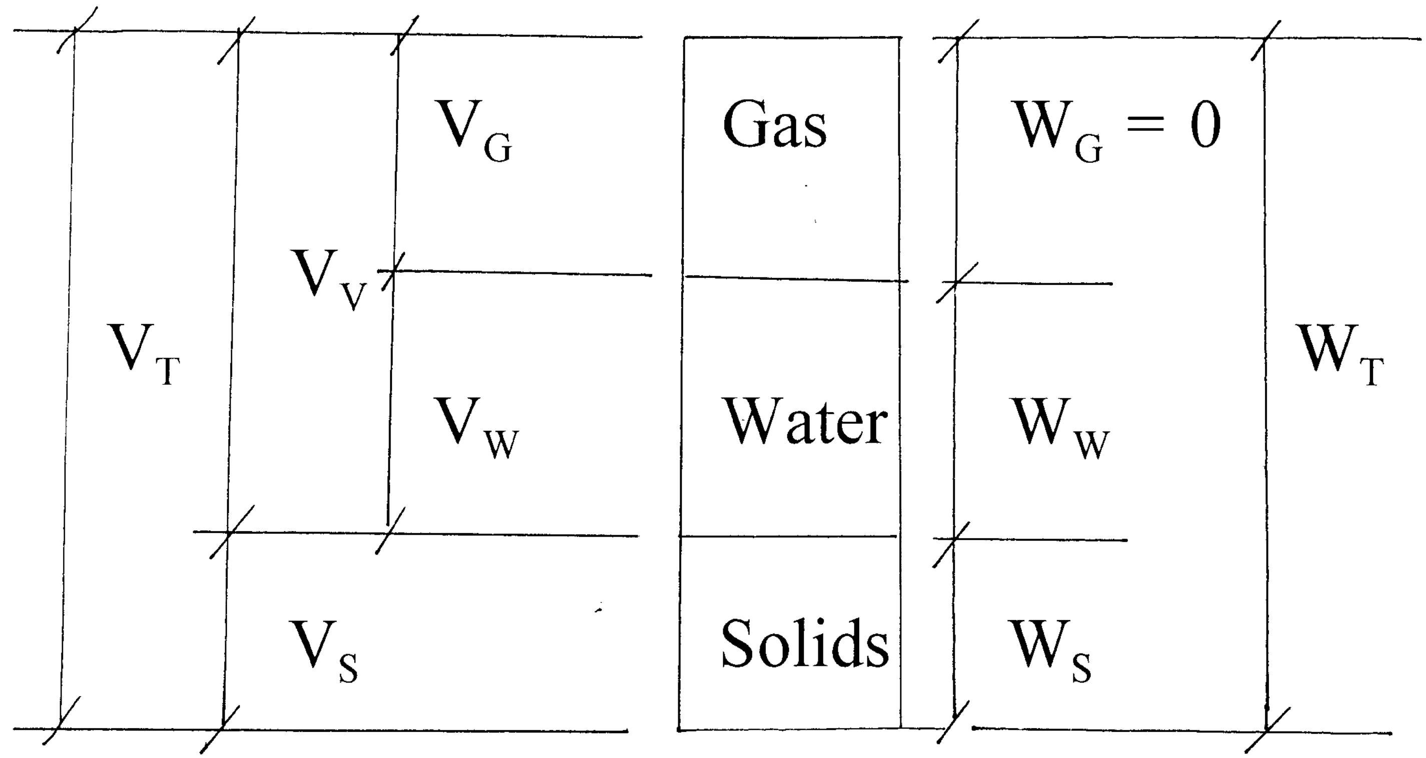
PHASE DIAGRAMS



G_S

Phase # 1

WEIGHT/WEIGHT

Water Content = w, %

$$w, \% = \frac{W_W * 100}{W_S}$$

Phase # 2

VOLUME/VOLUME

Void Ratio = e = V_V/V_S

Porosity, % = n = $V_V * 100/V_T$

Degree of Saturation, % = S = $V_W * 100/V_V$

Phase # 3

WEIGHT/VOLUME

Wet Unit Weight = $\gamma_{wet} = W_T/V_T$

Dry Unit Weight = $\gamma_{dry} = W_T/V_T$

Saturated Unit Weight = $\gamma_{sat} = W_T/V_T$
where $W_T = V_V * \gamma_{water}$

Phase # 4

What is normally determined in the field?

W_T and V_T
- By Core or Harris Cup
- By Scoop and Balloon
- By Scoop and Sand

w,%
- By Convection Oven
- By Microwave Oven

Then calculate W_S. Everything else can be calculated if G_S is known or assumed.

Alternate: Nuclear Density Meter

Phase # 5

Example # 1

A soil sample is collected from the field with a volume of 120 cubic centimeters and a total weight of 240 grams. A small portion of the total sample is weighed wet and then dried overnight in an oven. The wet weight of the small sample and the dish is 30 grams and the dry weight of the small sample and the dish is 20 grams. The dish weighs 2 grams.

a. What is the wet unit weight?

$W_T = 240\ g$

$V_T = 120\ cm^3$

$$\gamma_{wet} = 2\ \frac{mg}{cm^3}$$

$$(2)(62.4) = 124.8\ \frac{lbs}{ft^3}$$

Phase # 6a

b. What is the water content?

$$W_w = 30 - 20 = 10 \text{ g}$$

$$W_s = 20 - 2 = 18 \text{ g}$$

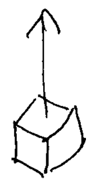
$$w\% = \frac{(10)100}{(18)} = 55.56$$

c. What is the dry unit weight?

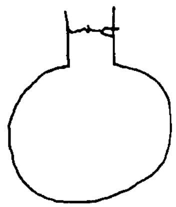
$$\gamma_{dry} = \frac{\gamma_{wet}}{1 + \frac{w\%}{100}} = \frac{124.8}{1.5556} = 80.2 \frac{\text{lbs}}{\text{ft}^3}$$

Phase # 6b

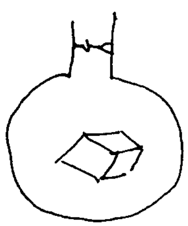
SPECIFIC GRAVITY



WEIGHT OF SOLID BLOCK (ON EARTH) WHEN WEIGHED IN AIR = 120 POUNDS.



WEIGHT OF WATER (ON EARTH) THAT FILLS CONTAINER = 624 POUNDS.
VOLUME MUST BE 10.0 CU FT.



PLACE BLOCK AND IN CONTAINER AND FILL TO MARK WITH WATER. WEIGHT THE WHOLE THING. TOTAL WEIGHT = 681.6 POUNDS.

WHAT IS THE SPECIFIC GRAVITY OF THE BLOCK?

Phase # 7a

$W_T = 681.6 \text{ lbs}$ (BLOCK & WATER)

$W_B = 120.0 \text{ lbs}$ (Block ONLY)

$W_W = 561.6 \text{ lbs}$ (WATER IN CONTAINER)

$V_W = \dfrac{(561.6) \text{ lb ft}^3}{(62.4) \text{ lbs}} = 9.0 \text{ ft}^3$ of Water in Container

$V_B = 1.0 \text{ ft}^3$ Volume of Block

$G_S = \dfrac{(120.0) \cancel{\text{lbs}} \cancel{\text{ft}^3}}{(1) \cancel{\text{ft}^3} (62.4) \cancel{\text{lbs}}} = \underline{\underline{1.933}}$

Phase # 7b

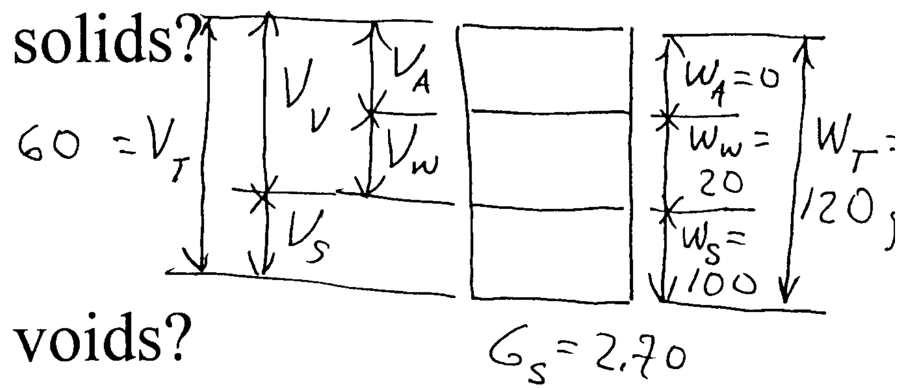
Example # 2

If the specific gravity of the soil in Example # 1 is 2.70 compute the following:

a. What is the volume of the solids?

$$\frac{(100)\,g\ cm^3}{(1)\,g\,(2.7)} = 37.04$$

(1)



b. What is the volume of the voids?

$$V_V = (60)\,cm^3 - 37.04\ cm^3$$

$$V_V = 22.96$$

(2)

c. What is the volume of the water?

$$V_W = \frac{(20)\,g\ cm^3}{(1)\,g} = 20\ cm^3$$

Phase # 8a

d. What is the volume of the gas?

$$V_g = V_U - V_W = 37.04 - 20$$
$$= 17.04\ cm^3$$

Phase # 8b

Example # 3 Continued

e. What is the void ratio? $e = \frac{V_v}{V_s} = 0.62$

f. What is the porosity? $n = \frac{V_v}{V_T} 100 =$

g. What is the degree of saturation? $S\% = \frac{V_w 100}{V_v}$

$S\% = \frac{(20)(100)}{(\cancel{27.04})(22.96)}$

$S\% = 87.1\%$

h. What is the saturated unit weight?

$W_s = 100$

$W_w = 22.96$ if Sat

$W_T = 122.96$

$\gamma_{SAT} = \frac{122.96}{60} = 2.05\ g/cm^3$

$127.88\ lbs/ft^3$

Phase # 8c

Example # 4

Soil is excavated from a borrow pit and compacted at another location. The volume of the soil in the borrow pit is found to be 10,000 cubic yards and the average wet unit weight of the soil in the borrow pit is 130 pounds per cubic foot at a water content of 20%.

The soil is compacted into a fill with a dry unit weight of 125 pounds per cubic foot and a water content of 30%.

Compute the following:

a. What is the volume of the soil in the fill?

Phase # 9a

b. How much water must be added if none is lost by evaporation?

Phase # 9b

Original Volume = 10,000 cy

W_T = (130)pcf(27)cf/cy(10,000)cy = 35,100,000 lbs

w(%) = 20% W_s = 35,100,000/1.20 = 29,250,000 lbs

W_w = 5,850,000 lbs

Final Volume = ?
W_s is still 35,100,000 lbs

W_w = 0.30*W_s = 8,775,000 lbs

W_T = 380,025,000 lbs

Phase 9c

W_s/V_T = (125)pcf(27)cf/cy

V_T = 8,666.67 cy

Final Water = 8,775,000 lbs
Orig Water = 5,850,000 lbs
Difference = 2,925,000 lbs

46,875 cf of water
350,719 gallons of water

Phase 9d

OPENING SIZES OF STANDARD SIEVES

SIEVE SIZE	OPENING
1.5 in	37.5 mm
1	25
0.75	12.5
No. 4	4.75
10	2.00
20	0.850
40	0.425
70	0.212
100	0.150
200	0.075

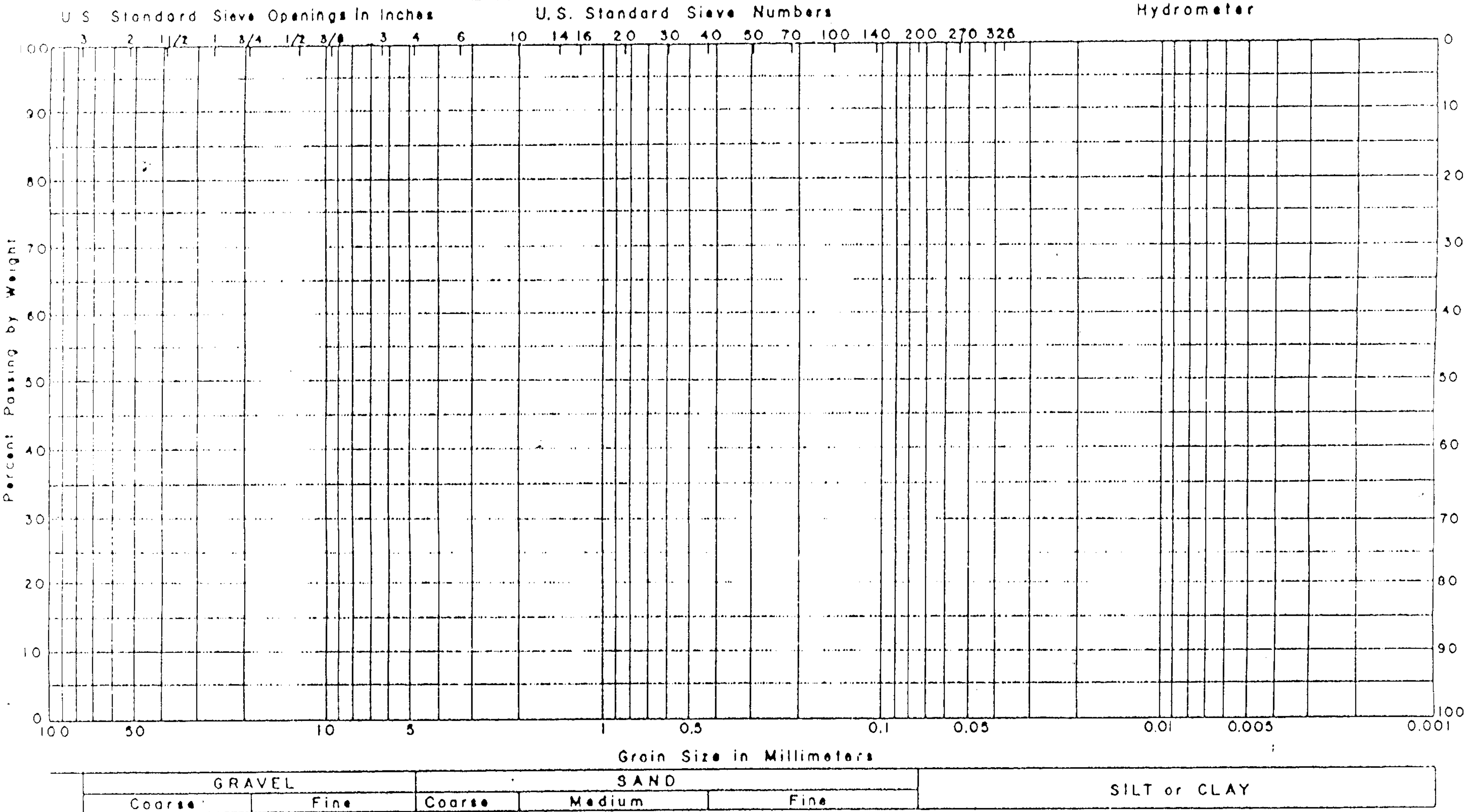
Sieve # 1

SIEVE ANALYSIS

	RETAINED Grams	CUMULATIVE RETAINED	CUMULATIVE PASSED
#4	20	20%	80%
#100	25	45%	55%
#200	30	75%	25%
Pan	25	100%	0%
Total	100 grams		

Sieve # 2

MECHANICAL ANALYSIS CHART



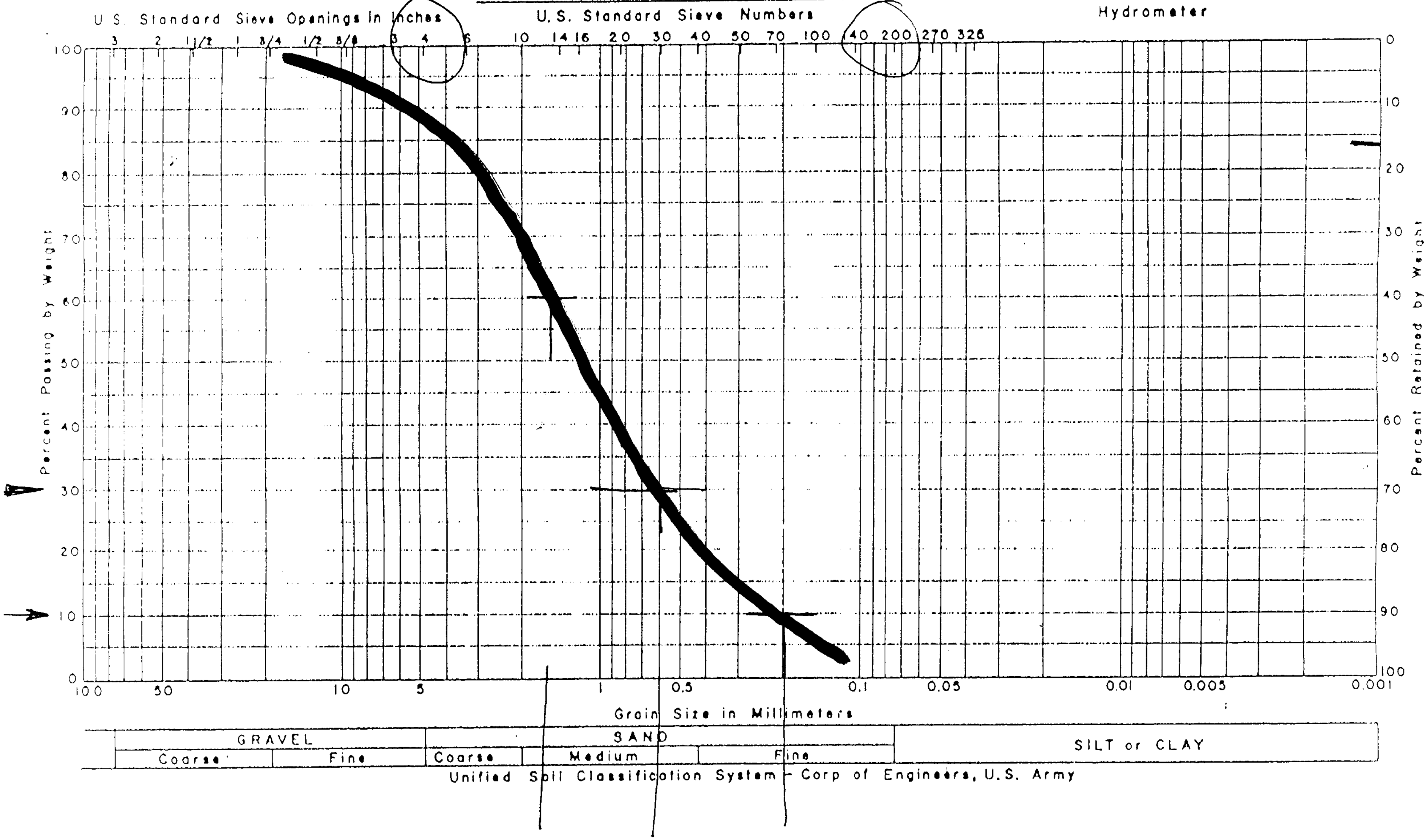
Unified Soil Classification System – Corp of Engineers, U.S. Army

3

Sieve #3

MECHANICAL ANALYSIS CHART

#4 #200



D_{60} 1.8 D_{30} 0.6 D_{10} 0.2

F Sieve #4

$D_{10} = 0.2$

$D_{30} = 0.6$

$D_{60} = 1.8$

UNIFORMITY COEFFICIENT

$C_u = D_{60}/D_{10} = 1.8/0.2 = 9$

COEFFICIENT OF CURVATURE

$C_z = (D_{30})^2/(D_{10} * D_{60}) = 0.6^2/(0.2*1.8) = 1$

Sieve # 5

CLASSIFICATION OF SANDS & GRAVELS

For Sands

A sand is WELL GRADED if C_u is more than 6 and if C_z is between 1 and 3.
Otherwise it is POORLY GRADED.

For Gravels

A gravel is WELL GRADED if C_u is more than 4 and if C_z is between 1 and 3.
Otherwise it is POORLY GRADED.

Sieve # 6

SIEVE ANALYSIS & SOIL CLASSIFICATION

% RETAINED ON # 4 SIEVE IS GRAVEL = 17
% PASSING # 4 SIEVE IS SAND + FINES = 83

% RETAINED ON # 200 IS GRAVEL + SAND (OR COARSE) = 100
% PASSING # 200 IS FINE = 0

Sieve # 7

UNIFIED SOIL CLASSIFICATION
BY SIEVE ANALYSIS

Gravel - - - - 17%
Sand - - - - - - 83%
Coarse - - - -100%
Fine - - - - - - 0%
Total - - - -100%

Soil is a sand.

It is a clean sand.

Sieve # 8

ATTERBERG LIMITS

Liquid

LIQUID LIMIT (LL)

Plastic

PLASTIC LIMIT (PL)

Semi-Solid

SHRINKAGE LIMIT (SL)

Solid

Atterberg # 1

PLASTICITY CHART

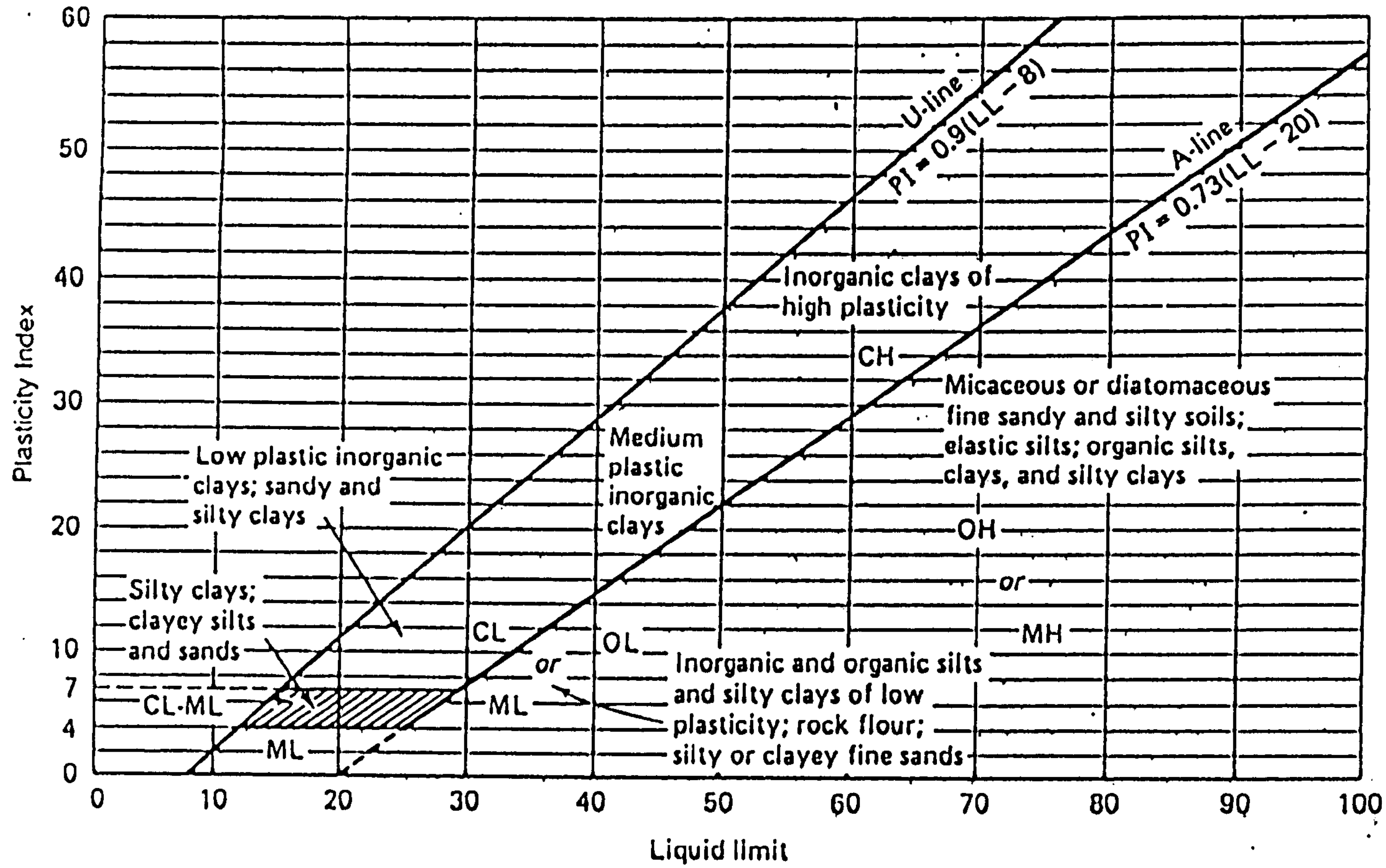


Fig. 3.2 Casagrande's plasticity chart, showing several representative soil types (developed from Casagrande, 1948, and Howard, 1977).

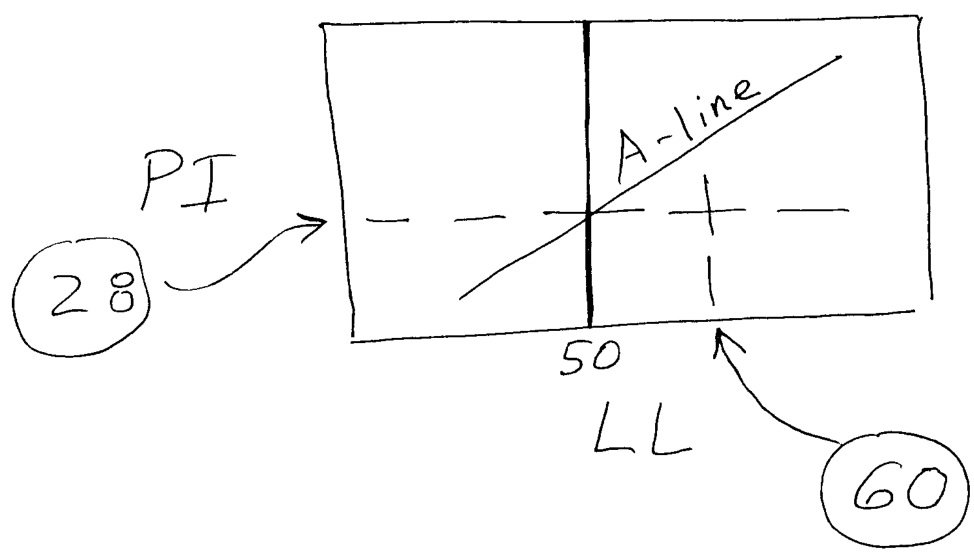
Atterberg # 2

29

A soil has a LL = 60
and a PL = 32

PI = 28

Classify the soil with the Plasticity chart.



Does it plot above or below A-line?

Atterberg # 4a

Equation for A-line is:

$$PI = 0.73(LL - 20)$$

If LL is 60 then the PI on the A-line is

$$= 0.73(60 - 20) = 0.73(40) = 29.2$$

The actual PI is 28.

Then the soil plots below the A-line and the soil is MH.
Why MH and not ML? Because LL is greater than 50.

Why MH and not OH? Because OH is determined by smell, taste and texture.
Atterberg 4b

UNIFIED SOIL CLASSIFICATION BY SIEVE ANALYSIS

Coarse grained soils are clean if they contain less than 5% fines. Clean coarse grained soils require Well Graded (W) or Poorly Graded (P).

Coarse grained soils are dirty if they contain more than 12% fines. Dirty coarse grained soils requird Clay (C) or Silt (M).

Coarse grained soils are "dual" if they contain between 5% and 12% fines. Dual coarse grained soils require both W or P and C or M.

Sieve # 9

UNIFIED SOIL CLASSIFICATION BY SIEVE ANALYSIS

3 coarse grained soils have the following characteristics:

	A	B	C
Cum % Retained on # 4 Sieve =	30	5	12
Cum % Passing # 200 Sieve =	42	8	3
Cu	4.2	5.5	6.3
Cz	1.5	2.1	3.1

Sieve # 10a

Gravel	30	5	12
Sand	28	87	85
Coarse	58	92	97
Fine	42	8	3
Total	100	100	100
	Soil A	Soil B	Soil C
	Gravel	Sand	Sand
	Dirty	Dual	Clean

Sieve # 10b

SHRINKAGE LIMIT

The shrinkage limit is usually determined by plotting the LL and the PI for the soil on the Plasticity Chart. The point will plot above or below the A-line. The distance fromthe A-line to the point is called Δp_i. If the point plots above the A-line the Shrinkage Limit is equal to 20 - Δp_i. If the point plots below the A-line the Shrinkage Limit is equal to 20 + Δp_i.

Atterberg # 5

UNIFIED SOIL CLASSIFICATION SYSTEM

You need the Sieve Analysis
for D10, D30 & D60 sizes to calculate Cu and Cz
for determination of Cumulative % Retained on # 4 (gravel) and Cumulative % passing #200 (fine)

Atterberg Limits

Plasticity Chart

USCS # 1

UNIFIED SOIL CLASSIFICATION SYSTEM

	A	B	C
% Cum Ret on # 4	30	49	10
% Cum Passing # 200	50	3	15
LL	70	30	40
PL	30	22	28
PI	40	8	12
Cu	4.5	3.8	6.2
Cz	2.1	1.7	0.8

USCS # 2a

	A	B	C
Gravel	30	49	10
Sand	20	48	75
Coarse	50	97	85
Fine	50	3	15
Total	100	100	100
	Fine	Coarse Gravel Clean	Coarse Sand Dirty
	CH	GW	SP
USCS # 2b		$C_u > 4$ $1 < C_z < 3$	$C_u > 6$ $1 < C_z < 3$ No

TABLE 5-4 *Characteristics Pertinent to Roads and Airfields**

Major Divisions (1)	Major Divisions (2)	Symbol: Letter (3)	Symbol: Hatching (4)	Symbol: Color (5)	Name (6)	Value as Subgrade When not Subject to Frost Action (7)	Value as Subbase When not Subject to Frost Action (8)	Value as Base When not Subject to Frost Action (9)	Potential Frost Action (10)	Compressibility and Expansion (11)
COARSE-GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Excellent	Good	None to very slight	Almost none
		GP		Red	Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Good	Fair to good	None to very slight	Almost none
		GM d		Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Good	Fair to good	Slight to medium	Very slight
		GM u		Yellow		Good	Fair	Poor to not suitable	Slight to medium	Slight
		GC			Clayey gravels, gravel-sand-clay mixtures	Good	Fair	Poor to not suitable	Slight to medium	Slight
	SAND AND SANDY SOILS	SW		Red	Well-graded sands or gravelly sands, little or no fines	Good	Fair to good	Poor	None to very slight	Almost none
		SP		Red	Poorly graded sands or gravelly sands, little or no fines	Fair to good	Fair	Poor to not suitable	None to very slight	Almost none
		SM d		Yellow	Silty sands, sand-silt mixtures	Fair to good	Fair to good	Poor	Slight to high	Very slight
		SM u		Yellow		Fair	Poor to fair	Not suitable	Slight to high	Slight to medium
		SC			Clayey sands, sand-clay mixtures	Poor to fair	Poor	Not suitable	Slight to high	Slight to medium
FINE-GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor to fair	Not suitable	Not suitable	Medium to very high	Slight to medium
		CL		Green	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable	Not suitable	Medium to high	Medium
		OL			Organic silts and organic silt-clays of low plasticity	Poor	Not suitable	Not suitable	Medium to high	Medium to high
	SILTS AND CLAYS LL IS GREATER THAN 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Not suitable	Medium to very high	High
		CH		Blue	Inorganic clays of high plasticity, fat clays	Poor to fair	Not suitable	Not suitable	Medium	High
		OH			Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable	Not suitable	Medium	High
HIGHLY ORGANIC SOILS		Pt		Orange	Peat and other highly organic soils	Not suitable	Not suitable	Not suitable	Slight	Very high

*After U.S.Army Waterways Experiment Station (1960).

Unified Soil Classification # 3

TABLE 5-4 Continued

Major Divisions (1)	(2)	Symbol: Letter (3)	Symbol: Hatching (4)	Symbol: Color (5)	Drainage Characteristics (12)	Compaction Equipment (13)	Unit Dry Densities: lbf/ft^3 (14)	Unit Dry Densities: Mg/m^3 (15)	Typical Design Values: CBR (16)	Typical Design Values: Subgrade Modulus k (lbf/in.^{3}) (17)
COARSE-GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		Red	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125-140	2.00-2.24	40-80	300-500
		GP		Red	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110-140	1.76-2.24	30-60	300-500
		GM d		Yellow	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	125-145	2.00-2.32	40-60	300-500
		GM u		Yellow	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	115-135	1.84-2.16	20-30	200-500
		GC		Yellow	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	130-145	2.08-2.32	20-40	200-500
	SAND AND SANDY SOILS	SW		Red	Excellent	Crawler-type tractor, rubber-tired roller	110-130	1.76-2.08	20-40	200-400
		SP		Red	Excellent	Crawler-type tractor, rubber-tired roller	105-135	1.68-2.16	10-40	150-400
		SM d		Yellow	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120-135	1.92-2.16	15-40	150-400
		SM u		Yellow	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-130	1.60-2.08	10-20	100-300
		SC		Yellow	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-135	1.60-2.16	5-20	100-300
FINE-GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML		Green	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90-130	1.44-2.08	15 or less	100-200
		CL		Green	Practically impervious	Rubber-tired roller, sheepsfoot roller	90-130	1.44-2.08	15 or less	50-150
		OL		Green	Poor	Rubber-tired roller, sheepsfoot roller	90-105	1.44-1.68	5 or less	50-100
	SILTS AND CLAYS LL IS GREATER THAN 50	MH		Blue	Fair to poor	Sheepsfoot roller, rubber-tired roller	80-105	1.28-1.68	10 or less	50-100
		CH		Blue	Practically impervious	Sheepsfoot roller, rubber-tired roller	90-115	1.44-1.84	15 or less	50-150
		OH		Blue	Practically impervious	Sheepsfoot roller, rubber-tired roller	80-110	1.28-1.76	5 or less	25-100
HIGHLY ORGANIC SOILS		Pt		Orange	Fair to poor	Compaction not practical				

Unified Soil Classification # 4

TABLE 5-4 Continued

Notes:

1. In column 3, division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is on basis of Atterberg limits; suffix d (e.g., GMd) will be used when the liquid limit is 25 or less and the plasticity index is 5 or less; the suffix u will be used otherwise.
2. In column 13, the equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled. In some instances, several types of equipment are listed because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.
   a. *Processed base materials and other angular materials*. Steel-wheeled and rubber-tired rollers are recommended for hard, angular materials with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.
   b. *Finishing*. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.
   c. *Equipment size*. The following sizes of equipment are necessary to assure the high densities required for airfield construction:

      Crawler-type tractor—total weight in excess of 30,000 lb (14 000 kg).

      Rubber-tired equipment—wheel load in excess of 15,000 lb (7000 kg), wheel loads as high as 40,000 lb (18 000 kg) may be necessary to obtain the required densities for some materials (based on contact pressure of approximately 65 to 150 psi or 450 kPa to 1000 kPa).

      Sheepsfoot roller—unit pressure (on 6 to 12 $in.^2$ or 40 to 80 cm^2 foot) to be in excess of 250 psi (1750 kPa) and unit pressures as high as 650 psi (4500 kPa) may be necessary to obtain the required densities for some materials. The area of the feet should be at least 5% of the total peripheral area of the drum, using the diameter measured to the faces of the feet.
3. In columns 14 and 15, densities are for compacted soil at optimum water content for modified AASHTO compaction effort.
4. In column 16, the maximum value that can be used in design of airfields is, in some cases, limited by gradation and plasticity requirements.

Unified Soil Classification # 5

TABLE 5-5 Tabulations of Engineering Properties of Compacted Soils Used in Earth Structures*

Typical names of Soil Groups	Group Symbols	Important Engineering Properties: Permeability When Compacted	Shear Strength When Compacted and Saturated	Compressibility When Compacted and Saturated	Workability as a Construction Material	Relative Desirability for Various Uses (No. 1 is considered the best) — Rolled Earthfill Dams: Homogeneous Embankment	Rolled Earthfill Dams: Core	Rolled Earthfill Dams: Shell	Canal Sections: Erosion Resistance	Canal Sections: Compacted Earth Lining	Foundations: Seepage Important	Foundations: Seepage Not Important	Roadways — Fills: Frost Heave Not Possible	Roadways — Fills: Frost Heave Possible	Roadways: Surfacing
Well-graded gravels, gravel-sand mixtures, little or no fines	GW	Pervious	Excellent	Negligible	Excellent	—	—	1	1	—	—	1	1	1	3
Poorly graded gravels, gravel-sand mixtures, little or no fines	GP	Very pervious	Good	Negligible	Good	—	—	2	2	—	—	3	3	3	—
Silty gravels, poorly graded gravel-sand-silt mixtures	GM	Semipervious to impervious	Good	Negligible	Good	2	4	—	4	4	1	4	4	9	5
Clayey gravels, poorly graded gravel-sand-clay mixtures	GC	Impervious	Good to fair	Very low	Good	1	1	—	3	1	2	6	5	5	1
Well-graded sands, gravelly sands, little or no fines	SW	Pervious	Excellent	Negligible	Excellent	—	—	3 if gravelly	6	—	—	2	2	2	4
Poorly graded sands, gravelly sands, little or no fines	SP	Pervious	Good	Very low	Fair	—	—	4 if gravelly	7 if gravelly	—	—	5	6	4	—
Silty sands, poorly graded sand-silt mixtures	SM	Semipervious to impervious	Good	Low	Fair	4	5	—	8 if gravelly	5 erosion critical	3	7	8	10	6

Unified Soil Classification # 6

Clayey sands, poorly graded sand-clay mixtures	SC	Impervious	Good to fair	Low	Good	3	2	—	5	2	4	8	7	6	2
Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	Semipervious to impervious	Fair	Medium	Fair	6	6	—	—	6 erosion critical	6	9	10	11	—
Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	CL	Impervious	Fair	Medium	Good to fair	5	3	—	9	3	5	10	9	7	7
Organic silts and organic silt-clays of low plasticity	OL	Semipervious to impervious	Poor	Medium	Fair	8	8	—	—	7 erosion critical	7	11	11	12	—
Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	Semipervious to impervious	Fair to poor	High	Poor	9	9	—	—	—	8	12	12	13	—
Inorganic clays of high plasticity, fat clays	CH	Impervious	Poor	High	Poor	7	7	—	10	8 volume change critical	9	13	13	8	—
Organic clays of medium to high plasticity	OH	Impervious	Poor	High	Poor	10	10	—	—	—	10	14	14	14	—
Peat and other highly organic soils	Pt	—	—	—	—	—	—	—	—	—	—	—	—	—	—

*After USBR (1974).

Unified Soil Classification # 7

EXAMPLE 5.4

Given:

A soil, classified as a CL according to the USCS, is proposed for a compacted fill.

Required:

Consider the soil to be used as:

a. Subgrade
b. Earth dam
c. Foundation support for a structure

Use Tables 5-4 and 5-5 and comment on:

1. The overall suitability of the soil
2. Potential problems of frost
3. Significant engineering properties
4. Appropriate compaction equipment to use

Solution:

Prepare a table for soil type CL.

Item Use:	Subgrade	Earth Dam	Structural Foundation
1. Applicability	Poor to fair	Useful as central core	Acceptable if compacted dry of optimum and if not saturated during service life
2. Frost potential	Medium to high	Low if covered by nonfrost heaving soil of sufficient depth	Medium to high if not controlled by temperature and water availability
3. Engineering properties	Medium compressibility fair strength CBR ≤ 15	Low permeability, compact for low permeability and high strength but also for flexibility	Potential for poor strength and therefore poor performance
4. Appropriate compaction equipment	Sheepsfoot and/ or rubber-tired roller	Sheepsfoot and/ or rubber-tired roller	Sheepsfoot and/ or rubber-tired roller

Note: Once you have finished with this book and a course in foundation engineering, you could readily expand the information in this table.

COMPACTION

Standard Proctor	Modified Proctor
1/30 cu ft	1/30 cu ft
3 Lifts	5 Lifts
25 Tamps/Lift	25 Tamps/Lift
5.5 lb Hammer	10 lb Hammer
12 in Drop	18 in Drop
ASTM D698	ASTM
AASHO T99	AASHO

Compaction # 1

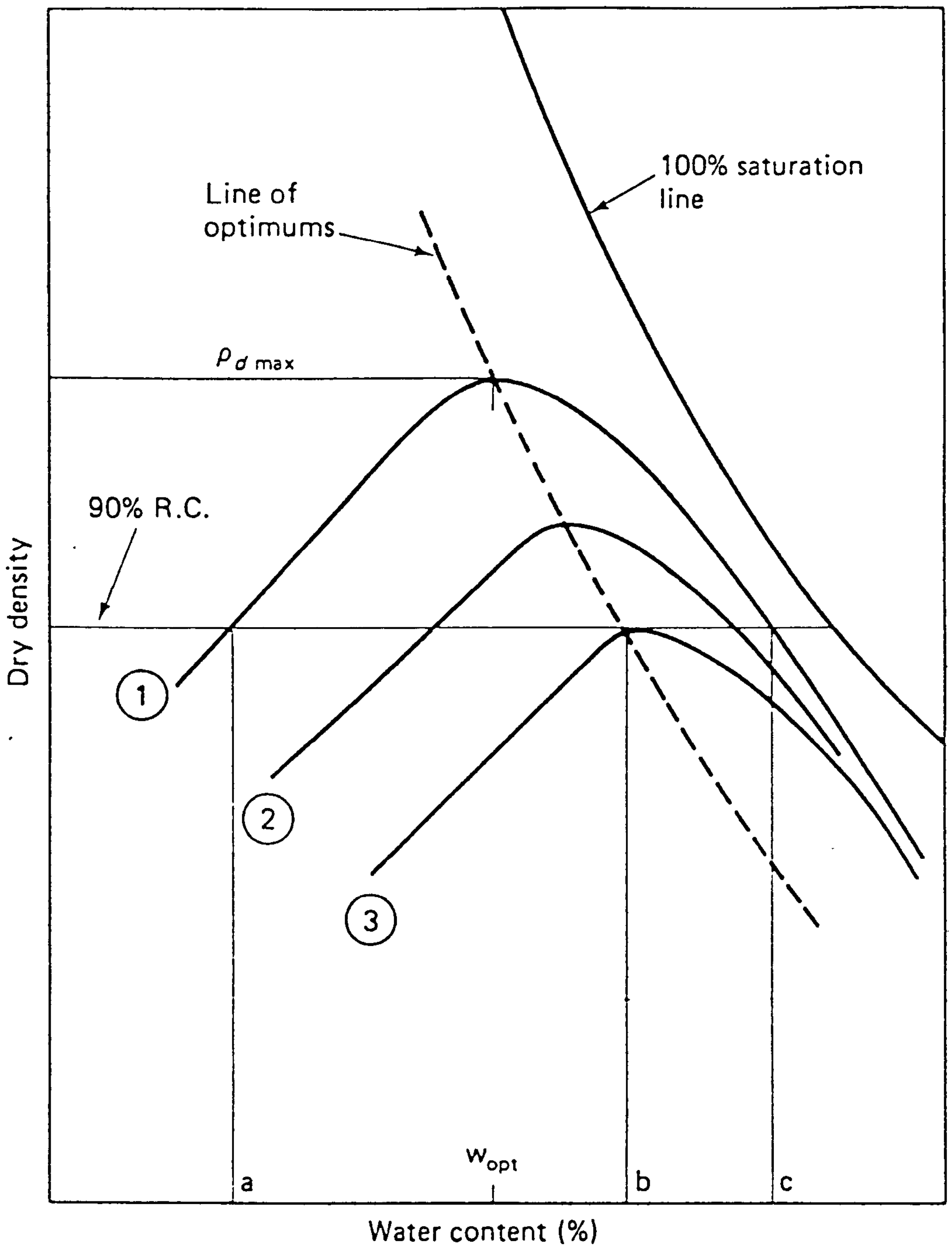


Fig. 5.23 Dry density versus water content, illustrating the most efficient conditions for field compaction (after Seed, 1964).

Compaction # 7

ZERO AIR VOIDS CURVE

$$\gamma_{dry} = \frac{\gamma_{water} S}{w + \dfrac{\gamma_{water}}{G_s * \gamma_{water}} S}$$

S = Degree of Saturation,%
γ = Unit Weight
G_s = Specific Gravity
w = Water Content,%

Compaction # 2

COMPACTION

What is the dry unit weight that corresponds with a water content of 30% in a soil with a specific gravity of 2.7when the soil is saturated?

$$\text{Dry unit weight} = \frac{62.4 * 100}{30 + \left\{\dfrac{62.4}{2.7 * 62.4} * 100\right\}} = 93.08 \text{ pcf}$$

Compaction # 3

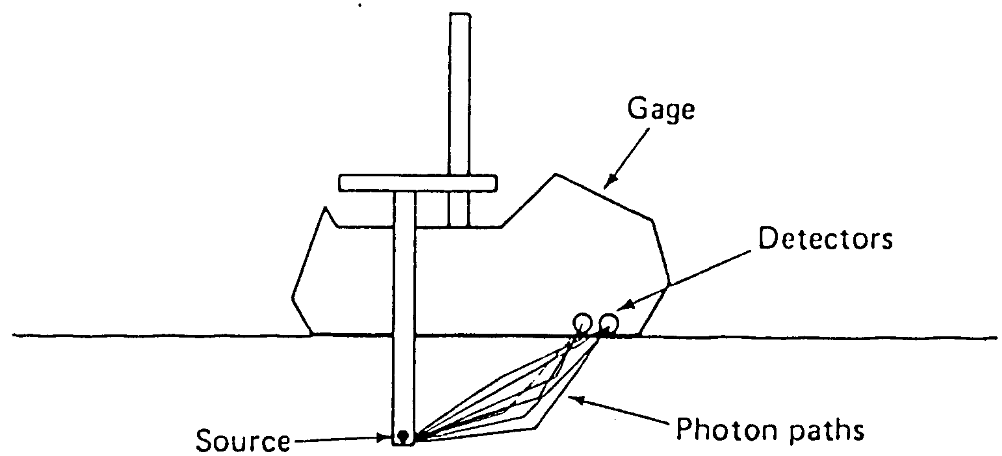
COMPACTION

A soil must be compacted to a density of 95% of Standard Proctor or more and the water content must be wet of the optimum water content (Standard Proctor).

The Standard Proctor optimum water content is 33% and the optimum dry density is 100 pounds per cubic foot.

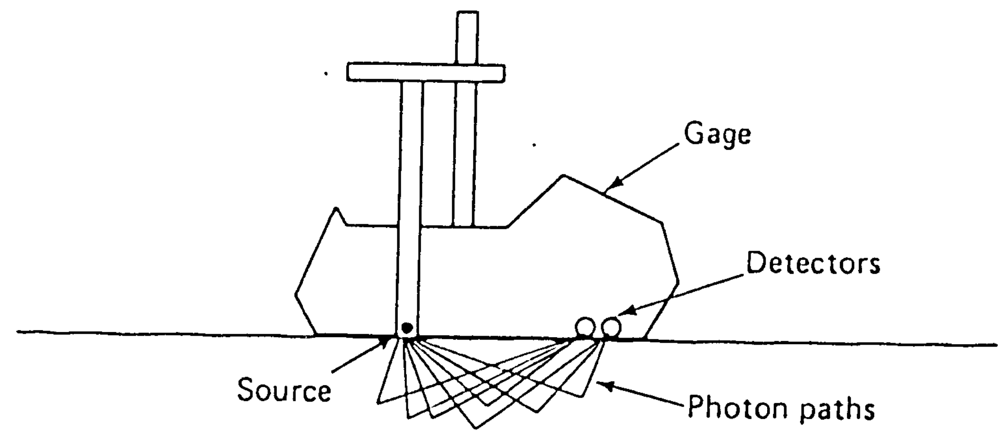
The field density is 93.6 pounds and the water content is 34%.

Water content is ok.
Field Density is too low. 100 pcf * 0.95 = 95 pcf
  93.6 is lower than 95
Compaction # 4



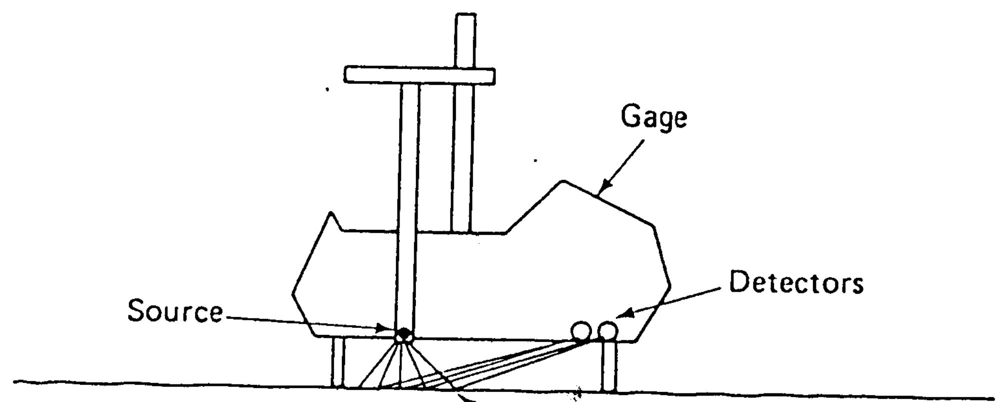
(a)

Compaction # 5



(b)

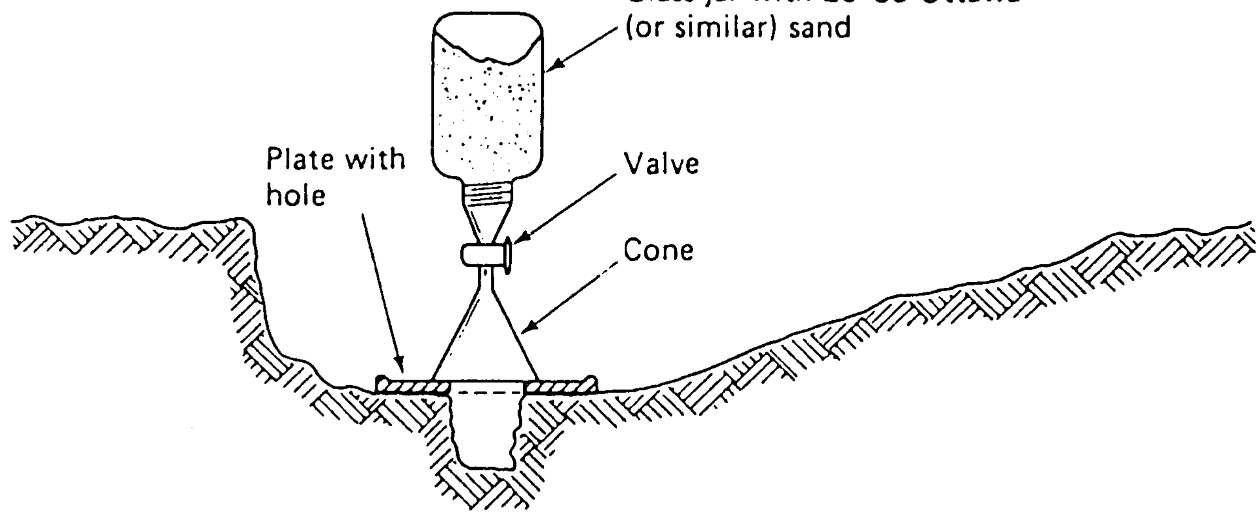
Compaction # 5, continued



(c)

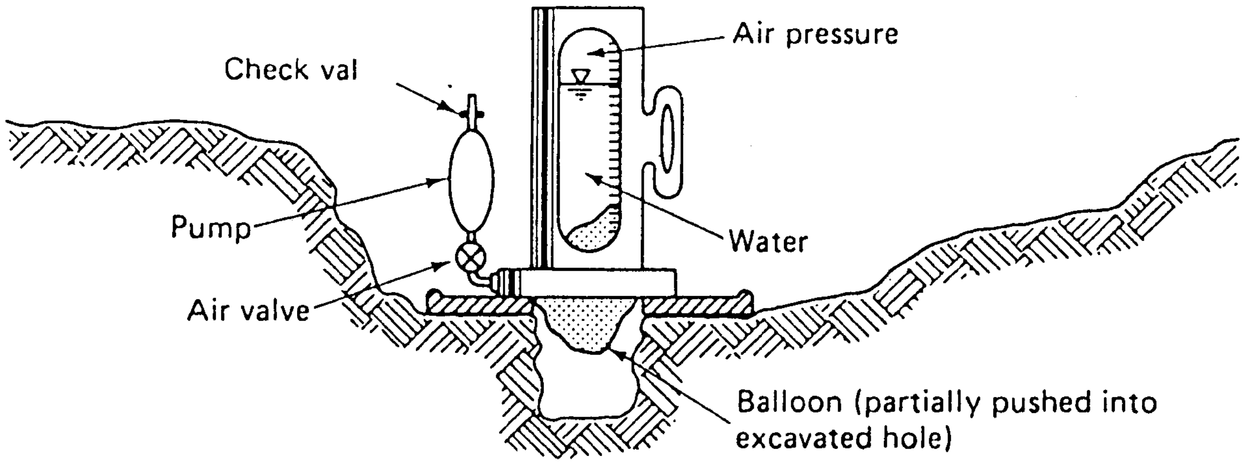
Fig. 5.27 Nuclear density and water content determination: (a) direct transmission; (b) backscatter; (c) air gap (after Troxler Electronic Laboratories, Inc., Research Triangle Park, North Carolina).

Compaction # 5, continued



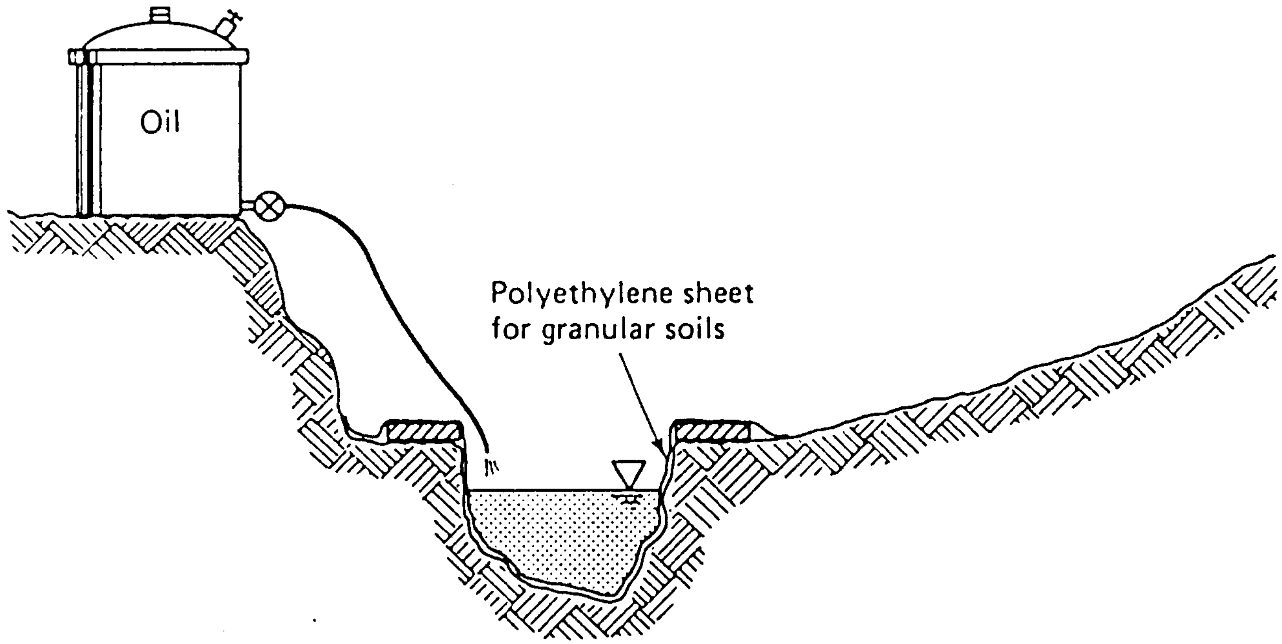
(a) Sand cone

Compaction # 6



(b) Balloon

Compaction # 6, continued



(c) Oil (or water) method

Fig. 5.24 Some methods for determining density in the field.

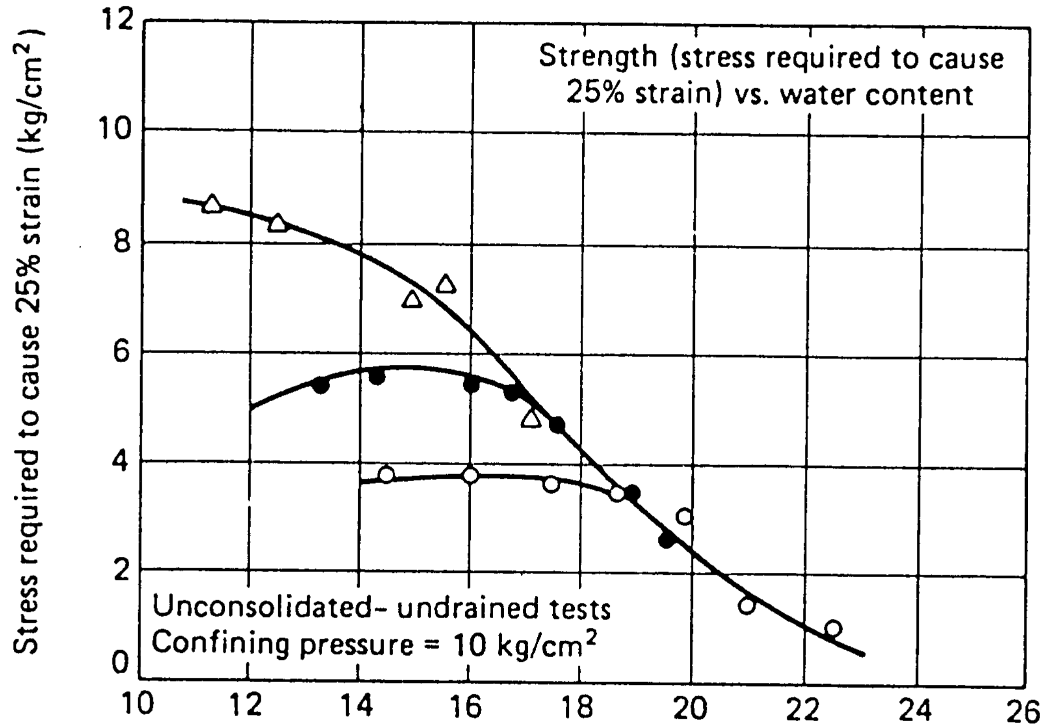
Compaction # 6, continued

TABLE 5-1 Comparison of Soil Properties between Dry of Optimum and Wet of Optimum Compaction*

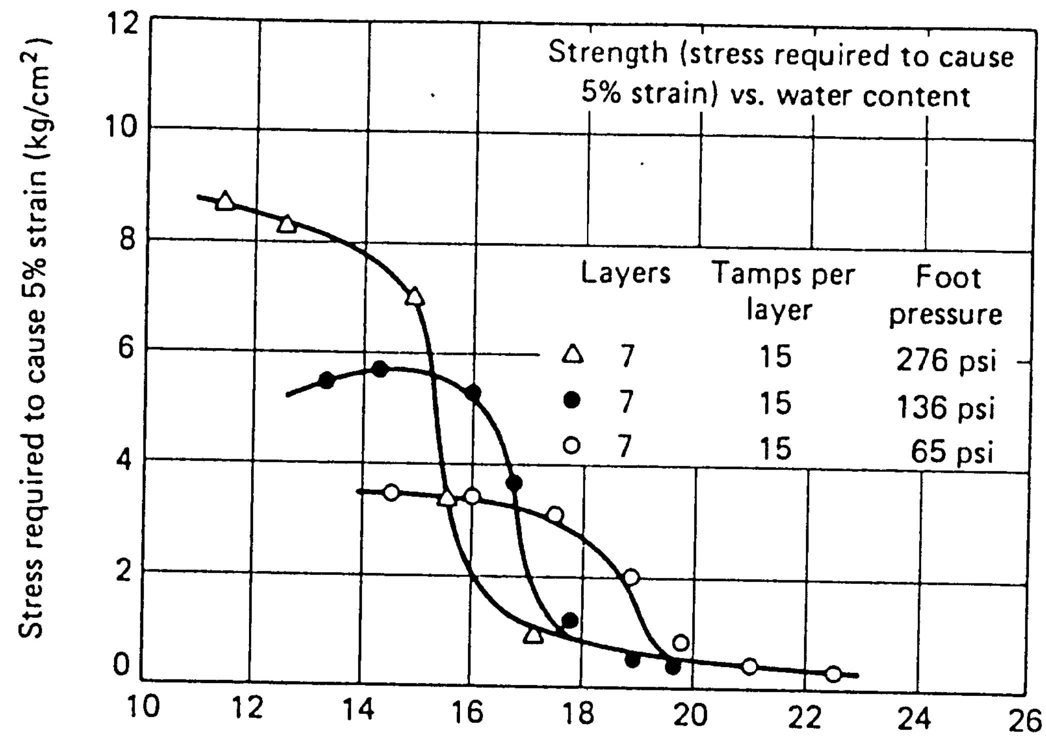
Property	Comparison
1. Structure:	
A. Particle arrangement	Dry side more random
B. Water deficiency	Dry side more deficient; thus imbibes more water, swells more, has lower pore pressure
C. Permanence	Dry side structure sensitive to change
2. Permeability:	
A. Magnitude	Dry side more permeable
B. Permanence	Dry side permeability reduced much more by permeation
3. Compressibility:	
A. Magnitude	Wet side more compressible in low pressure range, dry side in high pressure range
B. Rate	Dry side consolidates more rapidly
4. Strength:	
A. As molded:	
(a) Undrained	Dry side much higher
(b) Drained	Dry side somewhat higher
B. After saturation:	
(a) Undrained	Dry side somewhat higher if swelling prevented; wet side can be higher if swelling permitted
(b) Drained	Dry side about the same or slightly greater
C. Pore water pressure at failure	Wet side higher
D. Stress-strain modulus	Dry side much greater
E. Sensitivity	Dry side more apt to be sensitive

*After Lambe (1958b).

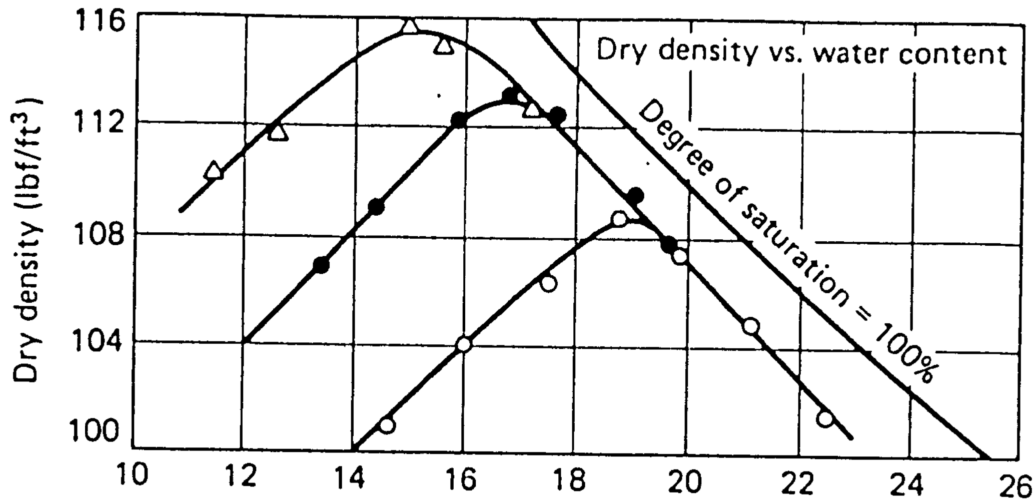
Compaction # 8



Compaction # 9



Compaction # 9, continued



(b)

Fig. 5.7(b) Strength as a function of compactive effort and molding water content (after Seed and Chan, 1959).

Compaction # 9, continued

57

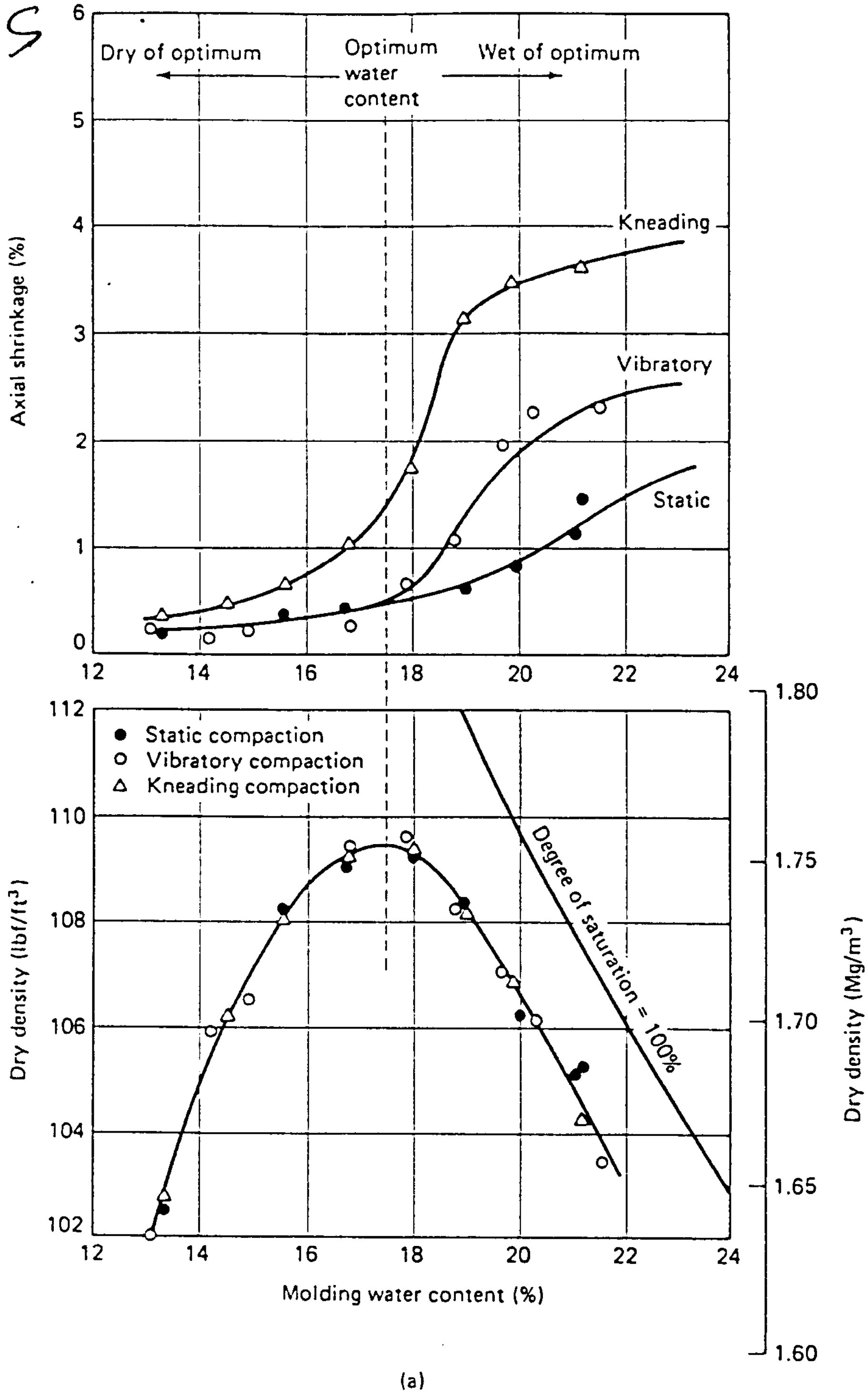
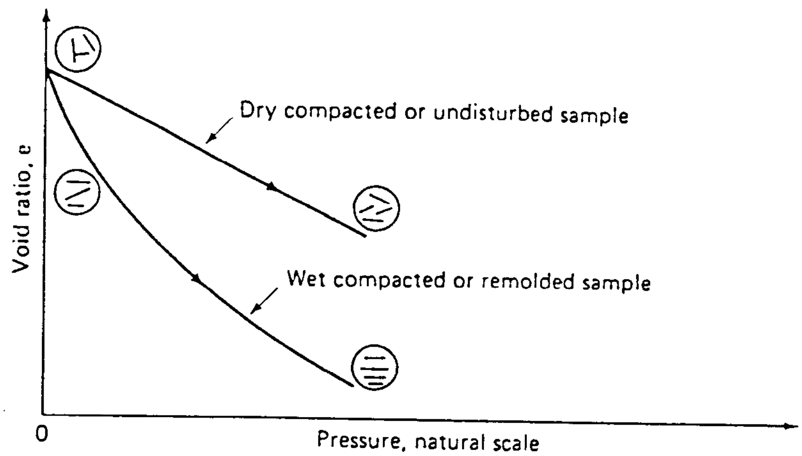


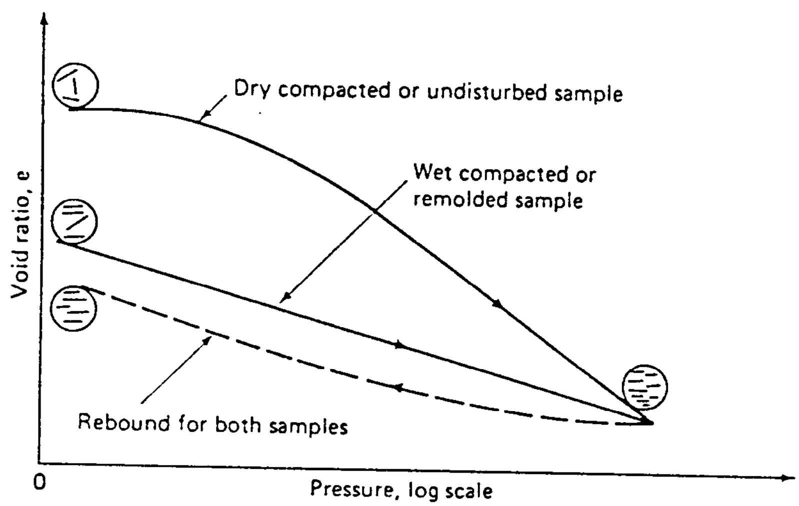
Fig. 5.7(a) Shrinkage as a function of water content and type of compaction (after Seed and Chan, 1959).

Compaction # 10

85



Low pressure consolidation.



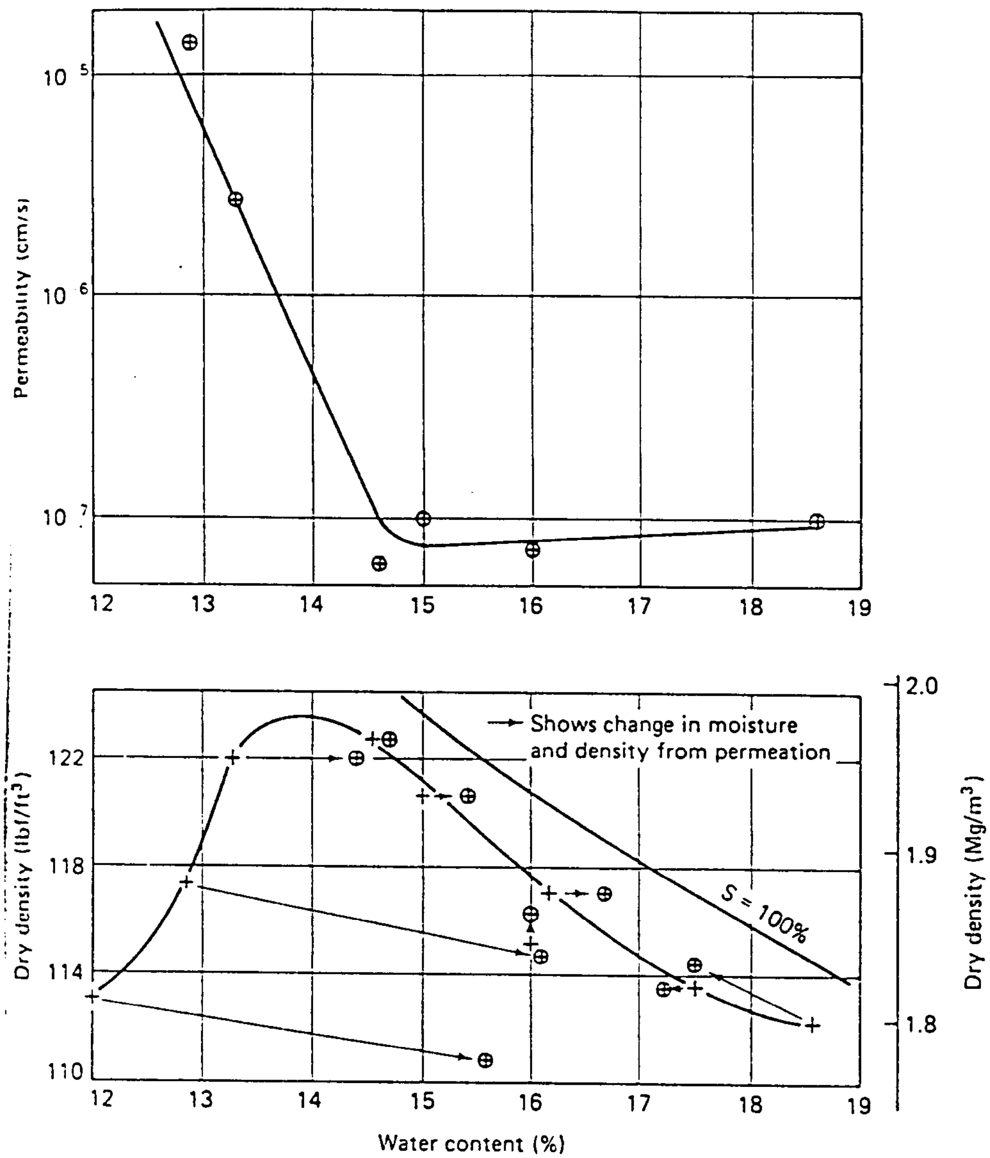
High-pressure consolidation.

(b)

Fig. 5.6(b) Change in compressibility with molding water content (after Lambe, 1958b).

Compaction # 11

bS



(a) Compaction-permeability tests on Jamaica sandy clay.

Fig. 5.6(a) Change in permeability with molding water content (after Lambe, 1958b).

Compaction # 12

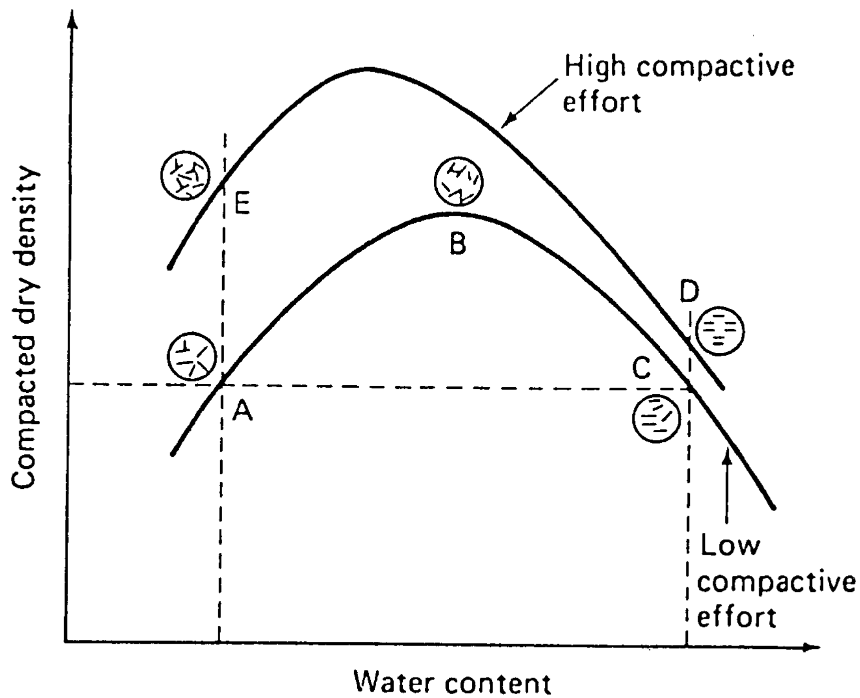


Fig. 5.5 Effect of compaction on soil structure (after Lambe, 1958a).

Compaction # 13

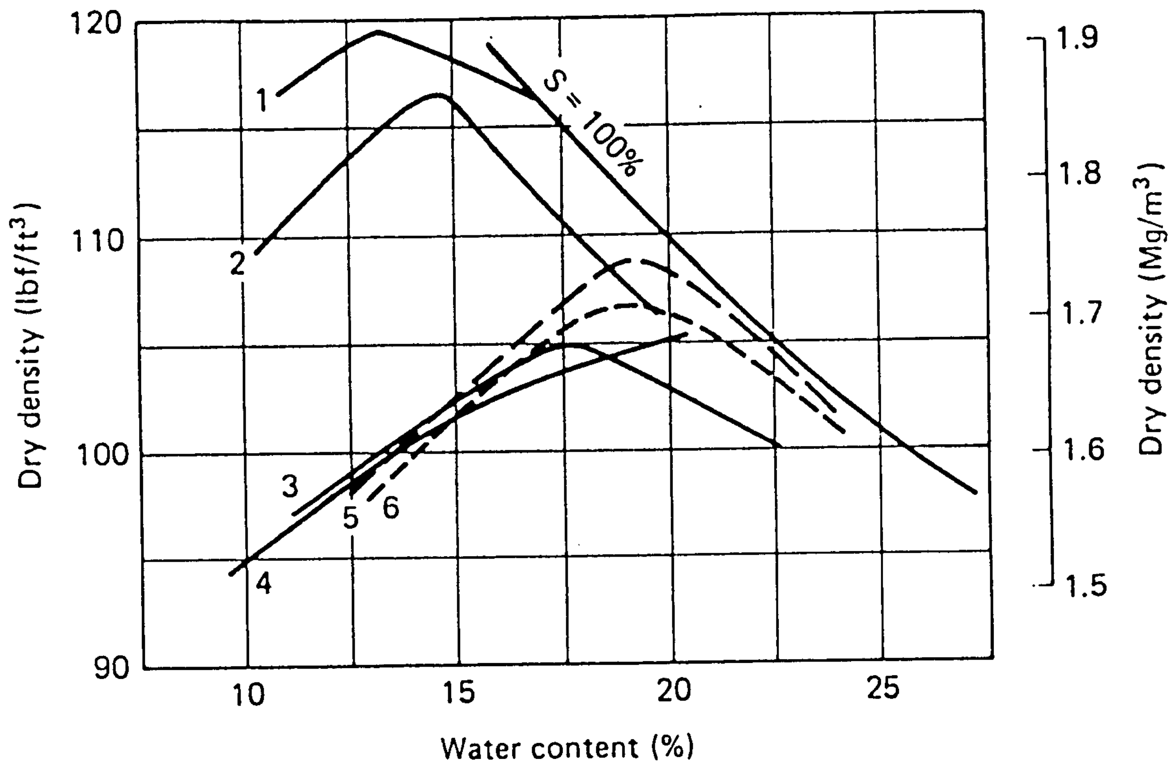


Fig. 5.4 Comparison of field and laboratory compaction. (1) Laboratory static compaction, 2000 psi; (2) modified Proctor; (3) standard Proctor; (4) laboratory static compaction, 200 psi; (5) field compaction, rubber-tired load, 6 coverages; (6) field compaction, sheepsfoot roller, 6 passes. Note: Static compaction from top and bottom of soil sample. (After Turnbull, 1950, and as cited by Lambe and Whitman, 1969.)

Compaction # 14

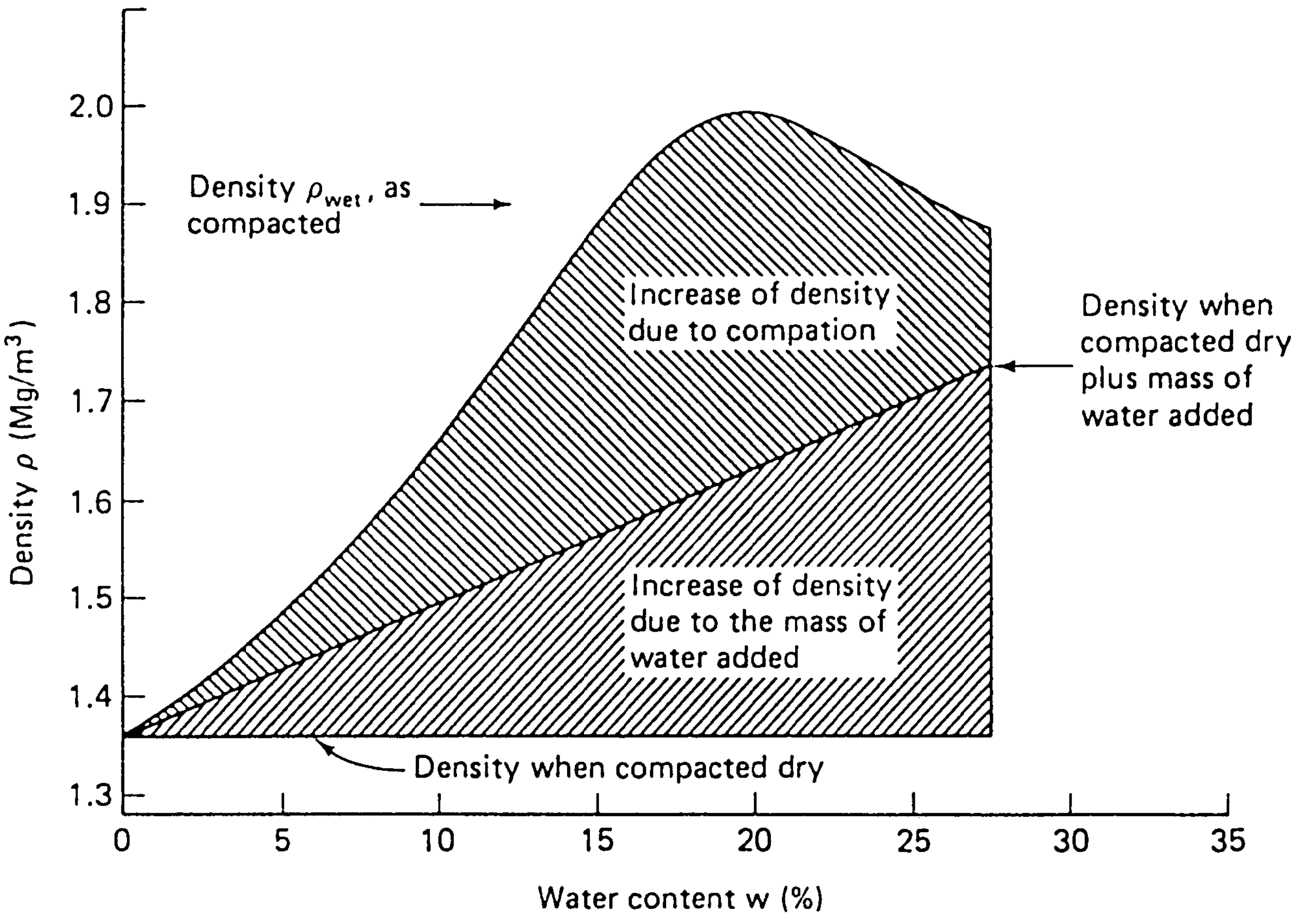


Fig. 5.3 The water content-density relationship indicating the increased density resulting from the addition of water and that due to the applied compaction effort. Soil is a silty clay, LL = 37, PI = 14, standard Proctor compaction (after Johnson and Sallberg, 1960).

62

Compaction # 15

Soil texture and plasticity data

No.	Description	Sand	Silt	Clay	LL	PI
1	Well-graded loamy sand	88	10	2	16	N.P.
2	Well-graded sandy loam	72	15	13	16	N.P.
3	Med-graded sandy loam	73	9	18	22	4
4	Lean sandy silty clay	32	33	35	28	9
5	Lean silty clay	5	64	31	36	15
6	Loessial silt	5	85	10	26	2
7	Heavy clay	6	22	72	67	40
8	Poorly graded sand	94	– 6 –		N.P.	–

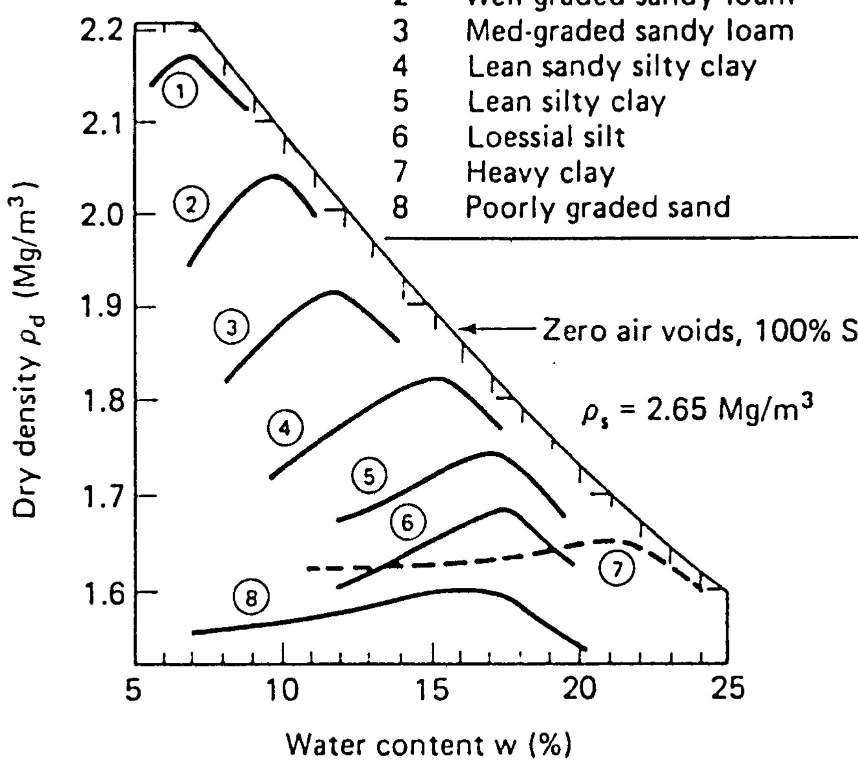


Fig. 5.2 Water content-dry density relationships for eight soils compacted according to the standard Proctor method (after Johnson and Sallberg, 1960).

63 Compaction # 16

$$\rho = \frac{M_t}{V_t} \qquad (2\text{-}1$$

$$\rho_d = \frac{\rho}{1 + w} \qquad (2\text{-}14)$$

When the dry densities of each sample are determined and plotted versus the water contents for each sample, then a curve called a *compaction curve* for standard Proctor compaction is obtained (Fig. 5.1, curve *A*).

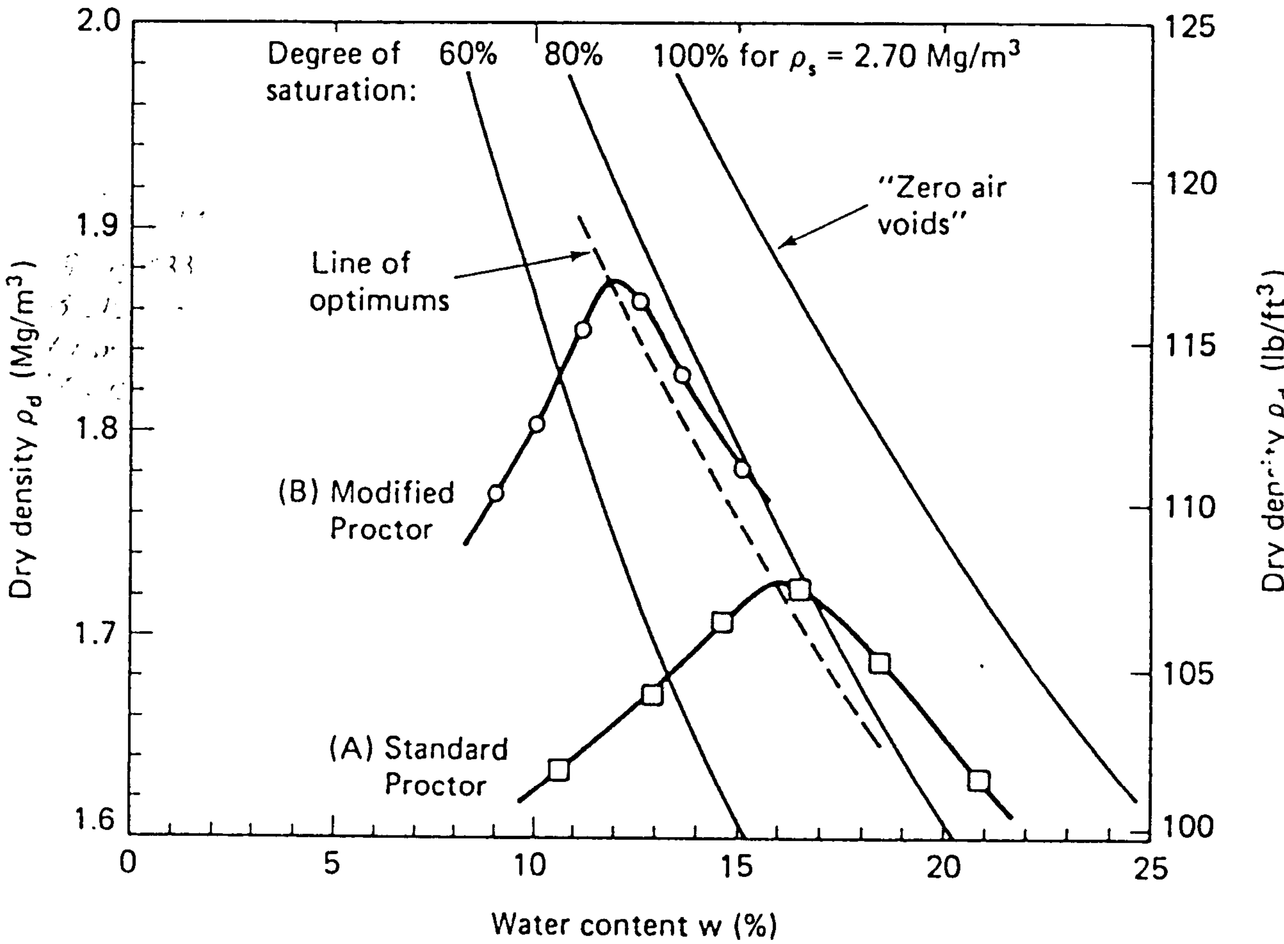
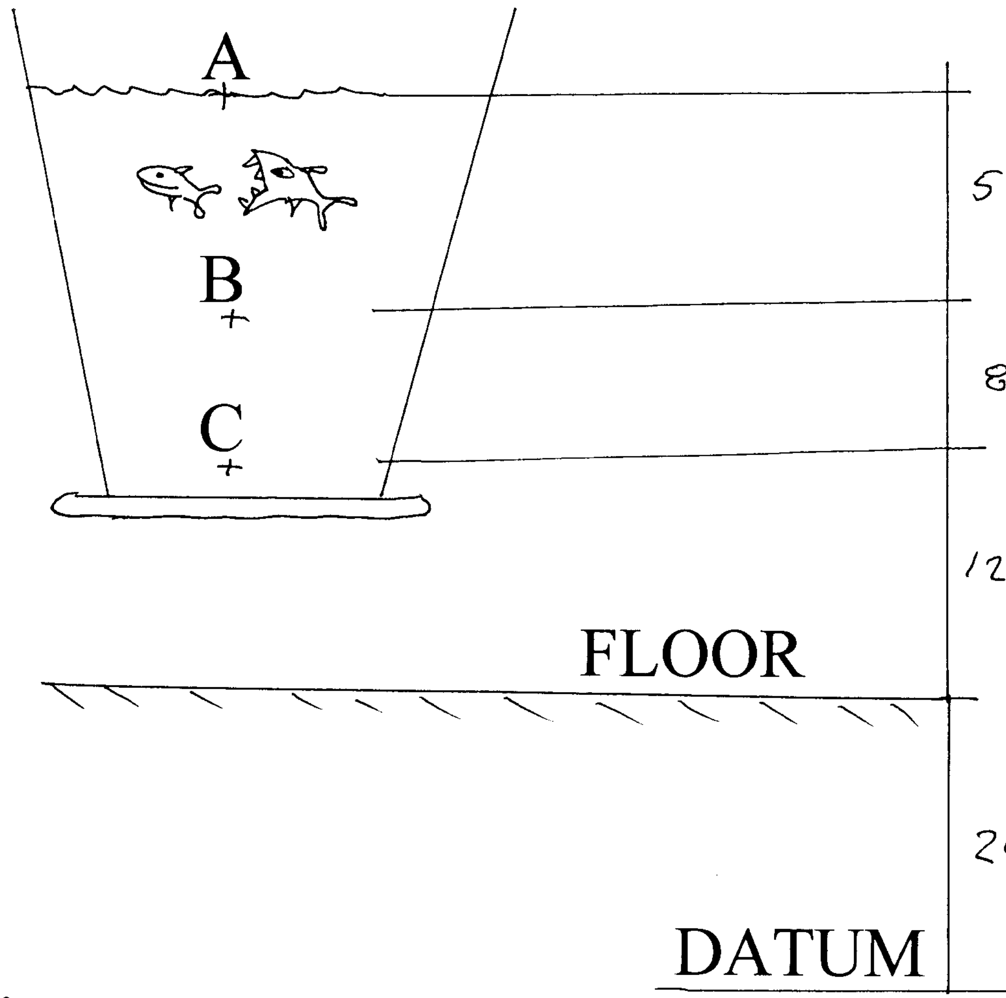


Fig. 5.1 Standard and modified Proctor compaction curves for Crosby B till.

POTENTIAL ENERGY IN WATER



Permeability #1a

What is the elevation head,the pressure head, and the total head at points A, B and C?

	Feet H E A D		
Point	Elevation	Pressure	Total
A	45	0	45
B	40	5	45
C	32	13	45

Pearmeability # 1b

How do you make a flow net? By sketching. For two-dimensional steady-state problems, you simply draw the medium with its boundaries to some convenient scale. By trial and error (mostly error, until you get some practice!) sketch a network of flow lines and equipotential lines spaced so that the enclosed figures resemble "squares." Their sides intersect at right angles. Look again at Fig. 7.15 and the "square" enclosed by flow lines 1 and 2 and equipotential lines *a* and *b*. Not all the "squares" in a flow net have to be the same size either. Since the squares are made of curved lines, they are only squares in the strictest sense when they can be subdivided down the truly equilateral figures. Note that a flow line cannot intersect an impervious boundary; in fact, an impervious boundary is a flow line. Note too that all equipotential lines must meet impervious boundaries at right angles. Neither the number of *flow channels* (channels between flow lines) nor the number of *equipotential drops* (a *drop* is the decrease in head Δh from one equipotential line to the next) need to be a whole number; fractional squares are allowed. Figure 7-16 defines some of the terms associated with flow nets. Look at the "square" with dimensions $a \times b$.

Permeability 2a

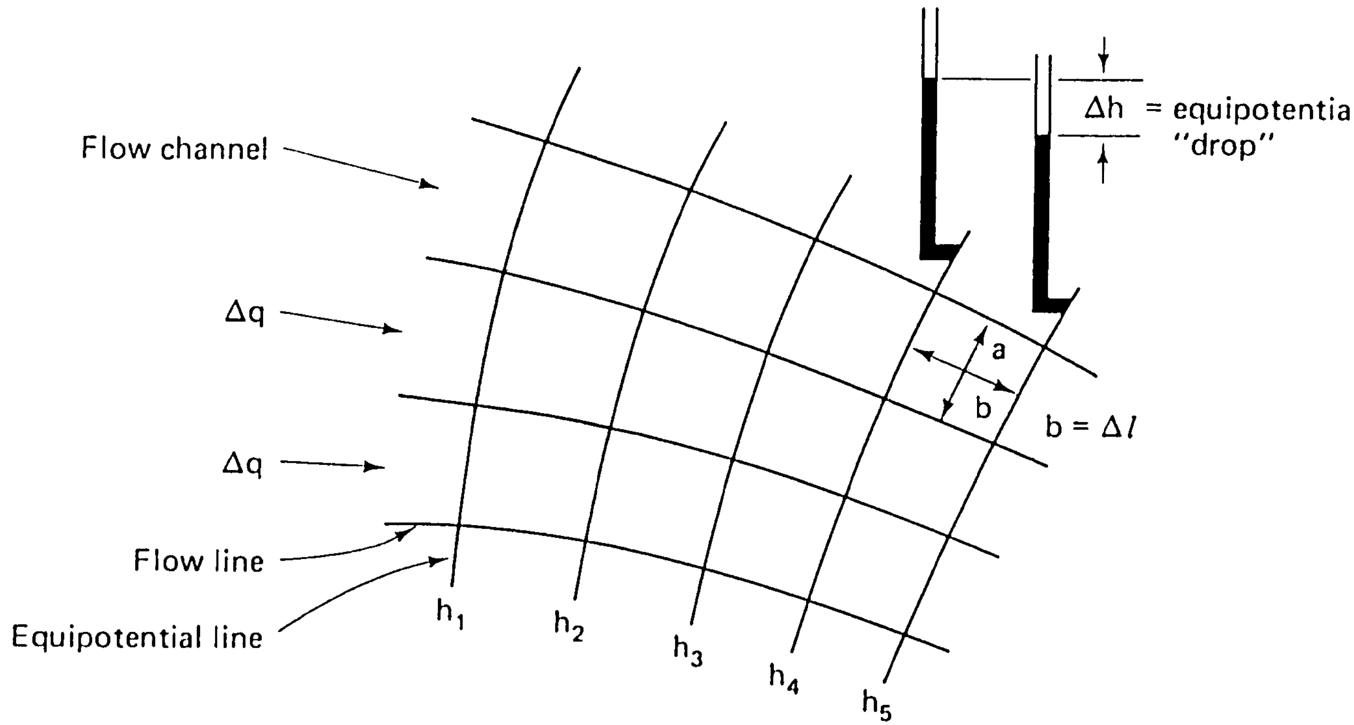


Fig. 7.16 Flow net illustrating some definitions.

Permeability 2b

Note that the gradient is

$$i = \frac{\Delta h}{\Delta l} = \frac{\Delta h}{b} = \frac{h_L/N_d}{b} \tag{7-25}$$

where the length of the flow path in one square is $b = \Delta l$. The equipotential drop between two flow lines is $\Delta h = h_L/N_d$, where N_d is the total number of potential drops, and h_L is the total head lost in the system. From Darcy's law we know that the flow in each flow channel is

$$\Delta q = k\frac{\Delta h}{\Delta l}A = k\left(\frac{h_L/N_d}{b}\right)a$$

and the total discharge q per unit depth (perpendicular to the paper) is

$$q = \Delta q N_f = kh_L\left(\frac{a}{b}\right)\left(\frac{N_f}{N_d}\right) \tag{7-26}$$

where N_f is the total number of flow channels in the flow net. If we sketched squares in our flow net, then $a = b$. Thus we can readily estimate the quantity of flow q by simply counting the number of potential drops N_d and the number of flow channels N_f, if we know the k of the material and the total head loss h_L. Even a crude flow net provides a fairly accurate estimate of the flow quantities.

Permeability 2c

7-38. For the completed flow net of Fig. P7-38, compute the flow under the dam per metre of dam if the coefficient of permeability is 3.5×10^{-4} cm/s.

(1) *Compute elevations*

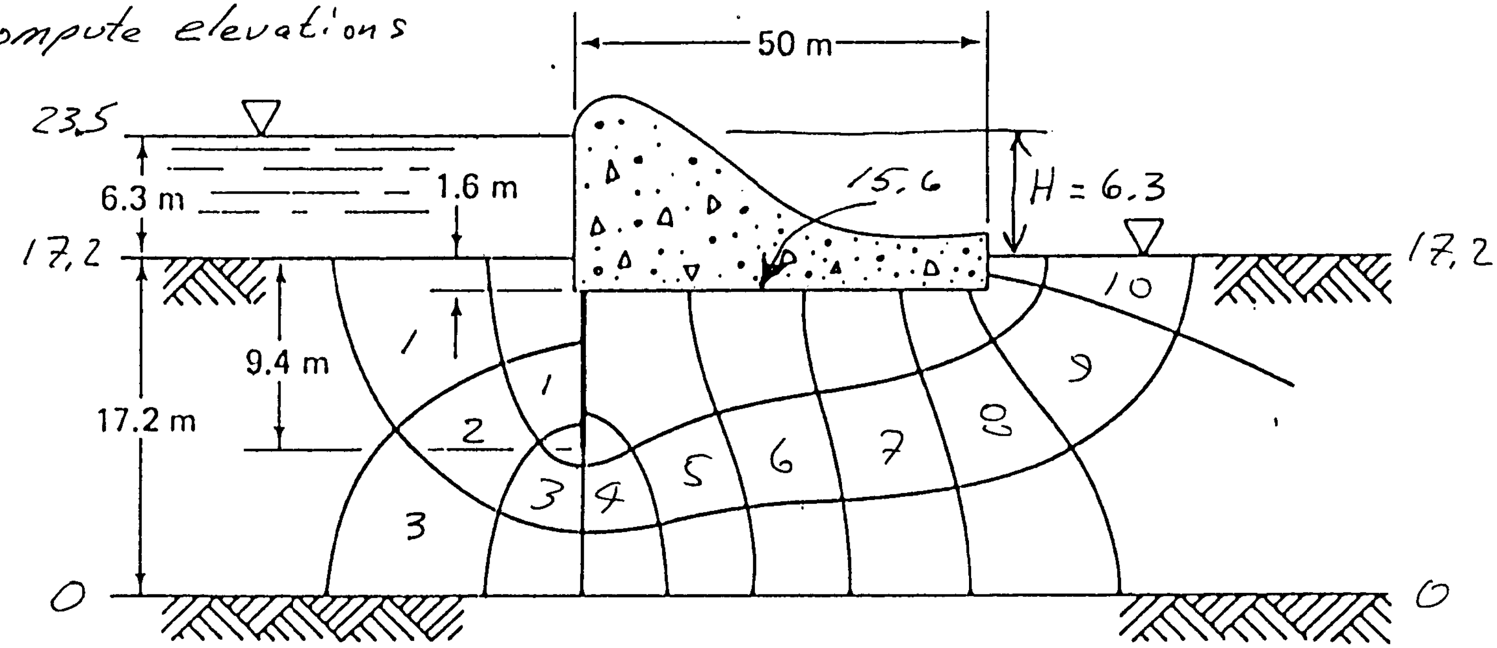


Fig. P7-38

(2) *Number the "flow Channels". Three* $N = 3$

(3) *Number the "drops in energy." Ten* $N_d = 10$

Permeability 3a

7-38. For the completed flow net of Fig. P7-38, compute the flow under the dam per metre of dam if the coefficient of permeability is 3.5×10^{-4} cm/s.

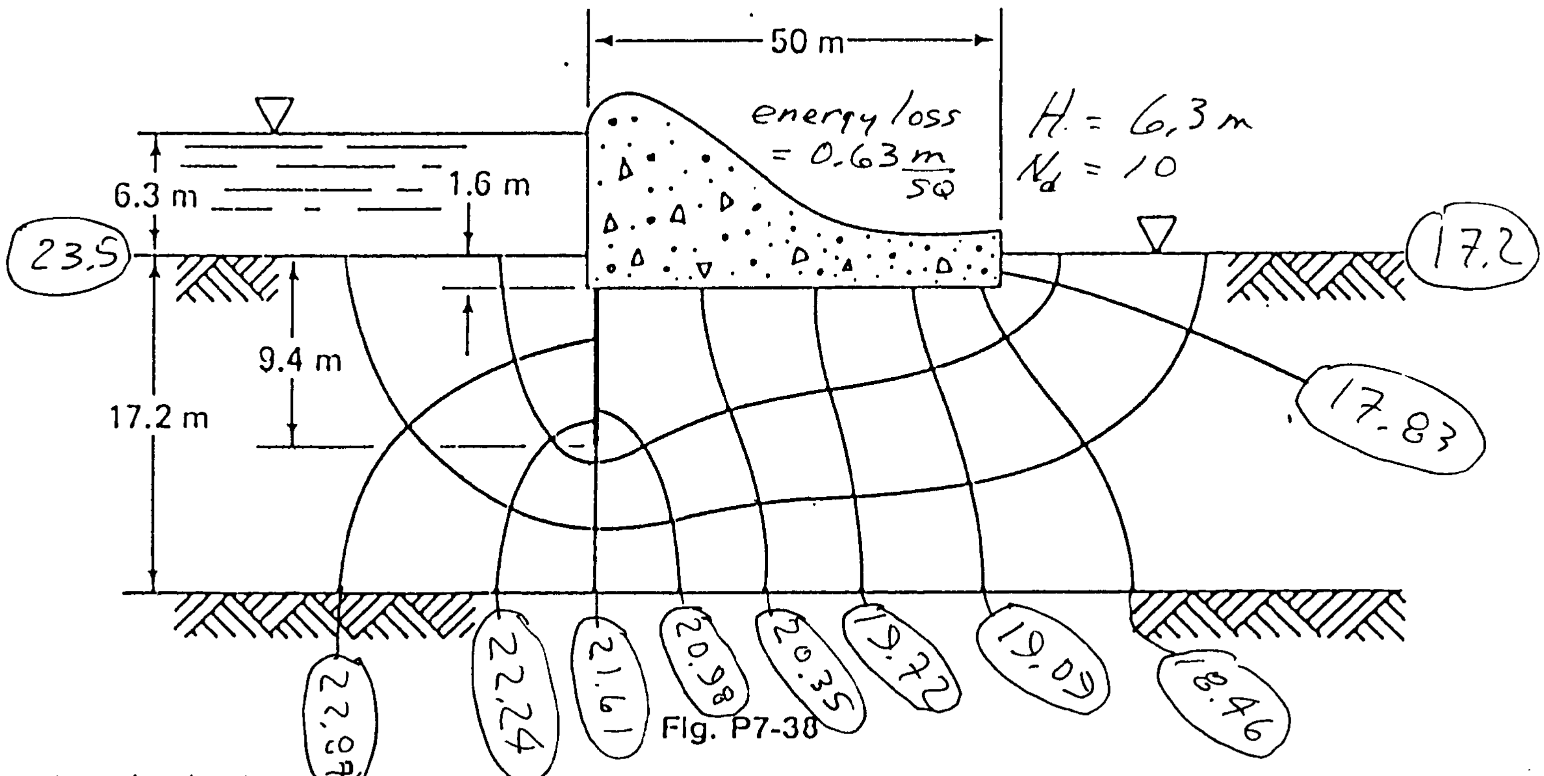


Fig. P7-38

④ Label the lines of equal potential energy.

Permeability 3b

7-38. For the completed flow net of Fig. P7-38, compute the flow under the dam per metre of dam if the coefficient of permeability is 3.5×10^{-4} cm/s.

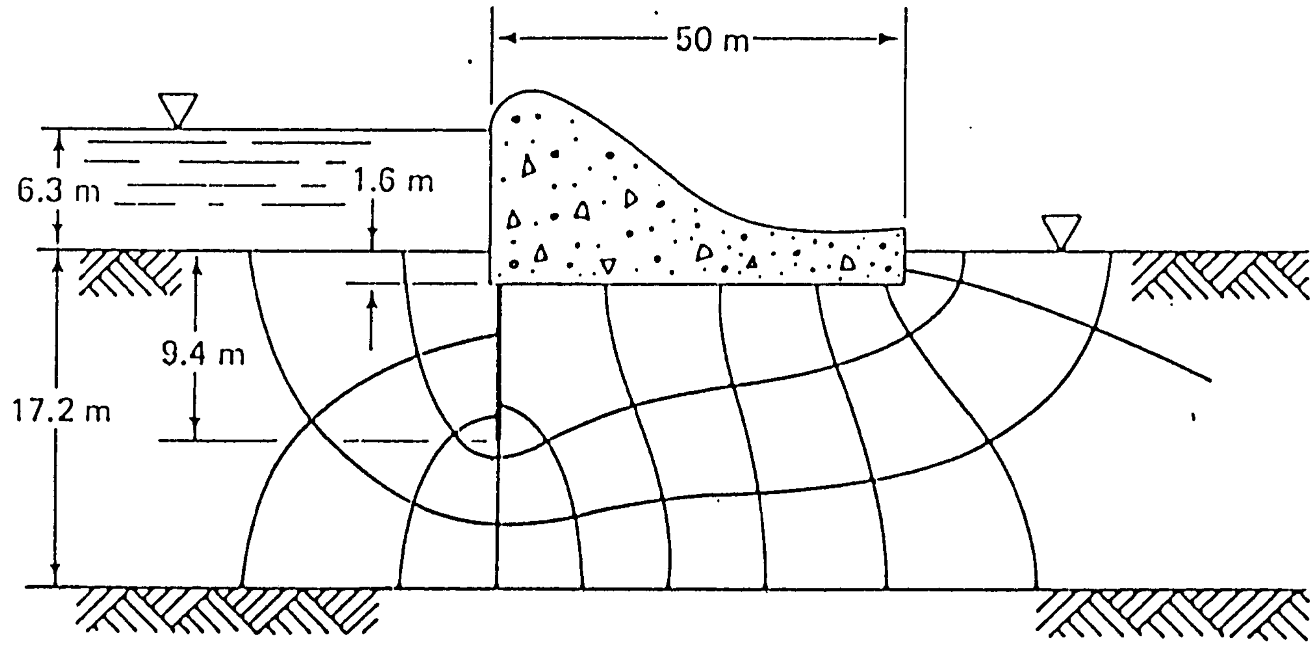


Fig. P7-38

(5) $q = \left(\frac{N_f}{N_d}\right)(H)m(K)\frac{\cancel{cm}}{sec}\frac{m}{(100)\cancel{cm}}\frac{(1)m}{m}$

$q = \left(\frac{3}{10}\right)(6.3)(3.5)(10^{-4})(1)\frac{}{(100)} = \underline{\underline{0.000007}}\ \frac{m^3}{m\ sec}$

Permeability 3c

7-38. For the completed flow net of Fig. P7-38, compute the flow under the dam per metre of dam if the coefficient of permeability is 3.5×10^{-4} cm/s.

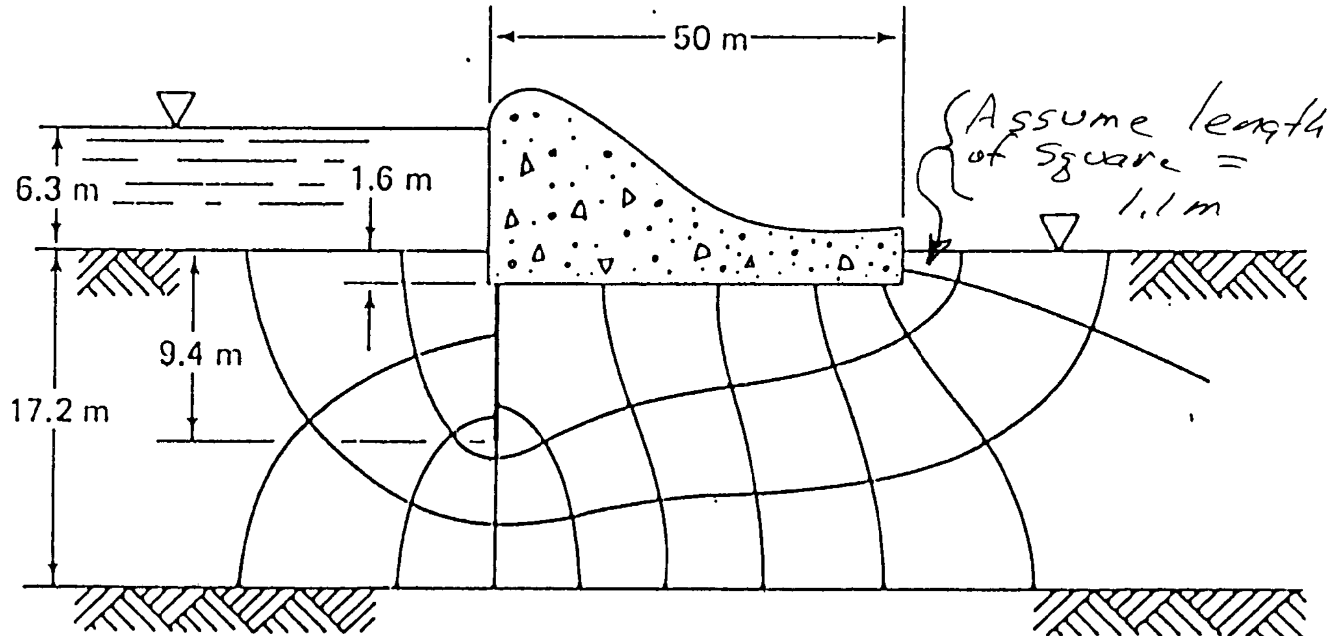


Fig. P7-38

(6) Compute the exit gradient.

Energy loss in square = 0.63 m
Length of square = 1.1 m

$$\text{exit gradient} \approx \frac{0.63}{1.1} = 0.57 \ \frac{m}{m}$$

Permeability 3d

CRITICAL HYDRAULIC GRADIENT

$$i_c = \frac{G_s - 1}{1 + e}$$ where G_s = Specific Gravity, e = Void ratio, and i_c is the critical hydraulic gradient.

Safety Factor = exit gradient/critical gradient

Safety Factor = 1/0.57 = 1.75

7-38. For the completed flow net of Fig. P7-38, compute the flow under the dam per metre of dam if the coefficient of permeability is 3.5×10^{-4} cm/s.

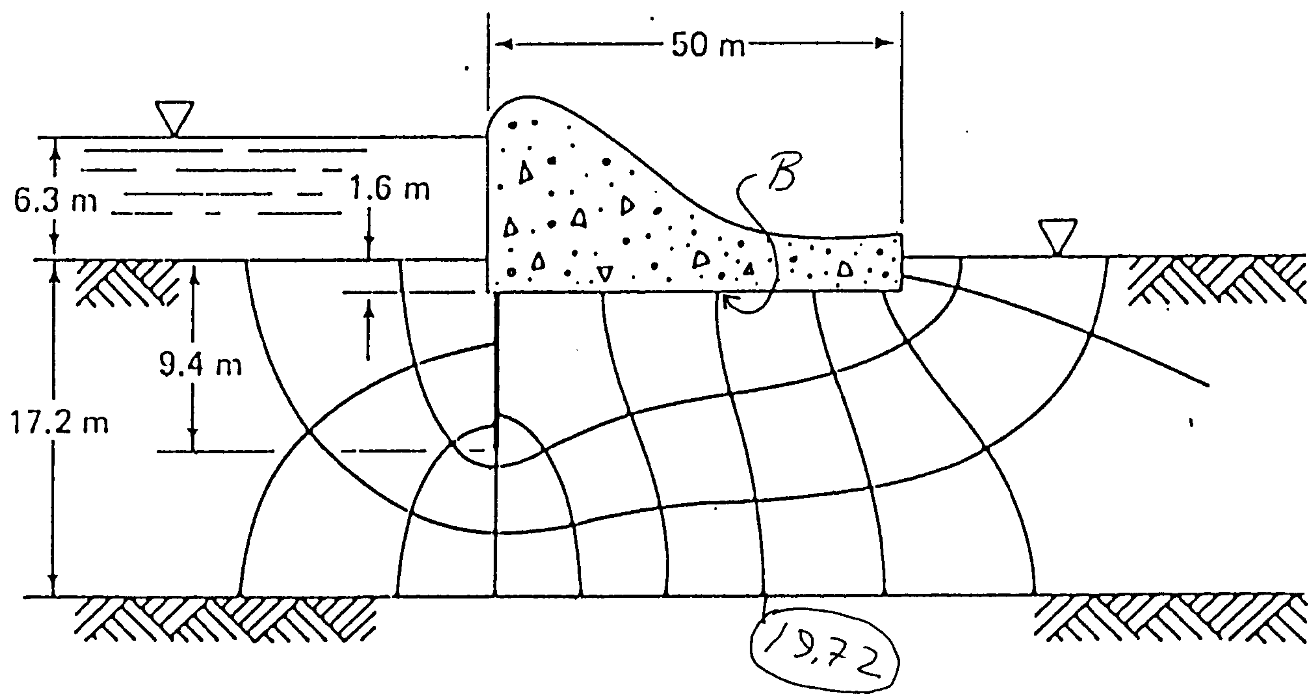


Fig. P7-38

(7) Compute uplift pressure @ B.

Total energy = 19.72 m
Elevation energy = 15.60 m

Pressure head = 4.12 m

Permeability 3f

HOW TO DETERMINE ϕ
ANGLE OF INTERNAL FRICTION

The angle of internal friction is zero for saturated clays and silts loaded rapidly.

The angle of internal friction is determined by measuring slope from Mohr's Circles from triaxial tests on sands, gravels, silts and clays.

Consolidated-Drained Triaxial Tests
Consolidated-Undrained Triaxial Tests

The angle ofinternal friction is determined by the Standard Penetration Test for sands and gravels.

Angle of Internal Friction # 1

15.1 Estimating Soil Strength from Standard Penetration Test Data

One of the most common methods for sampling soil is to drive a split-spoon or split-barrel sampler with a drop hammer. The *standard penetration test* or SPT (ASTM 1586) is a standard procedure for split-spoon sampling which provides a quantitative measure of driving resistance, the standard penetration resistance N. The value N has been widely correlated with soil properties and is commonly used to estimate density and strength of sands and consistency of clays. For the standard penetration test, the split-spoon sampler has an inside diameter of 1-3/8 inches (35mm) and an outside diameter of 2 inches (51 mm). It is driven 18 inches by repeatedly dropping a 140 lb weight from a height of 30 inches. The number of blows are recorded for each six inches of driving. The blows for the second and third increment are added to obtain the standard penetration resistance N.

EXAMPLE 15.1

The following data is recorded from a standard penetration test:

Depth, (feet)	Resistance, (blows)
12.0 - 12.5	8
12.5 - 13.0	11
13.0 - 13.5	13

Determine the standard penetration resistance.

Solution. The resistance for the first increment (8) is discarded. The combined resistance for the last one foot is 11 + 13 = 24. Thus

$$N = 24 \text{ blows/ft}$$

Angle of Internal Friction # 2

For sands, N increases with increasing effective stress and increasing relative density (or strength). For a sand deposit with constant relative density (or strength), N will increase with depth due to the stress increase with depth. In order to use standard penetration test data to estimate strength, it must be corrected to a constant vertical effective stress. Several correction equations have been proposed; the Liao and Whitman (1986) equation is simple and fits most research results. The actual or uncorrected N value may be adjusted to the corrected value N_1 expected under a reference stress of 1 ton per square foot (2000 psf) as:

$$N_1 = CN \tag{15.1.1}$$

where

$$C = \sqrt{\frac{1}{\sigma'_v}}$$

The above equation is unit dependent, and σ'_v is the vertical effective stress in tons per square foot.

EXAMPLE 15.2

The data in Example 15.1 was obtained at a depth of 15 feet in a thick sand deposit. The water table is at 10 feet. The unit weight of moist sand above the water table is estimated to be 115 lb/ft^3, the total unit weight of saturated sand below the water table is estimated to be 125 lb/ft^3. Determine the corrected N value.

Solution. The vertical effective stress is

$$\sigma'_v = (10)(115) + (5)(125) - (5)(62.4)$$

$$= 1460 \text{ lb/ft}^2 \text{ or } 0.73 \text{ tsf}$$

$$C = \sqrt{\frac{1}{0.73}} = 1.17$$

$$N_1 = CN = (1.17)(24) = 28.1 \cong 28$$

The corrected N value is 28. If the sand sample with $N = 24$ under a stress of 0.73 tsf were at a greater depth where the vertical effective stress was 1 tsf, a standard penetration value of 28 would be expected. The 28 value should be used to estimate relative density and strength values.

Angle of Internal Friction # 3

15.2 Estimating Clay Strength from Cone Penetration Test Data

The standard penetration test is termed an *in-situ* test as it is performed in the ground at the site of the sample. A more sophisticated in-situ test for determining the undrained strength of clay, coming into more common use, is the *cone penetration test* (ASTM D 3441) or *Dutch cone test*. A rod with a conical tip is pushed into the ground. The conical tip has a cross-sectional area of 10 cm^2 and an apex angle of 60 degrees. The force required to advance the tip is measured mechanically or electrically and divided by the tip area to get the *cone resistance* q_c which has units of pressure. As advancing the cone causes a bearing capacity failure, bearing capacity theory can be used to backcalculate the undrained strength. The undrained strength s_u is calculated as:

$$s_u = \frac{q_c - \sigma_v}{N_k} \tag{15.2.1}$$

where q_c is the cone resistance
σ_v is the total vertical overburden pressure
N_k is the cone factor, which is site-dependent and empirical

N_k is commonly in the range of 10 to 30, often about 15 or 20.

79 Angle of Internal Friction # 4

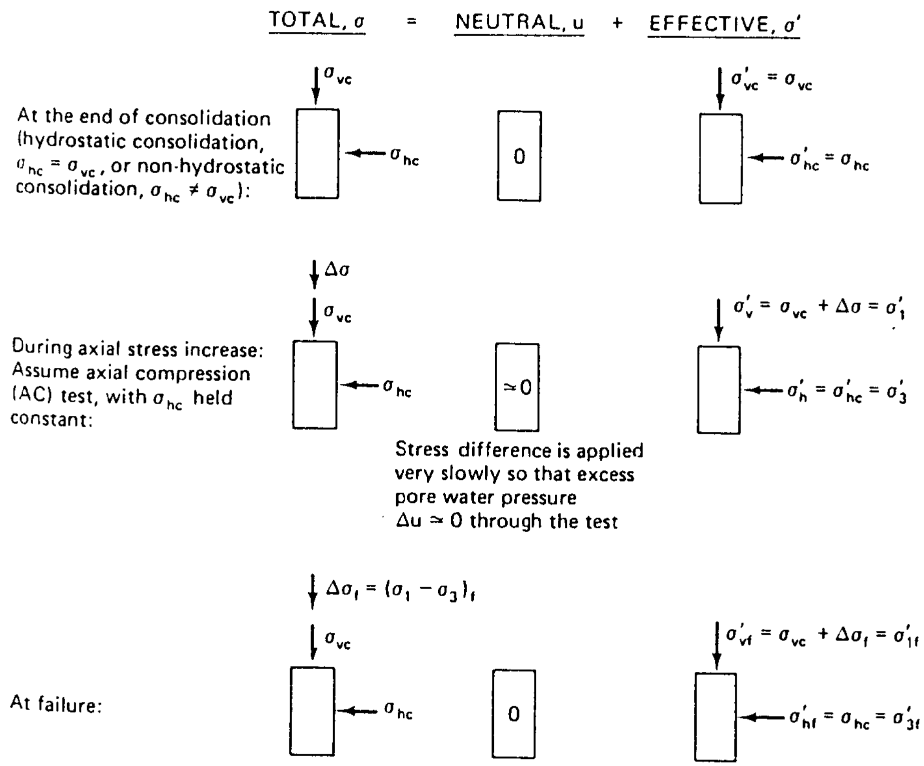
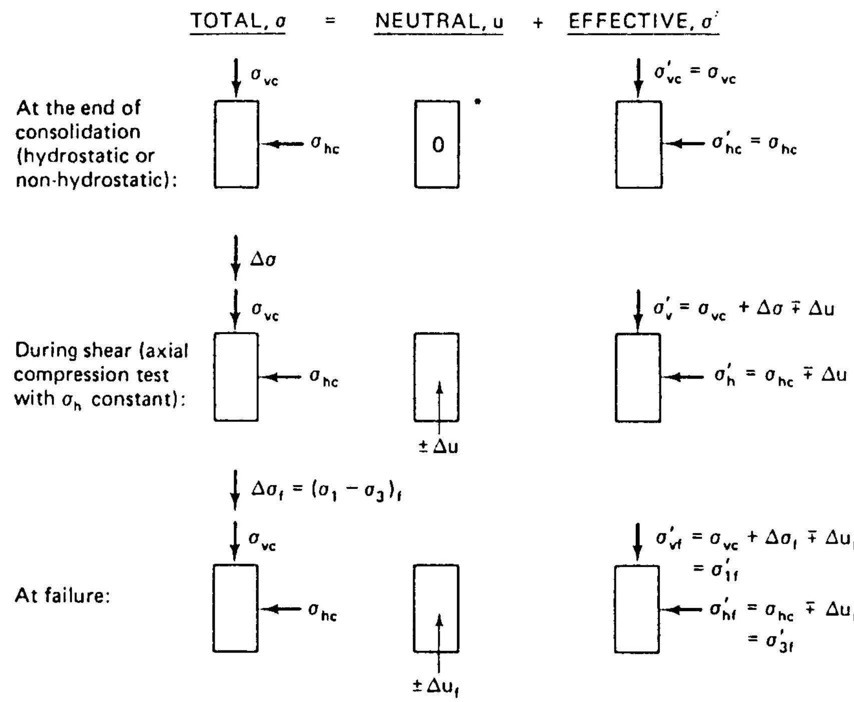


Fig. 11.23 Stress conditions in the consolidated-drained (CD) axial compression triaxial test.

Triaxial Test # 1

Triaxial Test # 2



*In practice, to ensure 100% saturation, which is necessary for good measurements of the pore water pressure, a *back pressure* is applied to the pore water. To keep the effective consolidation stresses constant, the total stresses during consolidation are accordingly increased by an amount exactly equal to the applied back pressure, which is the same as raising atmospheric pressure by a constant amount—the effective stresses on the clay do not change.

Example: Initial conditions with back pressure:

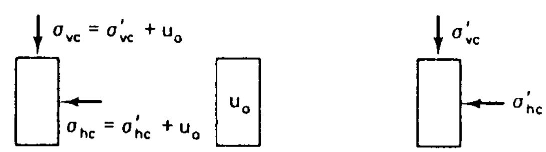


Fig. 11.29 Conditions in specimen during a consolidated-undrained axial compression (CU) test.

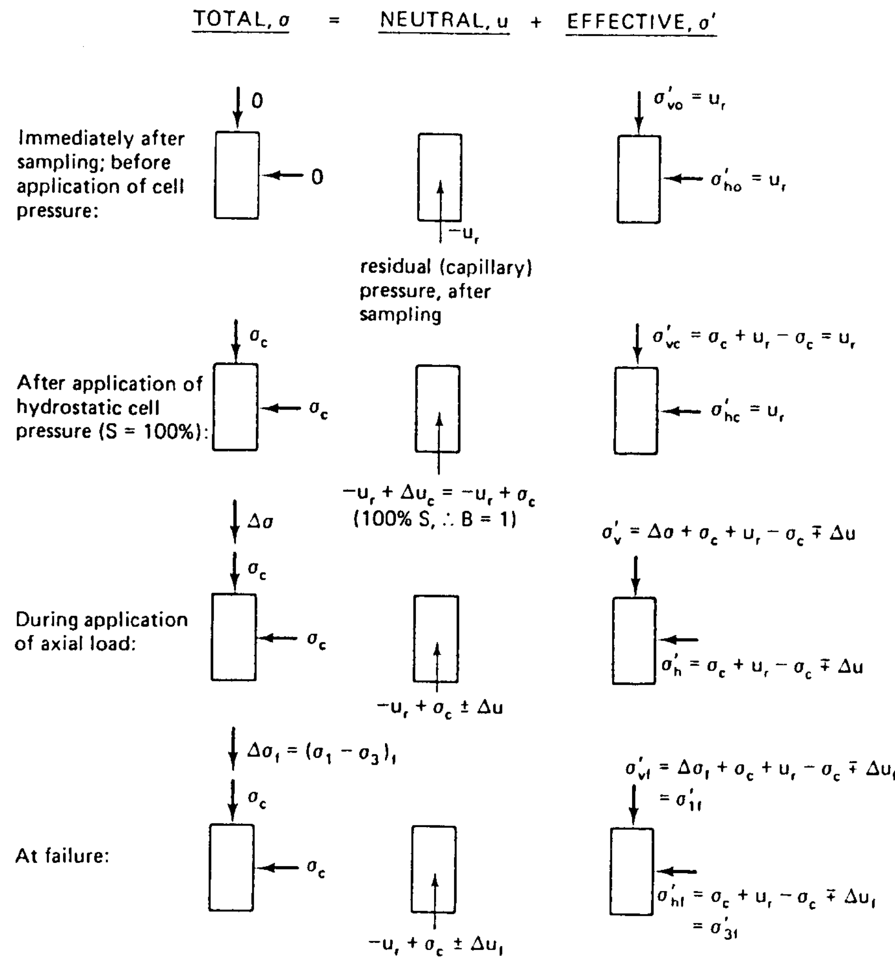


Fig. 11.38 Conditions in the specimen during the unconsolidated-undrained (UU) axial compression test.

Triaxial Test # 3

TERZAGHI-MEYERHOFF EQUATION

Friction Term Cohesion Term Surcharge Term

$$p_g = \underbrace{\frac{1}{2}\gamma B N_\gamma}_{1} + \underbrace{c N_c}_{2} + \underbrace{(p_q + \gamma D_f) N_q}_{3} \qquad 10.1$$

p_g = gross or ultimate bearing capacity (gross pressure)

Friction Term 1

γ_2 = effective unit weight of soil below base of footing

B = least dimension of footing

$N\gamma$ = Terzaghi bearing capacity due to friction

Cohesion Term 2

c = cohesion

N_c = Terzaghi bearing capacity due to chhesion

Surcharge Term 3

p_g = surface surcharge

γ_1 = effetive unit weight abovebase of footing

D_f = depth of footing

N_q = Terzaghi bearing capacity due to surcharge

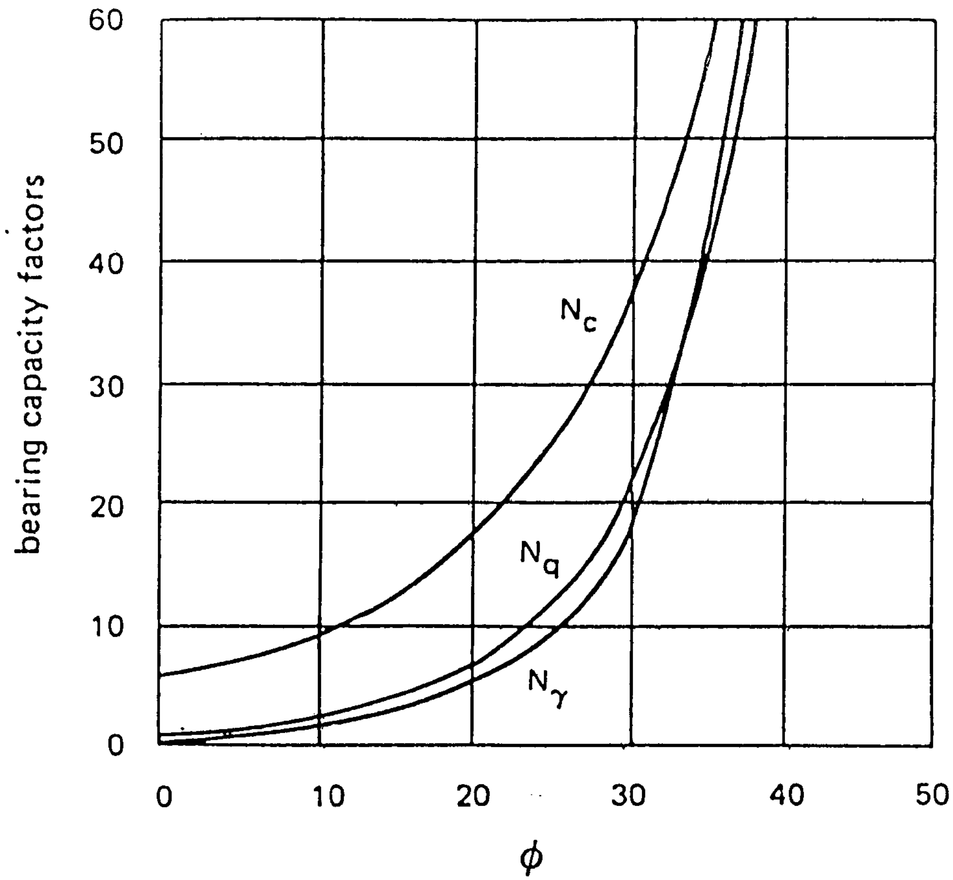


Figure 10.2 Bearing Capacity Factors

98 Foundation 1d

Table 10.2

N_c Bearing Capacity Factor Multipliers for Various Values of B/L

(See figure 10.3)

B/L	multiplier
1 (square)	1.25
0.5	1.12
0.2	1.05
0.0	1.00
1 (circular)	1.20

Table 10.3

Terzaghi Bearing Capacity Factors for General Shear[3]

ϕ	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
34	52.6	36.5	35.0
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
48	258.3	287.9	780.1
50	347.5	415.1	1153.2

3 In *general shear*, the soil resists an increased load until failure is sudden. There is another case, that of *local shear*, which results with looser soil. However, it is unlikely that foundations would be designed for loose soil without compaction. With compaction, the general shear case holds.

Table 10.4

N_γ Multipliers for Various Values of B/L

(See figure 10.3)

B/L	multiplier
1.0 (square)	0.85
1.0 (circular)	0.70
0.5	0.90
0.2	0.95
0.0	1.0

FOUNDATIONS - SPECIAL CASES

Footing on Surface - no surcharge, no term 3.

Footing on Sand - no cohesion, no term 2.

Footing on Saturated Clay with load applied rapidly - no friction, no term 1. Use the unconfined compression strength for c (c = unconfined compression strength/2).

FOUNDATION ON CLAY
(Shear Failure)

1. Use Terzaghi-Meyerhoff Equation (10.1)
2. Use Table 10.3 to find N_c, N_q, and N_γ
3. Correct N_c and $N\gamma$ for B/L
4. If a water table is located "near" the footing use appropriate values for $\gamma_{effective}$
5. Use the appropriate safety factor (or load factor)
   Generally 3 for "average conditions"
   Generally 2 for "improbable combinations" of wind, snow, siesmic and other loads

EXAMPLE 1

$\phi = 25$ degrees

If the proposed foundation is 6 feet wide by 12 feet long:

$B/L = 6/12 = 0.5$

What is N_γ?

$(9.7)(0.9) = 8.7$

Table 10.3 Table 10.4

What is N_c?

$(25.1)(1.12) = 28.1$

Table 10.3 Table 10.2

What is N_q?

$(12.7)(1) = 12.7$

Table 10.3 No Correction

FOUNDATION ON SURFACE OFSATURATED CLAY WITH LOAD APPLIED RAPIDLY WATER TABLE AT SURFACE

$p_g = 1 + 2 + 3$

Saturated Clay	On Surface
No 1	No surcharge, no 3

$p_g = c\ N_c$ corrected for BL (Table 10.2)

No water table correction is required for cohesion term

continued

B/L correction varies from a low of 1.0 (long beam) to a high of 1.25 (square). The correction will be made later.

$\phi = 0$ degrees, therefore $N_c = 5.7$

S_{nc} = unconfined compressive strength
c = cohesion = $S_{nc}/2$
$p_g = c\, N_c = N_c\, S_{nc}/2$
$p_{net} = p_g - \gamma D_f$ (Eqn 10-2), but γD_f is zero
Therefore, $p_{net} = N_c\, S_{nc} = 5.7 * S_{nc}/2$

$p_{allow} = p_{net}/F = 5.7 * S_{nc}/2$ Let $F = 3$ $p_{allow} = 0.95 * S_{nc}$

 Foundation 5b

continued

Now let's make the B/L correction. The correction varies from 1.0 to 1.25.

p_{allow} is equal to 0.95 to 1.19 times the unconfined compressive strength of the soil with a safety factor of 3 if the foundation is on the surface of a saturated clay with a rapidly applied load.

SQUARE FOUNDATION ON SATURATED CLAY
LOAD APPLIED RAPIDLY
FOUNDATION ON SURFACE

An individual square column footing carries a live load of 50,000 pounds and a dead load of 30,000 pounds. The unconfined compressive strength of the clay is 0.8 tons/sq ft.

p_{allow} is 1.19 times the unconfined compressive strength or 0.95 tons/sq ft or 1,904 lbs/sq ft if a safety factor of 3 is used for the entire load.

Area = 80,000 pounds/1,904 lbs/sq ft = 42.02 sq ft or B = 6.5 ft (6.482 ft).

Foundation 6a

continued

B/L correction varies from a low of 1.0 (long beam) to a high of 1.25 (square). The correction will be made later.

o = 0 degrees, therefore N_c =5.7

S_{nc} = unconfined compressive strength
c = cohesion = $S_{nc}/2$
$p_g = c\ N_c = N_c\ S_{nc}/2$
$p_{net} = p_g - \gamma D_f$ (Eqn 10-2), but γD_f is zero
Therefore, $p_{net} = N_c\ S_{nc} = 5.7*S_{nc}/2$

$p_{allow} = p_{net}/F = 5.7*S_{nc}/2$ Let $F = 3$ $p_{allow} = 0.95*S_{nc}$

 Foundation 5b

continued

Now let's make the B/L correction. The correction varies from 1.0 to 1.25.

p_{allow} is equal to 0.95 to 1.19 times the unconfined compressive strength of the soil with a safety factor of 3 if the foundation is on the surface of a saturated clay with a rapidly applied load.

MOMENTS ON FOOTINGS

If a moment is exerted on a footing the values of B and L are adjusted.

In Equation 10.1 use the adjusted value of B in term 1.

Use the real values of B & L in Table 10.2 and 10.4.

Use the real value of B in Table 10.3.

Use the adjusted values of B & L in the following equation:

$$P = p_{allow} * B' * L'$$

ADJUST B

$P = 2{,}000$ pounds, $M_B = 1{,}000$ ft lbs, $B = 4$ ft

$\epsilon_B = M_B/P = 1{,}000$ ft lbs/2,000 lb $= 0.5$ ft

$B' = B - 2^* {}_B = 4 - 2^*0.5 = 3$ ft

ADJUST L

$P = 2{,}000$ pounds, $M_L = 1{,}600$ ft lbs, $L = 8$ ft

$\epsilon_B = 1{,}600$ ft lbs/2,000 lbs $= 0.8$ ft

$B' = 8 - 2^*0.8 = 6.4$ ft

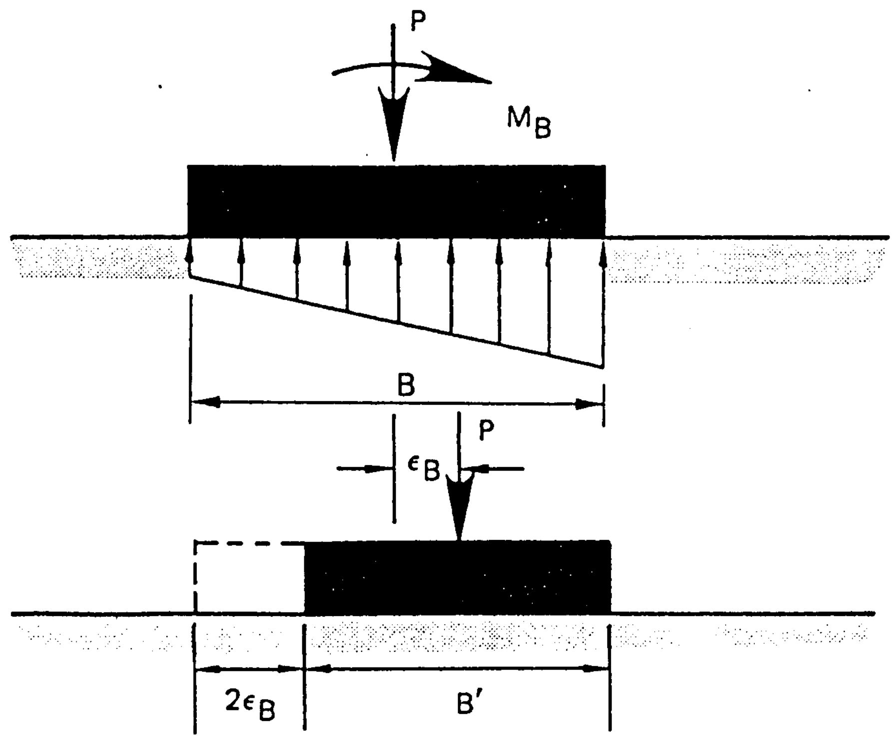


Figure 10.5 A Footing with an Overturning Moment

101 Foundation # 7c

GENERAL CONSIDERATIONS FOR RAFTS

A **raft** or a **mat** is a combined footing-slab that covers the entire area beneath a building and supports all walls and columns.

- o Rafts are generally used when the required foundations cover half or more of the area beneath a building.

- o Rafts are generally large, see Appendix C.

  - For square raft the increase in vertical stress in the soil is 0.1 of the applied stress at the mat at a distance of 2B. For an infinitely long raft the depth is about 6.5B.

- For example, a mat with a least dimension of 60 feet will stress the soil down to a depth of 120 feet or more.

- Subsidence is important and sometimes the soils data isn't available to sufficient depth.

o The solution is to drill deeper exploratory holes or to make the rafts **fully compensated**.

A raft is said to be a **Fully Compensated Foundation**

o when the overburden removed from the excavation for the raft is greater than the stress caused by the total load (including the foundation).

Raft # 2

- At a depth below the **Fully Compensated Foundation** there is **no** increase in stress due to the construction of the raft.

A **Partially Compensated Foundation** is constructed at a depth that is less than the depth required for full compensation.

$$F = \frac{cN_c}{\frac{\text{total load}}{\text{raft area}} - \gamma D_f} \qquad 10.16$$

Raft # 3

13 RAFTS ON SAND

Rafts on sand are always well protected against bearing capacity failure. Therefore, settlement will govern the design. Since differential settlement will be much smaller for various locations on the raft (due to the raft's rigidity), the allowable soil pressure may be doubled.

$$p_a = 0.22 C_n N \ (\text{tons/ft}^2) \qquad 10.17$$

N should always be at least 5 after correcting for overburden. Otherwise, the sand should be compacted or a pier/pile foundation used.

The net soil pressure should be compared with the allowable pressure. The net soil pressure is

$$p_{\text{net}} = \frac{\text{total load}}{\text{raft area}} - \gamma D_f \qquad 10.18$$

Table 10.5
Overburden Corrections

overburden		C_n
0	tons/ft^2	2
0.25		1.45
0.5		1.21
1.0		1.00
1.5		0.87
2.0		0.77
2.5		0.70
3.0		0.63
3.5		0.58
4.0		0.54
4.5		0.50
5.0		0.46

 Raft # 5

FOOTINGS ON SAND

$$p_a = (0.11)*C_n*N \qquad \text{Eqn. } 10.11$$

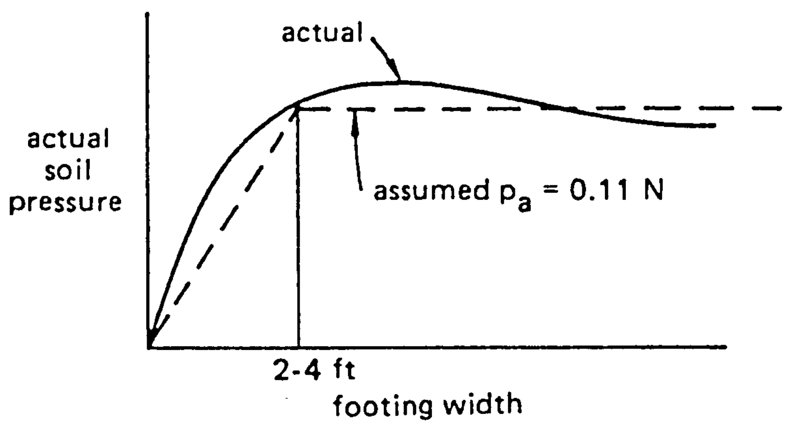


Figure 10.4 Soil Pressure on Sand with Constant Settlement

Table 10.5
Overburden Corrections

overburden	C_n
0 tons/ft^2	2
0.25	1.45
0.5	1.21
1.0	1.00
1.5	0.87
2.0	0.77
2.5	0.70
3.0	0.63
3.5	0.58
4.0	0.54
4.5	0.50
5.0	0.46

107 Footings on Sand # 1

CHANGE IN STRESS AT A DEPTH DUE TO A FOUNDATION LOAD SQUARE FOUNDATION

A foundation has B = 10 and it has a uniform load of 3,000 psf (including the weight of the footing). What is the change in stress under the center at a depth of 2B under the ground surface for:

a. Foundation on the surface

Contact Pressure = 3,000 psf

Depth Below Footing = 2B

Change in Stress = (3,000) psf (0.1) = 300 psf

Stress at Depth # 1

b. Foundation in the bottom of a 5 ft cut (cut same size as foundation).

Contact Pressure = 3,000 psf

Depth Below Footing = 1.5 B

Change in Stress = $3{,}000 - \overset{600}{(5)(120)} = (2400)\,psf\,(0.21)$

$= 504$ psf

c. Foundation on top of a 5 ft fill with a B of 200 feet.

Contact Pressure = 3,000 psf

Depth Below Footing = 2.5 B

Change in Stress = $(3000)\,psf\,(0.06) + \overset{600}{(5)(120)}(1.0)$

$= 180 + 600$

$= 780$ psf

Stress at Depth # 2

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$B = 20'$ $L = 20'$ $q = 3000$ psf

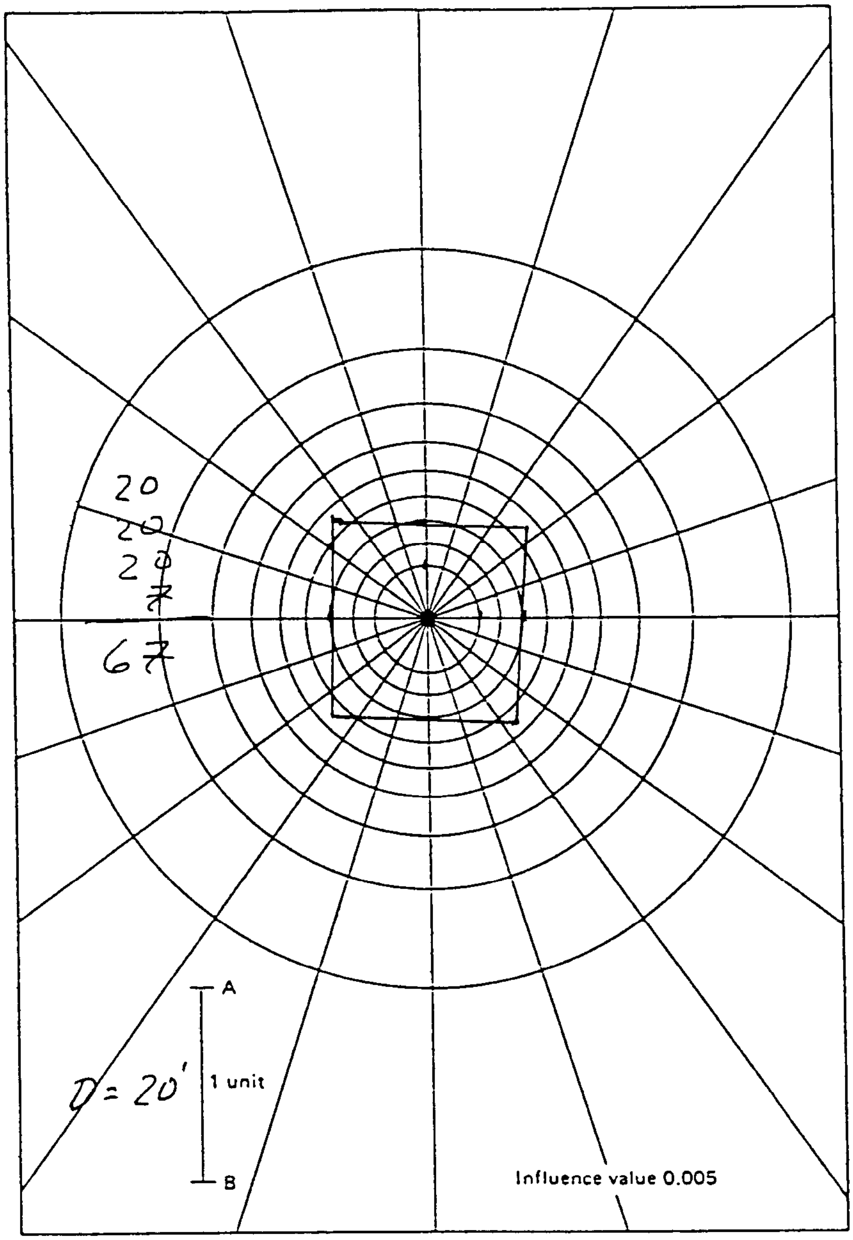

Figure 10.11 Influence Chart

Δ Stress = $(0.005)(67)(3000)$ psf = 1,005 psf

$(0.005)(67) = 0.335$

Stress at Depth # 3

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Table 10.6
Friction Angles

interface materials*	friction angle, δ, degrees**
concrete or masonry on the following foundation materials:	
clean, sound rock	35
clean gravel, gravel-sand mixtures, and coarse sand	29–31
clean fine to medium sand, silty medium to coarse sand, and silty or clayey gravel	24–29
clean fine sand, and silty or clayey fine to medium sand	19–24
fine sandy silt, and non-plastic silt	17–19
very stiff clay, and hard residual or preconsolidated clay	22–26
medium stiff clay, stiff clay, and silty clay	17–19
steel sheet piles against the following soils:	
clean gravel, gravel-sand mixtures, and well-graded rock fill with spalls	22
clean sand, silty sand-gravel mixtures, and single-size hard rock fill	17
silty sand, gravel or sand mixed with silt or clay	14
fine sandy silt, and non-plastic silt	11
formed concrete or concrete sheet piling against the following soils:	
clean gravel, gravel-sand mixtures, and well-graded rock fill with spalls	22–26
clean sand, silty sand-gravel mixtures, and single-size hard rock fill	17–22
silty sand, and gravel or sand mixed with silt or clay	17
fine sandy silt, and non-plastic silt	14
miscellaneous combinations of structural materials:	
masonry on masonry, igneous and metamorphic rocks:	
dressed soft rock on dressed soft rock	35
dressed hard rock on dressed soft rock	33
dressed hard rock on dressed hard rock	29
masonry on wood (cross grain)	26
steel on steel at sheet-steel interlocks	17

* Angles given are ultimate values. Sufficient movement is required before failure will occur.

** For materials not listed, use $\delta = \frac{2}{3}\phi$.

Friction Angle # 1

Table 10.6
Friction Angles

interface materials*	friction angle, δ, degrees**
concrete or masonry on the following foundation materials:	
clean, sound rock	35
clean gravel, gravel-sand mixtures, and coarse sand	29–31
clean fine to medium sand, silty medium to coarse sand, and silty or clayey gravel	24–29
clean fine sand, and silty or clayey fine to medium sand	19–24
fine sandy silt, and non-plastic silt	17–19
very stiff clay, and hard residual or preconsolidated clay	22–26
medium stiff clay, stiff clay, and silty clay	17–19
steel sheet piles against the following soils:	
clean gravel, gravel-sand mixtures, and well-graded rock fill with spalls	22
clean sand, silty sand-gravel mixtures, and single-size hard rock fill	17
silty sand, gravel or sand mixed with silt or clay	14
fine sandy silt, and non-plastic silt	11
formed concrete or concrete sheet piling against the following soils:	
clean gravel, gravel-sand mixtures, and well-graded rock fill with spalls	22–26
clean sand, silty sand-gravel mixtures, and single-size hard rock fill	17–22
silty sand, and gravel or sand mixed with silt or clay	17
fine sandy silt, and non-plastic silt	14
miscellaneous combinations of structural materials:	
masonry on masonry, igneous and metamorphic rocks:	
dressed soft rock on dressed soft rock	35
dressed hard rock on dressed soft rock	33
dressed hard rock on dressed hard rock	29
masonry on wood (cross grain)	26
steel on steel at sheet-steel interlocks	17

* Angles given are ultimate values. Sufficient movement is required before failure will occur.

** For materials not listed, use $\delta = \frac{2}{3}\phi$.

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Friction Angle # 2

Table 10.6
Friction Angles

interface materials*	friction angle, δ, degrees**
concrete or masonry on the following foundation materials:	
clean, sound rock	35
clean gravel, gravel-sand mixtures, and coarse sand	29–31
clean fine to medium sand, silty medium to coarse sand, and silty or clayey gravel	24–29
clean fine sand, and silty or clayey fine to medium sand	19–24
fine sandy silt, and non-plastic silt	17–19
very stiff clay, and hard residual or preconsolidated clay	22–26
medium stiff clay, stiff clay, and silty clay	17–19
steel sheet piles against the following soils:	
clean gravel, gravel-sand mixtures, and well-graded rock fill with spalls	22
clean sand, silty sand-gravel mixtures, and single-size hard rock fill	17
silty sand, gravel or sand mixed with silt or clay	14
fine sandy silt, and non-plastic silt	11

Friction Angle # 3

formed concrete or concrete sheet piling against the following soils:	
clean gravel, gravel-sand mixtures, and well-graded rock fill with spalls	22–26
clean sand, silty sand-gravel mixtures, and single-size hard rock fill	17–22
silty sand, and gravel or sand mixed with silt or clay	17
fine sandy silt, and non-plastic silt	14
miscellaneous combinations of structural materials:	
masonry on masonry, igneous and metamorphic rocks:	
dressed soft rock on dressed soft rock	35
dressed hard rock on dressed soft rock	33
dressed hard rock on dressed hard rock	29
masonry on wood (cross grain)	26
steel on steel at sheet-steel interlocks	17

* Angles given are ultimate values. Sufficient movement is required before failure will occur.

** For materials not listed, use $\delta = \frac{2}{3}\phi$.

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Friction Angle # 4

PILE GROUP

A pile group will not carry the full capacity of the total number of piles. If the piles are friction piles the group may carry the load computed by using the area in friction around the pile group if the piles are not spaced more than about 3D center-to-center. If the piles are point bearing piles the group may carry the load of a footing with the area describedby the pile group (again limited by a spacing of about 3D).

 Pile Group # 1

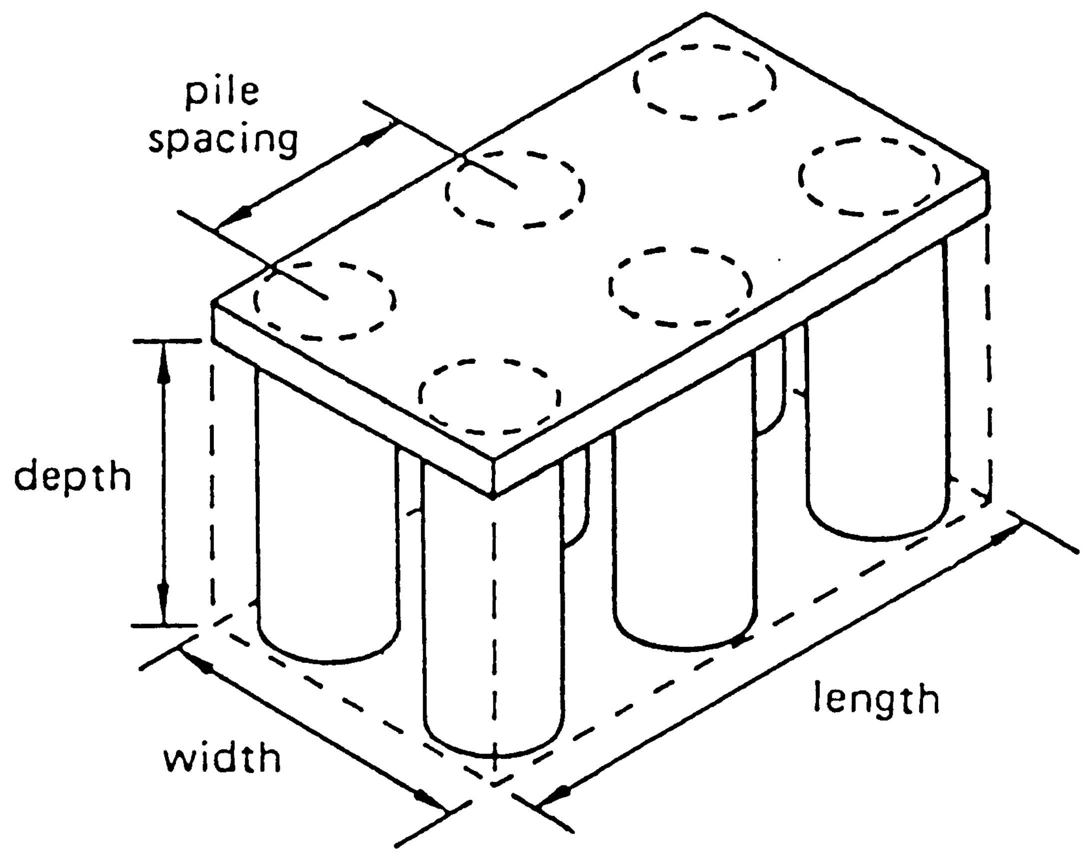


Figure 10.9 A Pile Group

Pile Group # 2

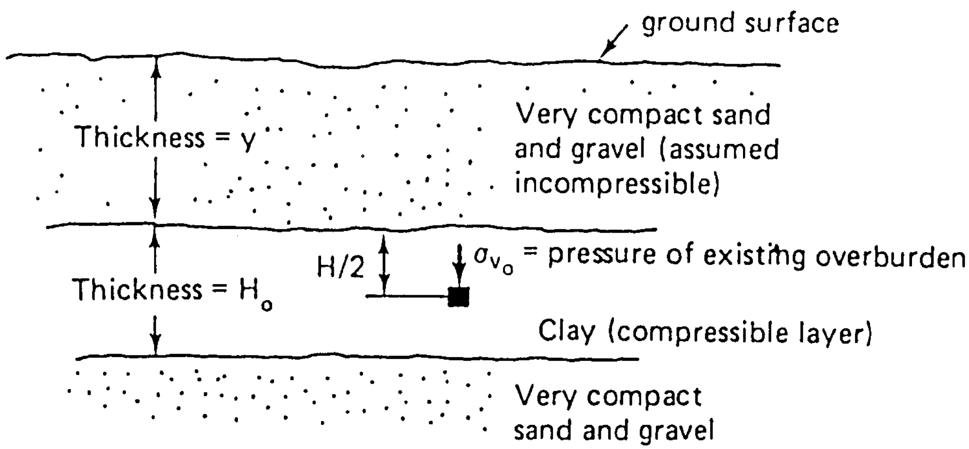


Figure 9-12. Soil conditions for problem of settlement due to compression in a buried clay layer.

117 Consolidation # 1a

Illustration 9-1: (a) Problem worked in U.S. customary units: Referring to the conditions shown by Fig. 9.12, assume that y is twelve ft, H_o is eight ft, and γ_{sand} and γ_{clay} are 135 pcf and 110 pcf, respectively. For the clay, e_o is 1.20 and the compression index C_c is 0.20 (both dimensionless values). The weight of the structure causes a stress of 600 psf at mid-height of the clay layer. (Therefore, $\Delta\sigma_v = 600$ psf). [*Note:* Values of stress increase, $\Delta\sigma_v$, are computed by using methods described in Chapter 8.]

The settlement would be

$$\Delta H = \frac{H_o}{1 + e_0} C_c(\log \sigma_{v_f} - \log \sigma_{v_0})$$

where $\sigma_{v_0} = (12 \text{ ft} \times 135 \text{ pcf}) + (4 \text{ ft} \times 110 \text{ pcf}) = 2060 \text{ psf}$

and $\sigma_{v_f} = \sigma_{v_0} + \Delta\sigma_v = (2060 + 600) \text{ psf} = 2660 \text{ psf}$

$$\Delta H = \frac{8 \text{ ft}}{1 + 1.20}(0.20)(\log 2660 - \log 2060)$$

$$= \frac{(8 \text{ ft})(0.20)}{2.20}[3.426 - 3.315] = 0.0806 \text{ ft} = 1 \text{ in. } (\pm)$$

118 Consolidation # 1b

If the groundwater table is at the soil surface, the soil overburden pressure σ_{v_0}, is due to the submerged, or effective, weight of the soil. If the unit weights of 135 pcf and 110 pcf represent saturated unit weights, the submerged unit weights will be

For the sand:

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = 135 \text{ pcf} - 62.4 \text{ pcf} = 72.6 \text{ pcf}$$

For the clay:

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = 110 \text{ pcf} - 62.4 \text{ pcf} = 47.6 \text{ pcf}$$

Consolidation # 1c

If the soils are not fully saturated, it is generally sufficiently accurate to assume that a submerged effective soil weight is about half its weight when not submerged. Using the values calculated, the overburden pressure σ_{v_0} is then

$$\sigma_{v_0} = (72.6 \text{ pcf} \times 12 \text{ ft}) + (47.6 \text{ pcf} \times 4 \text{ ft}) = 1062 \text{ psf}$$

and

$$\sigma_{v_f} = \sigma_{v_0} + \Delta\sigma_v = 1062 + 600 = 1662 \text{ psf}$$

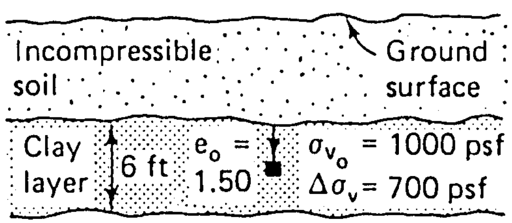
(Note that $\Delta\sigma_v$ is not affected by the water table or submergence.)

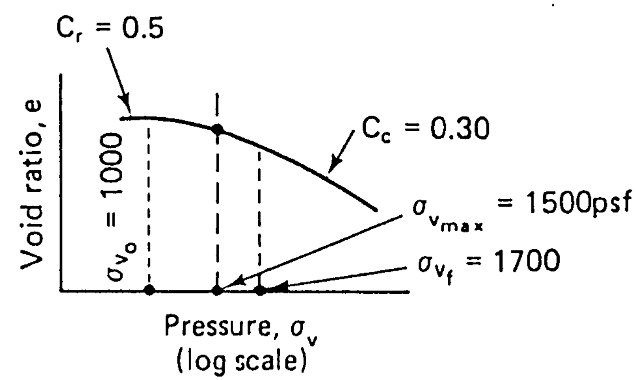
$$\Delta H = \frac{H_o}{1 + e_0} C_c[\log \sigma_{v_f} - \log \sigma_{v_0}]$$

$$= \frac{(8 \text{ ft})(0.20)}{2.20} [3.222 - 3.028] = 0.14 \text{ ft} = 1.7 \text{ in. } (\pm)$$

Thus, the effect of a high water table is to cause more settlement.

Illustration 9-2: Assume that a buried stratum of clay six feet thick will be subjected to a stress increase of 700 psf at the center of the layer. The magnitude of the preconstruction soil overburden pressure is 1000 psf at the center of the layer. A laboratory compression test indicates that the clay is overconsolidated, with $\sigma_{v_{max}}$ equal to 1500 psf. The value of C_c is 0.30, and the value of C_r is 0.05. What change in thickness results in the clay layer due to the stated conditions?





Consolidation # 2a

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The change in thickness of the clay layer, from Eq. 9-6 is

$$\Delta H = \frac{H_o}{1 + e_0}[C_r(\log \sigma_{v_{max}} - \log \sigma_{v_0}) + C_c(\log \sigma_{v_f} - \log \sigma_{v_{max}})]$$

$$= \frac{72 \text{ inches}}{2.50}[(0.05)(\log 1500 - \log 1000)$$

$$+ (0.30)(\log 1700 - \log 1500)]$$

$$= 0.73 \text{ in.}$$

Consolidation # 2b

Illustration 9-5: Referring to Illustration 9-1 and Fig. 9-12, assume that laboratory consolidation test data for the clay indicates that, for the range of loading applied to the soil, c_v is 0.2 ft^2 per month.

(a) How long will it take for half of the estimated settlement to occur?

Half of the estimated settlement is a U of 50 percent. Referring to Fig. 9-16, for U equals 50 percent, obtain the value $T_v = 0.20$.

$$t = \frac{T_v H_{dr}^2}{c_v} = \frac{(0.20)(8/2\ \text{ft})^2}{0.2\ \text{ft}^2/\text{month}} = 16\ \text{months}$$

[Note that for this type of problem, H_{dr} is one-half of the clay layer.]

Consolidation # 3a

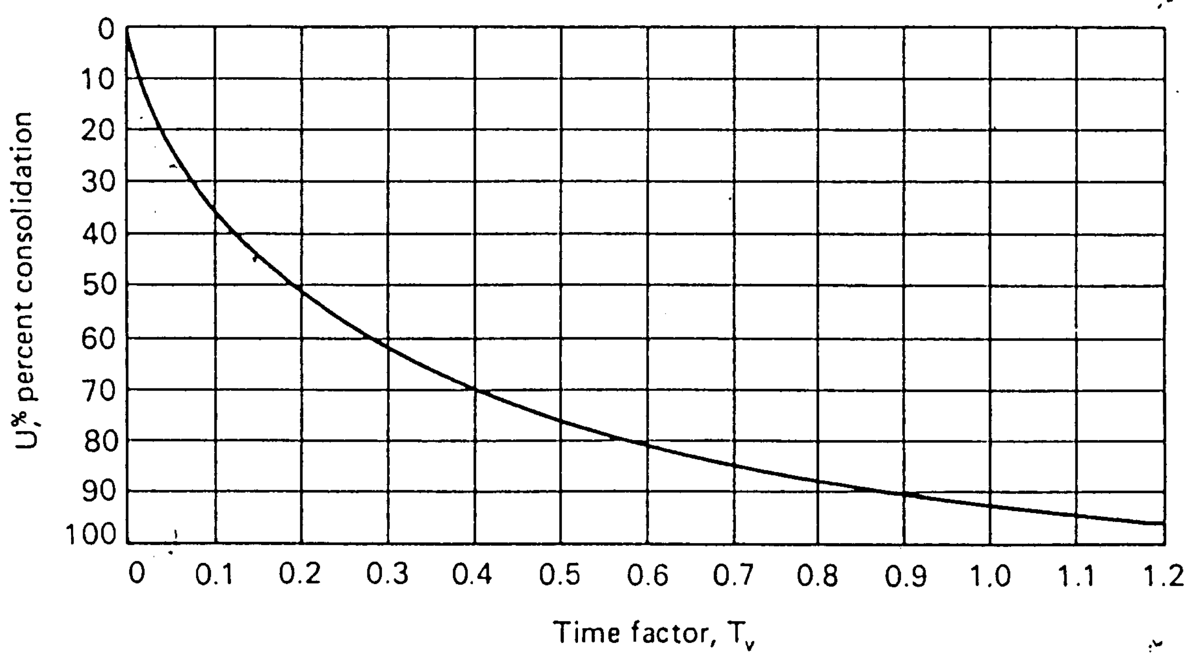

Figure 9-16. Variation of time factor T_v with percentage of consolidation *U*.

Consolidation # 3b

(b) How much settlement will occur in one year?

$$t = 1 \text{ year} = 12 \text{ months}$$

$$T_v = \frac{tc_v}{H_{dr}^2} = \frac{(12 \text{ months})(0.2 \text{ ft}^2/\text{month})}{(8/2 \text{ ft})^2} = 0.15$$

From Fig. 9-16, for $T_v = 0.15$ we get $U = 43$ percent, and therefore

$$\text{Settlement } \Delta H = 43 \text{ percent of 1 inch (from Illus. 9-1)}$$
$$= 0.43 \text{ in.}$$

Consolidation # 3c