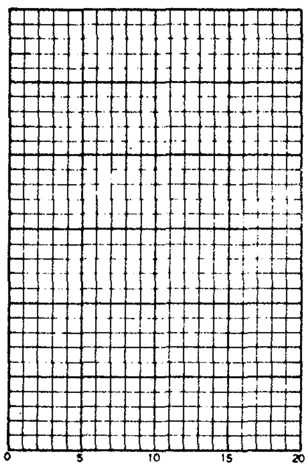
H_o ______ cm = ______ in. A_o ______ sq cm
H_c ______ cm = ______ in. A_c ______ sq cm

Test time to failure ____________ min Type failure ____________________

Technician ____________________

Figure 11-14 Triaxial Compression Test Report





SPECIMEN NO.						
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/CU FT	γ_{d_o}				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3				
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$				
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST

REMARKS:	PROJECT	
	BORING NO.	SAMPLE NO.
	DEPTH/ELEV	
	LABORATORY	DATE
	TRIAXIAL COMPRESSION TEST REPORT	

Figure 11-15 Triaxial Test, Back Pressure and Pore Pressure Data, Saturation

TRIAXIAL COMPRESSION TEST
(BACK PRESSURE AND PORE PRESSURE DATA)
(Saturation)

Date__________

Project__________ Boring No.__________ Sample No.__________

Test No.__________ Consolidation Pressure__________tons/sq ft =______ psi

Increment No.	Time	Elapsed Time min	Chamber Pressure psi	Applied Back Pressure psi	Measured Pore Pressure psi	Burette Reading cc	Volume Change (ΔV) cc

Technician__________

Figure 11-16 Triaxial Compression (R and S) Test, Preliminary Consolidation

TRIAXIAL COMPRESSION (R AND S) TEST
PRELIMINARY CONSOLIDATION

Date ____________

Project ____

Boring No. ____________ Sample No. ____________ Test No. ____________

Test Type ____________________ Consolidation Pressure ____________ tons/sq ft

Date and Time	Elapsed Time min	Burette Reading cc	Volume Change (ΔV) cc	Dial Indicator Reading 10^{-} in.	Date and Time	Elapsed Time min	Burette Reading cc	Volume Change (ΔV) cc	Dial Indicator Reading 10^{-} in.

Dial indicator reading before consolidation ____________ in.
Dial indicator reading after consolidation ____________ in.
Change in height during consolidation (ΔH_o) ____________ in. = ______ cm

Remarks ____

____

Technician ____________

Figure 11-17 Triaxial Compression (S) Test, Axial Load Data Sheet

TRIAXIAL COMPRESSION (S) TEST
AXIAL LOADING DATA

Date ____________

Project ____________

Boring No. ____________ Sample No. ____________ Test No. ____________

Specimen No. ____________ Consolidation Pressure ____________ tons/sq ft

Date and Time	Dial Reading 10^{-} in.	Cumulative Change (ΔH) 10^{-} in.	Burette Reading cc	Volume Change (ΔV) cc	P Axial Load lb	Axial Strain $\epsilon = \frac{\Delta H}{H_c}$	$C\epsilon$	$1 - C\epsilon$	$A'_{corr} = \frac{A_c}{1 - C\epsilon}$ sq cm	Stress $\frac{P}{A'_{corr}} \times 0.465$ tons/sq ft

Area of specimen after test, A_f = ________ sq cm   Axial strain at end of test, ϵ_e = ________

Area of specimen at end of test, $A_e = \frac{A_c}{1 - \epsilon_e}$ = ________ sq cm $C = \frac{A_f}{A_e}$ = ________

Test time to failure, t_f = ________

Remarks ____________

____

____

____

Technician ____________

Figure 11-18 Unconfined Compression Test Data Sheet

UNCONFINED COMPRESSION TEST

Date ____________

Project ____

Boring No. ____________________ Sample No. ____________________

Specimen No.				Classification				
Tare No.				Height	H_o	in.	H_o	cm
Weight, g	Tare plus wet specimen			Average diameter	D_o	in.	D_o	cm
Weight, g	Tare plus dry specimen			Initial area	A_o	sq in.	A_o	sq cm
Weight, g	Water	W_w		Volume in cc = A_oH_o			V_o	
Weight, g	Tare			Volume of solids in cc = $W_s + G_s$			V_s	
Weight, g	Wet specimen	W		Void ratio = $(V_o - V_s) + V_s$			e_o	
Weight, g	Dry specimen	W_s		Saturation in % = $G_s w_o + e_o$			S_o	%
Water content		w_o	%	Dry dens., lb/cu ft = $62.4\ (W_s + V_o)$			γ_d	
Specific gravity of solids		G_s		Note: If only a portion of the specimen is used for the water content determination, $W_s = W + \left(1 + \frac{w_o}{100}\right)$				

Elapsed Time min	Dial Reading 10^{-} in.	Cumulative Change, ΔH 10^{-} in.	Proving Ring Dial Reading 10^{-} in.	Axial Load, P lb	Axial Strain $\epsilon = \Delta H/H_o$	$1 - \epsilon$	$A_{corr} = A_o/(1-\epsilon)$ sq cm	Compr Stress tons/sq ft $0.465(P/A_{corr})$

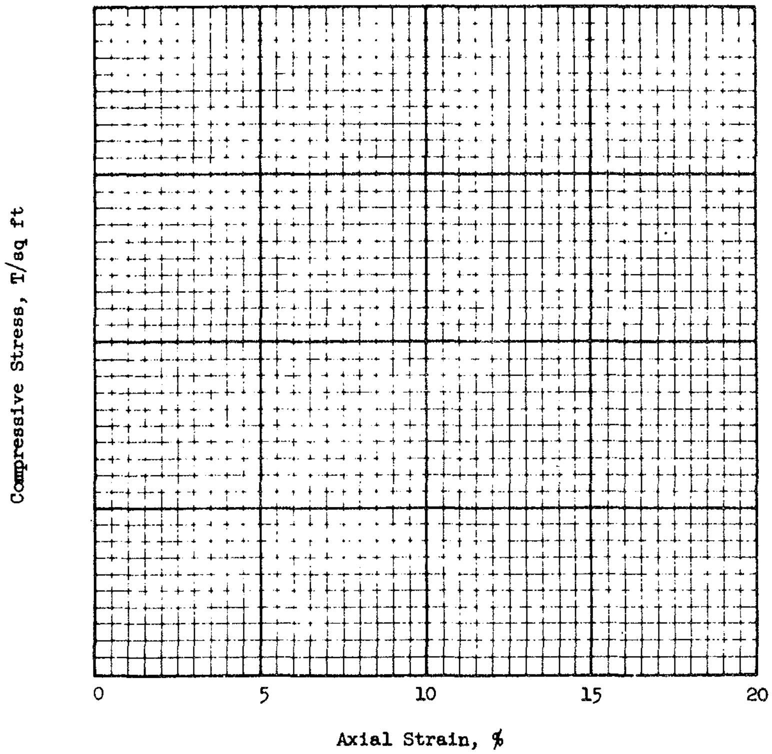
Remarks ____

______________________________________ Type Failure ____________

Technician ____________ Computed by ____________________ Checked by ____________

Figure 11-19 Unconfined Compression Test Failure Sketches

Failure Sketches



☐ Controlled stress

☐ Controlled strain

Test No.						
Type of specimen						
Initial	Water content	w_o	%	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d				
Time to failure, min		t_f				
Unconfined compressive strength, T/sq ft		q_u				
Undrained shear strength, T/sq ft		s_u				
Sensitivity ratio		S_t				
Initial specimen diameter, in.		D_o				
Initial specimen height, in.		H_o				

Classification			
LL	PL	PI	G_s

Remarks	Project	
	Area	
	Boring No.	Sample No.
	Depth El	Date
	UNCONFINED COMPRESSION TEST REPORT	

Figure 11-20 Constant Head Permeability Test Data Sheet

CONSTANT-HEAD PERMEABILITY TEST

DATE ____________

PROJECT ____________

BORING NO. ____________

Sample or Specimen No.						
Wt in grams	Tare plus dry soil			Diameter of specimen, cm	D	
	Tare			Area of specimen, sq cm	A	
	Dry soil	W_s		Initial height of specimen, cm	L	
Specific gravity		G		Initial vol of spec, cc = AL	V	
Vol of solids, cc = $W_s \div G$		V_s		Initial void ratio $= (V - V_s) \div V_s$	e	
Distance between piezometer taps, cm					L_1	

Test No.		1		2		3	
Height of specimen, cm	L						
Void ratio $= (AL - V_s) \div V_s$	e						
		1a	1b	2a	2b	3a	3b
Reading of piez 1, cm	h_1						
Reading of piez 2, cm	h_2						
Head loss, cm = $h_1 - h_2$	h						
Quantity of flow, cc	Q						
Elapsed time, sec	t						
Water temperature, °C	T						
Viscosity correction factor(1)	R_T						
Coefficient of permeability,(2) cm/sec	k_{20}						
	Avg						

(1) Correction factor for viscosity of water at 20 C obtained from table VII-1.

(2) $k_{20} = \frac{Q \times L \times R_T}{h \times A \times t}$.

where L = height of specimen or distance between piezometer taps if used.

Remarks ____________

Technician ____________ Computed by ____________ Checked by ____________

Figure 11-21 Falling Head Permeability Test Data Sheet

FALLING-HEAD PERMEABILITY TEST

DATE ______

PROJECT ______

BORING NO. ______

SAMPLE OR SPECIMEN NO.					
WT IN GRAMS: TARE PLUS DRY SOIL			DIAMETER OF SPECIMEN, CM	D	
WT IN GRAMS: TARE			AREA OF SPECIMEN, SQ CM	A	
WT IN GRAMS: DRY SOIL	W_s		INITIAL HEIGHT OF SPECIMEN, CM	L	
SPECIFIC GRAVITY	G		INITIAL VOL OF SPEC, CC = AL	V	
VOL OF SOLIDS, CC = $W_s \div G$	V_s		INITIAL VOID RATIO = $(V - V_s) \div V_s$	e	
AREA OF STANDPIPE, SQ CM	a		CONSTANT = $(2.303 \times a) \div A$	C	

TEST NO.		1		2		3	
HEIGHT OF SPECIMEN, CM	L						
VOID RATIO = $(AL - V_s) \div V_s$	e						
		1a	1b	2a	2b	3a	3b
INITIAL TIME	t_o						
FINAL TIME	t_f						
ELAPSED TIME, SEC = $t_f - t_o$	t						
INITIAL HEAD, CM	h_o						
FINAL HEAD, CM	h_f						
LOG $(h_o \div h_f)$							
WATER TEMPERATURE, °C	T						
VISCOSITY CORRECTION FACTOR(1)	R_T						
COEFFICIENT OF PERMEABILITY,(2) CM/SEC	k_{20}						
	AVG						

(1) CORRECTION FACTOR FOR VISCOSITY OF WATER AT 20 C OBTAINED FROM TABLE VII-1.

(2) $k_{20} = 2.303 \frac{a}{A} \frac{L}{t} \text{LOG} \frac{h_o}{h_f} \times R_T = \frac{CL}{t} \left(\text{LOG} \frac{h_o}{h_f}\right) R_T$

REMARKS ______

TECHNICIAN ______ COMPUTED BY ______ CHECKED BY ______

Figure 11-22 Falling Head Permeability Test with Consolidometer Data Sheet

FALLING-HEAD PERMEABILITY TEST
WITH CONSOLIDOMETER

DATE ____________

PROJECT ____________

BORING NO. ____________

SAMPLE OR SPECIMEN NO.

WT IN GRAMS	TARE PLUS DRY SOIL			DIAMETER OF SPECIMEN, CM	D	
	TARE			AREA OF SPECIMEN, SQ CM	A	
	DRY SOIL	W_s		INITIAL HEIGHT OF SPECIMEN, CM	L	
SPECIFIC GRAVITY		G_s		INITIAL VOL OF SPEC, CC = AL	V	
VOL OF SOLIDS, CC = $W_s \div G_s$		V_s		INITIAL VOID RATIO = $(V - V_s) \div V_s$	e	
AREA OF STANDPIPE, SQ CM		a		CONSTANT = $(2.303 \times a) \div A$	C	
CAPILLARY RISE, CM		h_c		INITIAL DIAL READING, IN.	D_0	
HEIGHT OF TAILWATER, CM		h_t		CORRECTED TAILWATER, CM, $h_t + h_c$	Δh	

TEST NO.		1		2		3	
LOAD INCREMENT, T/SQ FT	P						
DIAL READING AT START, IN.	D_1						
CHANGE IN HT OF SPEC, IN. = $D_0 - D_1$	ΔD						
HT OF SPEC, CM = L − 2.54 ΔD	L						
VOID RATIO = $(AL - V_s) \div V_s$	e						
		1a	1b	2a	2b	3a	3b
INITIAL TIME	t_o						
FINAL TIME	t_f						
ELAPSED TIME, SEC = $t_f - t_o$	t						
INITIAL HEIGHT, CM	h_1						
FINAL HEIGHT, CM	h_2						
WATER TEMPERATURE, °C	T						
VISCOSITY CORRECTION FACTOR(1)	R_T						
COEFFICIENT OF PERMEABILITY,(2) CM/SEC	k_{20}						
	AVG						

(1) CORRECTION FACTOR FOR VISCOSITY OF WATER AT 20 C OBTAINED FROM TABLE VII-1.

(2) $k_{20} = 2.303 \frac{a}{A} \frac{L}{t} \left(\text{LOG} \frac{h_1 - \Delta h}{h_2 - \Delta h} \right) R_T = \frac{CL}{t} \left(\text{LOG} \frac{h_1 - \Delta h}{h_2 - \Delta h} \right) R_T$

REMARKS ____________

TECHNICIAN ____________ COMPUTED BY ____________ CHECKED BY ____________

Figure 11-23 Consolidation Test Specimen Data Sheet

CONSOLIDATION TEST

(Specimen Data)

Date ____________

Project ____________

Boring No. ____________ Sample No. ____________

Classification

			Before Test		After Test
			Specimen	Trimmings	Specimen
Tare No.			Ring and plates		
Weight in grams	Tare plus wet soil				
	Tare plus dry soil				
	Water	W_w	W_{wo}		W_{wf}
	Tare				
	Dry soil	W_s			
	Water content	w	w_o %	%	w_f %

Consolidometer No.		Area of specimen, A, sq cm	
Weight of ring, g		Height of specimen, H, in.	
Weight of plates, g		Sp gr of solids, G_s	

Height of solids, $H_s = \frac{W_s}{A \times G_s \times \gamma_w} = \frac{\qquad}{\quad \times \quad \times 1 \times 2.54} =$ in.

Original height of water, $H_{wo} = \frac{W_{wo}}{A \times \gamma_w} = \frac{\qquad}{\quad \times 1 \times 2.54} =$ in.

Final height of water, $H_{wf} = \frac{W_{wf}}{A \times \gamma_w} = \frac{\qquad}{\quad \times 1 \times 2.54} =$ in.

Net change in height of specimen at end of test, $\Delta H =$ in.

Height of specimen at end of test, $H_f = H - \Delta H =$ in.

Void ratio before test, $e_o = \frac{H - H_s}{H_s} = \underline{\qquad\qquad} =$

Void ratio after test, $e_f = \frac{H_f - H_s}{H_s} = \underline{\qquad\qquad} =$

Degree of saturation before test, $S_o = \frac{H_{wo}}{H - H_s} = \underline{\qquad\qquad} =$ %

Degree of saturation after test, $S_f = \frac{H_{wf}}{H_f - H_s} = \underline{\qquad\qquad} =$ %

Dry density before test, $\gamma_d = \frac{W_s}{H \times A} = \frac{\quad \times 62.4}{\quad \times \quad \times 2.54} =$ lb/cu ft

Remarks ____________

____

Technician ____________ Computed by ____________ Checked by ____________

Figure 11-24 Consolidation Test Time-Consolidation Data Sheet

CONSOLIDATION TEST

(Time-Consolidation Data)

Date ____________________

Project ____

Boring No. ____________________ Sample No. ______________ Consol. No. ______________

Date and Pressure	Time	Elapsed Time, min	Dial Rdg. 10^{-4} in.	Temp. $^{\circ}C$	Date and Pressure	Time	Elapsed Time, min	Dial Rdg. 10^{-4} in.	Temp. $^{\circ}C$

Technician ____________________

Figure 11-25 Consolidation Test Time Curves

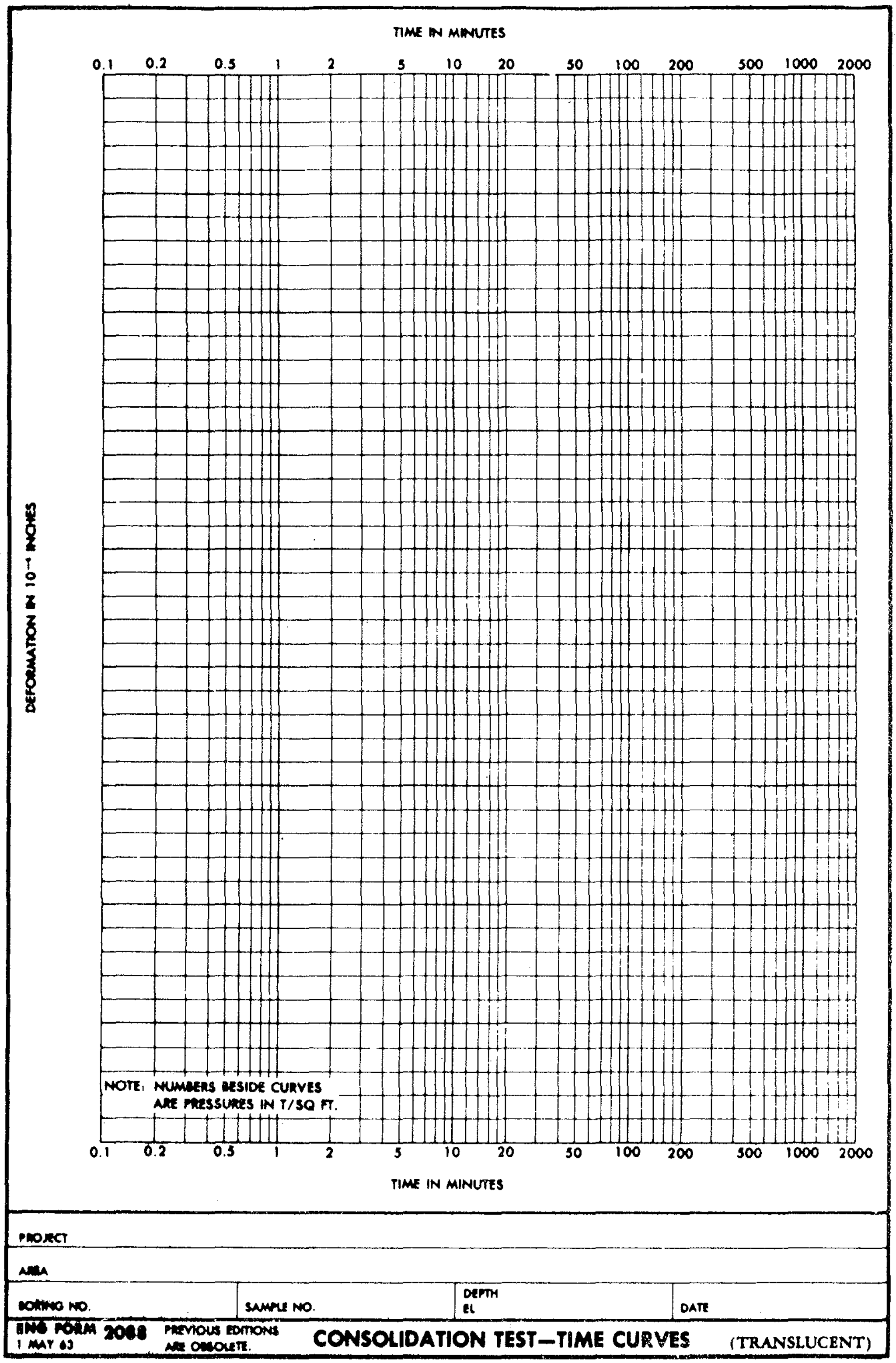


Figure 11-26 Consolidation Test Computation of Void Ratios

CONSOLIDATION TEST

(Computation of Void Ratios)

DATE ____________

PROJECT ____________

BORING NO. ____________ SAMPLE NO. ____________ CONSOLIDOMETER NO. ____________

Pressure, p T/sq ft	Date Increment Applied	Time in Min Increment Effective	Dial Reading 10^{-4} in.	Correction 10^{-4} in.	Change in Height, ΔH 10^{-4} in.	Height of Voids, H_v 10^{-4} in.	Void Ratio, e

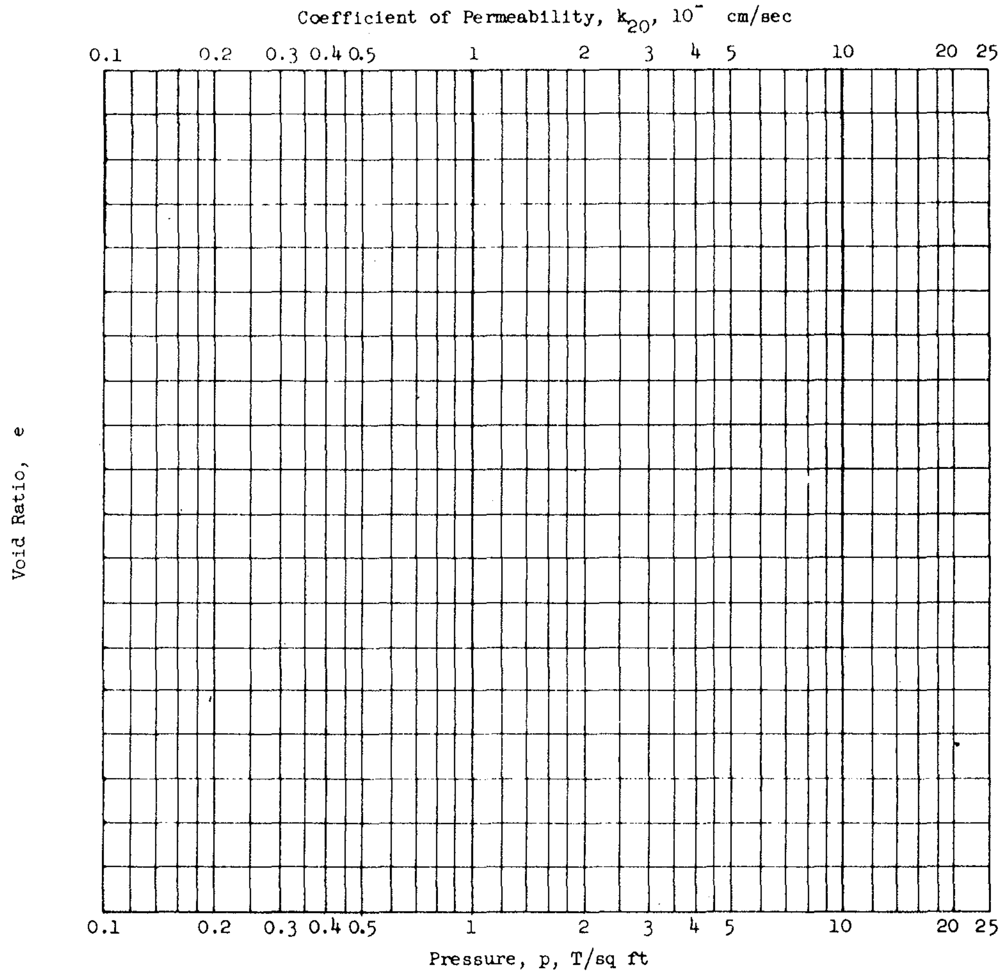
Note:

Height of voids, $H_v = (H - H_s) - \Delta H =$

Void ratio, $e = \frac{H_v}{H_s}$

TECHNICIAN ____________ COMPUTED BY ____________ CHECKED BY ____________

Figure 11-27 Consolidation Test Report



Type of Specimen		Before Test		After Test	
Diam in.	Ht in.	Water Content, w_o	%	w_f	%
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o		e_f	
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	%	S_f	%
Compression Index, C_c		Dry Density, γ_d	lb/ft^3		
Classification		k_{20} at e_o = $\times 10^{-}$ cm/sec			
LL	G_s	Project			
PL	D_{10}				
Remarks		Area			
		Boring No.		Sample No.	
		Depth El		Date	
		CONSOLIDATION TEST REPORT			

Figure 11-28 Compaction Test Data Sheet and Report

COMPACTION TEST

Date ____________

Project ____________

Boring No. ____________ Sample No. ____________

Mold No.	inch diam mold	Volume of mold, V, in cc =
Mold constant, C = 62.4 ÷ V =		Initial water content, w_o =
blows per each of	layers, with lb rammer	inch drop

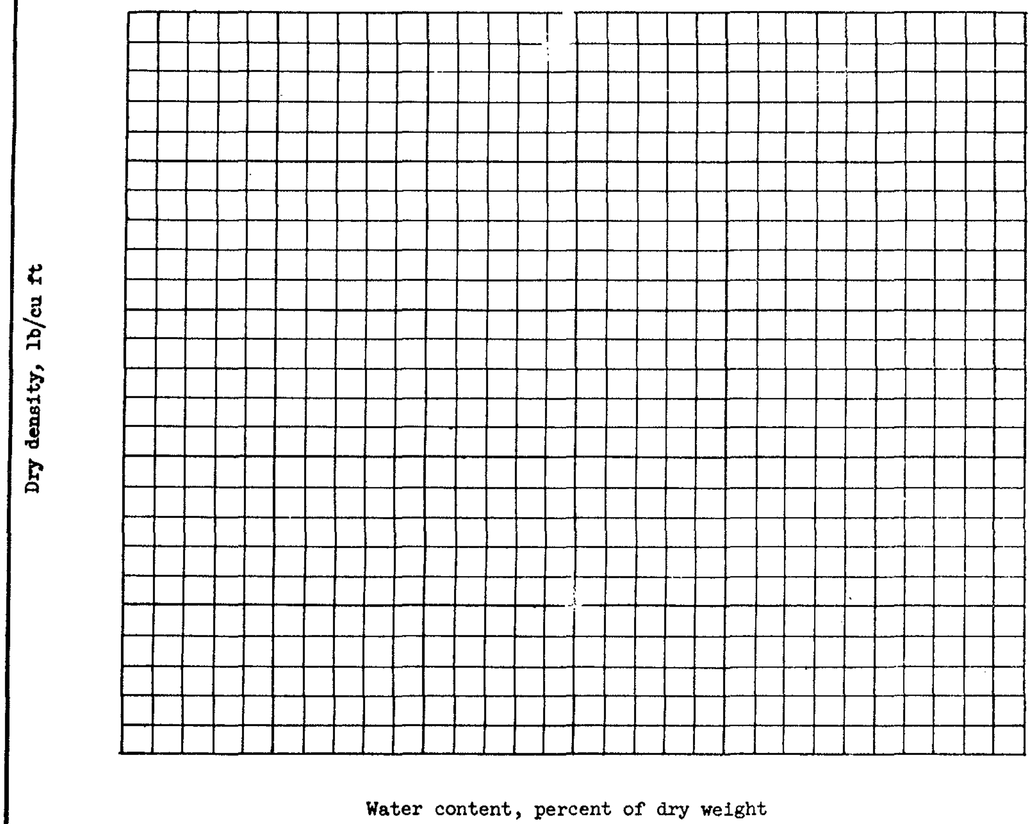
	Specimen No.							
	Preparation of specimen							
	$100 + w_o$							
Weight in grams	Oven-dry soil	W_s'						
Weight in grams	Wet soil = $\frac{W_s'(100 + w_o)}{100}$	W_o'						
Weight in grams	Tare							
Weight in grams	Tare plus wet soil							
	Test water content	w'	%	%	%	%	%	%
	Water added = $\frac{W_s'(w' - w_o)}{100}$ in cc							
	Compacted specimen							
Weight in grams	Mold plus wet soil							
Weight in grams	Mold tare							
Weight in grams	Wet soil	W						
Weight in grams	Dry soil = $\frac{100\ W}{100 + w}$	W_s						
	Water content = $\frac{W - W_s}{W_s} \times 100$	w	%	%	%	%	%	%
lb per cu ft	Wet unit wt = CW	γ_m						
lb per cu ft	Dry unit wt = CW_s	γ_d						
	Water content determinations							
	Specimen No.							
	Tare No.							
Weight in grams	Tare plus wet soil							
Weight in grams	Tare plus dry soil							
Weight in grams	Water	W_w						
Weight in grams	Tare							
Weight in grams	Dry soil	W_s						
	Water content = $\frac{W_w}{W_s} \times 100$	w	%	%	%	%	%	%

Remarks ____________

____

Technician ____________ Computed by ____________ Checked by ____________

Figure 11-28 Compaction Test Data Sheet and Report



__________ compaction test

__________ blows per each of __________ layers, with __________ lb rammer and

__________ inch drop. __________ inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.

Sample No.				
Natural water content, percent				
Optimum water content, percent				
Max dry density, lb/cu ft				

Remarks	Project	
	Area	
	Boring No.	Date
	COMPACTION TEST REPORT	

Figure 11-29 Compaction Test Work Sheet

PROJECT ____________
LAB. NO. ____________
DATE ____________
TESTED BY ____________

COMPACTION TEST
WORK SHEET

TRIAL NO.	WET WT SPECIMEN & MOLD LB	MOLD WEIGHT LB	SPECIMEN WET WT LB	MOISTURE CONTENT		SPECIMEN DRY WT LB	VOLUME OF MOLD (SPECIMEN) FT³	DRY DENSITY LB/CU FT
				WT OF WATER LB	PERCENT MOISTURE			
1	156.3	64.2	92.1	3.7	4.2	88.4	0.783	112.9
2	163.6	64.2	99.4	5.5	5.9	93.9	0.783	119.9
3	171.7	64.2	107.5	8.1	8.1	99.4	0.783	126.9
4	171.8	64.2	107.6	10.0	10.2	97.6	0.783	124.6
5	168.8	64.2	104.6	11.0	11.8	93.6	0.783	119.5

GRADATION

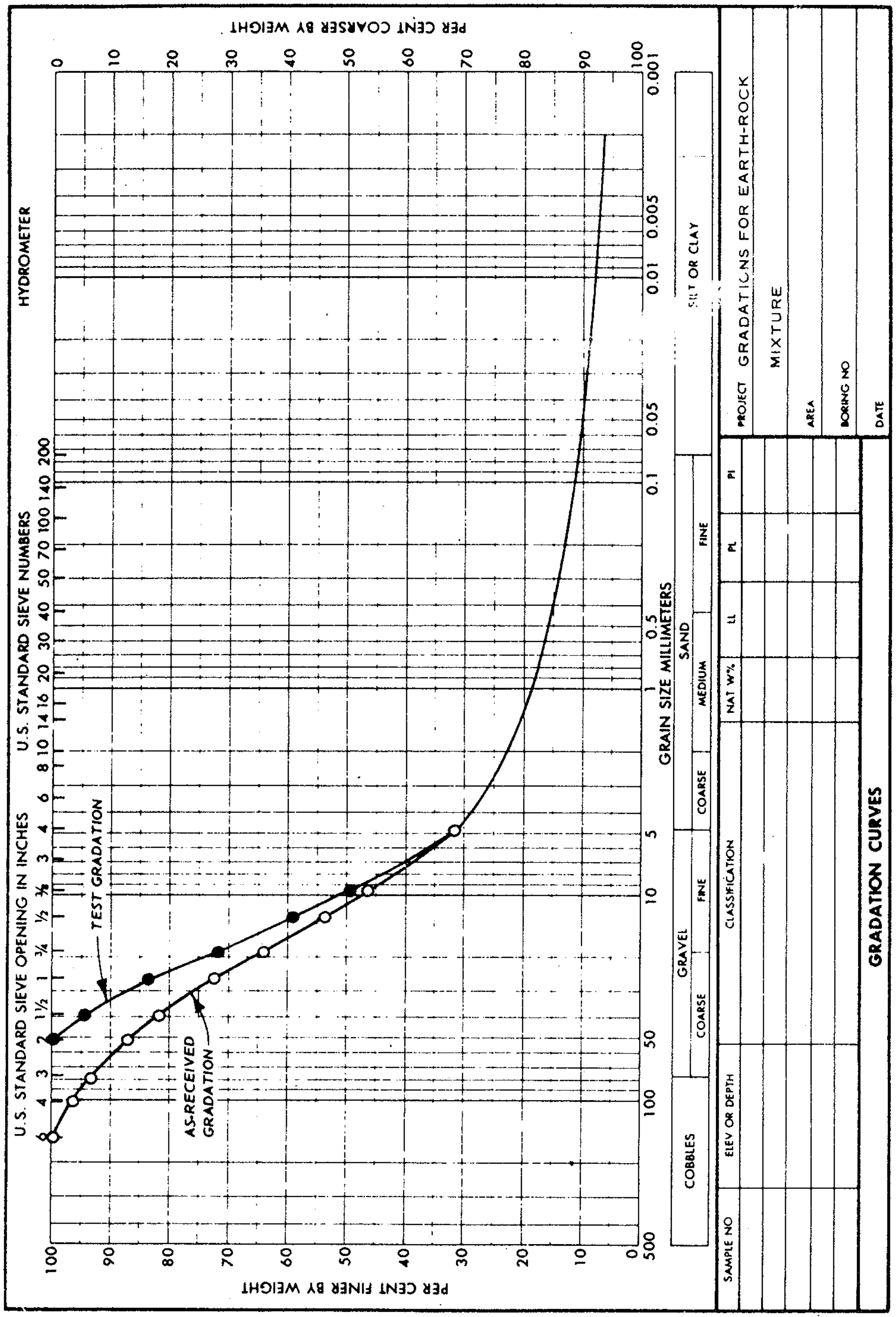
SIEVE SIZE	ORIGINAL SAMPLE		TEST SPECIMEN			WT FOR COMPACTION SPECIMEN
	WT RETAINED	% RETAINED	% ORIGINAL	% REPLACEM'T	% TOTAL	
2"	104.0	13.0	0.0	0.0	0.0	0.0
1-1/2"	40.0	5.0	5.0	1.2	6.2	8.1
1"	72.0	9.0	9.0	2.1	11.1	14.4
3/4"	72.0	9.0	9.0	2.1	11.1	14.4
1/2"	80.0	10.0	10.0	2.2	12.2	15.9
3/8"	64.0	8.0	8.0	1.9	9.9	12.9
#4	120.0	15.0	15.0	3.5	18.5	24.0
PAN	248.0	31.0	31.0	0.0	31.0	40.3
TOTAL	800.0	100.0	87.0	13.0	100.0	130.0

REMARKS 140 BLOWS/EACH OF 3 LAYERS WITH 11.5-LB RAMMER AND 24-IN. DROP. DIAMETER OF MOLD = 12 IN.

STANDARD ✓

MODIFIED ____

Figure 11-30 Gradation Curves Sample Sheet



11.3. Bibliography

This bibliography is by no means complete. In addition to these works, many works are referenced in the footnotes in the text, and the reader is encouraged to refer to these. Works in **bold** are those that were directly used in the compilation of this book.

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