CHAPTER 2
DETERMINATION OF PERMEABILITY OF SOIL AND CHEMICAL COMPOSITION OF WATER

2-1. Darcy's Law.

a. Development of Darcy's Law. Henry Darcy, a French engineer, conducted a laboratory experiment to study the flow of water in verticals and filters which he published in his 1856 treatise. The results of his experiment indicated that (Rouse and Ince 1957)

$$v = ki \tag{2-1}$$

or since $Q = vA$

$$Q = kiAt \tag{2-2}$$

or using $q = \frac{Q}{t}$

$$q = kiA \tag{2-3}$$

\where

v = discharge velocity

k = Darcy's coefficient of permeability[1]

i = hydraulic gradient (head loss/length over which head loss occurs)

Q = quantity of discharge

A = cross-sectional area of flow

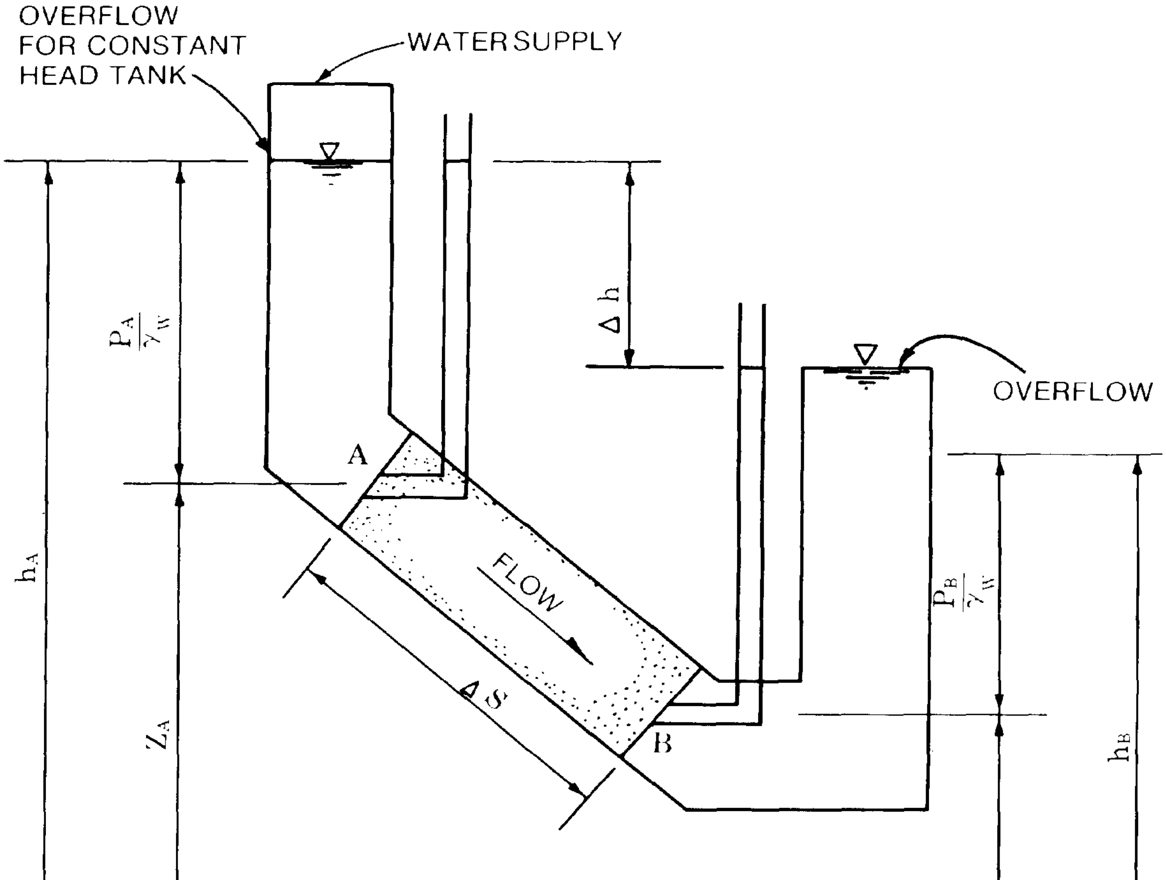
t = time of flow

q = rate of discharge

b. Extension to Inclined Soil Column. Darcy's law may be extended to flow through an inclined soil column given in figure 2-1 (Harr 1962). As indicated in equation 2-1, flow is a consequence of differences in total

(1) Commonly called the coefficient of permeability or the permeability.

head[1] and not of pressure gradients (Harr 1962 and Bear 1972). As shown in figure 2-1, flow is directed from point A to point B even though the pressure at point B is greater than that at point A.



POINT	HEAD: TOTAL	PRESSURE	ELEVATION
A	h_A	P_A/γ_w	Z_A
B	h_B	P_B/γ_w	Z_B

Figure 2-1. Darcy's law for flow through inclined soil column (prepared by WES)

(1) The elevation head at any point is the distance from some arbitrary datum. The pressure head is the water pressure divided by the unit weight of the water. The total head is the sum of the elevation head and the pressure head.

c. Discharge Velocity and Seepage Velocity. The discharge velocity is defined as the quantity of fluid that flows through a unit cross-sectional area of the soil oriented at a right angle to the direction of flow in a unit time. The discharge velocity is used in determining the quantity of flow or rate of discharge through a soil. As flow can occur only through the interconnected pores of the soil, as shown in figure 2-2, the actual rate of

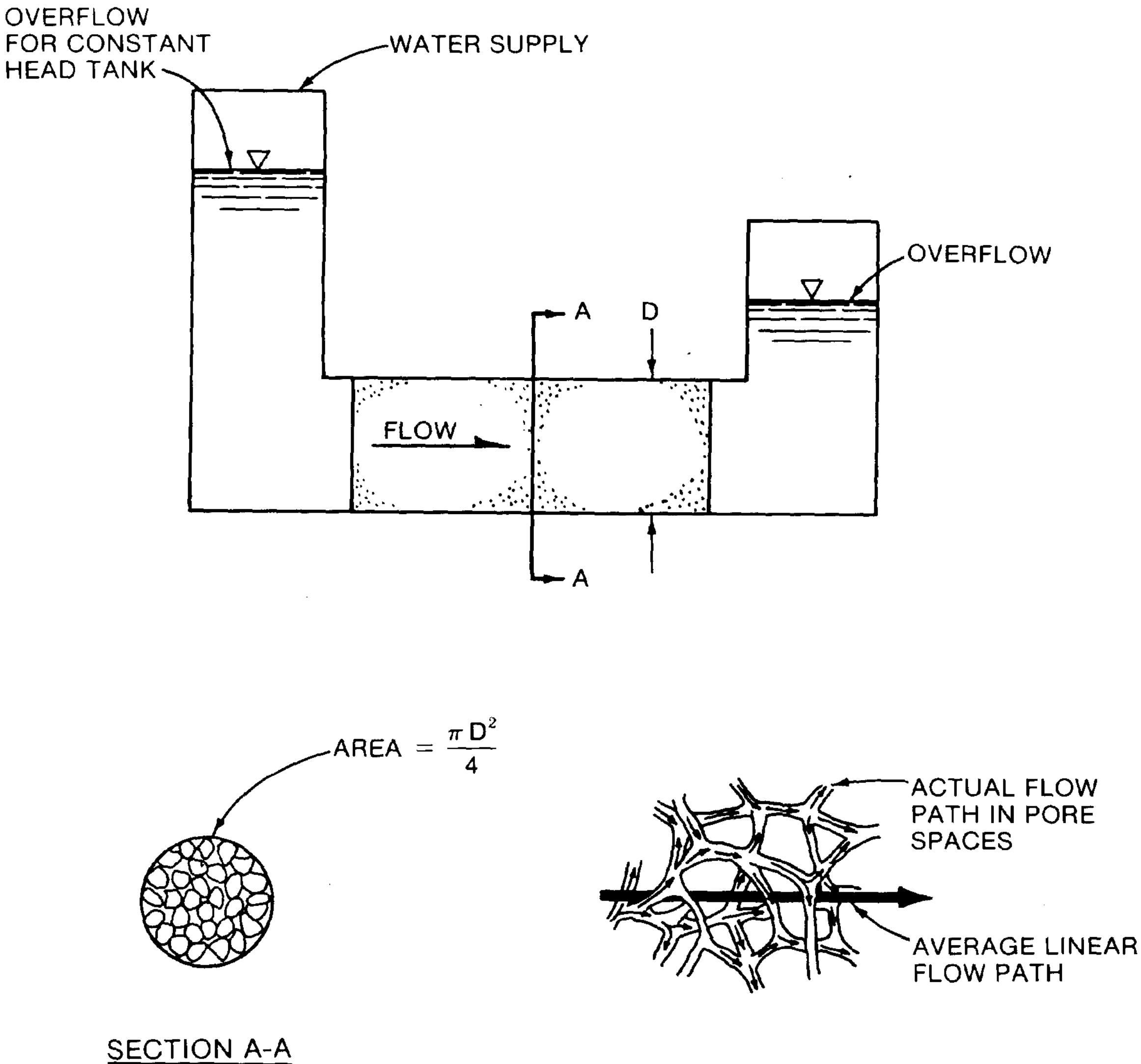


Figure 2-2. Concepts of flow paths through a soil column (prepared by WES)

movement of the water, as measured with dye tracers for instance, is the seepage velocity (Harr 1962 and Casagrande 1937) which exceeds the discharge velocity.

$$\bar{v} = \frac{v}{n} \tag{2-4}$$

where

$\bar{v}$ = seepage velocity

n = porosity (ratio of volume of voids to the total volume of the soil mass)

It follows that

$$k = \frac{\bar{v}n}{i} \qquad (2\text{-}5)$$

Equation 2-5 is a useful expression in estimating field permeabilities using dye tracers (Soil Conservation Service 1978).

2-2. Range of Validity of Darcy's Law.

a. Lower Bound. Darcy's law (equations 2-1 through 2-3) applies to linear flow (adjacent flow lines are locally straight and parallel). For flows through soils, there are two situations where the validity of this linear relationship may not hold. For highly plastic clays of low permeability, there may be a threshold hydraulic gradient below which flow does not take place. Such conditions may occur in deeply buried clays and clay shales. For many practical seepage problems the rate of flow through these soil layers is so small that they can be considered to be impervious (Mitchell 1976, Chugaev 1971, Basak and Madhav 1979, and Muskat 1946).

b. Upper Bound. Of greater practical importance is the upper limit on the range of validity of Darcy's law. It has been recognized that, at very high flow rates, Darcy's law does not hold (Chugaev 1971). The upper limit is usually identified using Reynolds number, a dimensionless number that expresses the ratio of internal to viscous forces during flow. It is often used in fluid mechanics to distinguish between laminar flow (fluid layer flows alongside of another at approximately the same velocity with no macroscopic mixing of fluid particles) at low velocities and turbulent flow (velocity fluctuations, both parallel and transverse, are imposed upon the mean motion with mixing of the fluid particles) at high velocities. The Reynolds number for flow through soils is

$$\Re = \frac{vD\rho}{\mu} \qquad (2\text{-}6)$$

where

$\Re$ = Reynolds number

D = average diameter of soil particles

ρ = density of fluid

μ = coefficient of dynamic viscosity of fluid

The critical value-of Reynolds number at which the flow in soils changes from laminar to turbulent has been determined experimentally by various investigators to range from 1 to 12 (Harr 1962 and Chugaev 1971). Assuming a water temperature of 20° C, substituting $\rho = 998.2\ kg/m^3$ and $\mu = 1.002 \times 10^{-3}$ kg/m sec into equation 2-6, and assuming values of D and solving for v with $\Re = 1$ and $\Re = 12$ gives the relationship shown in figure 2-3 which defines the upper bound of the validity of Darcy's law. Depending on the discharge velocity, Darcy's law is generally applicable for silts through medium sands.

c. Turbulent Flow.

(1) Estimating Permeability from Empirical Equation. For flow through soils more pervious than medium sands, flow is likely to be turbulent. Under turbulent conditions, the seepage velocity in a material with monosized soil particles (coarse sands and/or gravels) can be estimated from the following equation (Wilkins 1956, Leps 1973, and Stephenson 1979).

$$\bar{v} = WM^{0.5} i^{0.54} \tag{2-7}$$

where

$\bar{v}$ = seepage velocity in inches per second

W = an empirical constant, which depends on the shape and roughness of the soil particles and viscosity of water and varies from 33 for crushed gravel to 46 for polished marbles, in inch$^{1/2}$ per second

M = hydraulic mean radius of the rock voids (for a given volume of particles equal to the volume of voids divided by the total surface area of the particles, or the void ratio divided by the surface area per unit volume of solids) in inches

i = hydraulic gradient

The coefficient of permeability is obtained from the seepage velocity using equation 2-5. For well-graded soils, the D_{50} size (50 percent finer by weight) can be used to calculate the hydraulic mean radius provided that the minus 1-in.-size material is less than 30 percent by weight. If there is more than 30 percent of minus 1-in.-size material, the permeability should be determined experimentally (Leps 1973).

(2) Determining Permeability Experimentally. Alternatively, for flow through soils more pervious than medium sands, the relationship between hydraulic gradient and discharge velocity can be determined experimentally (Cedergren 1977).

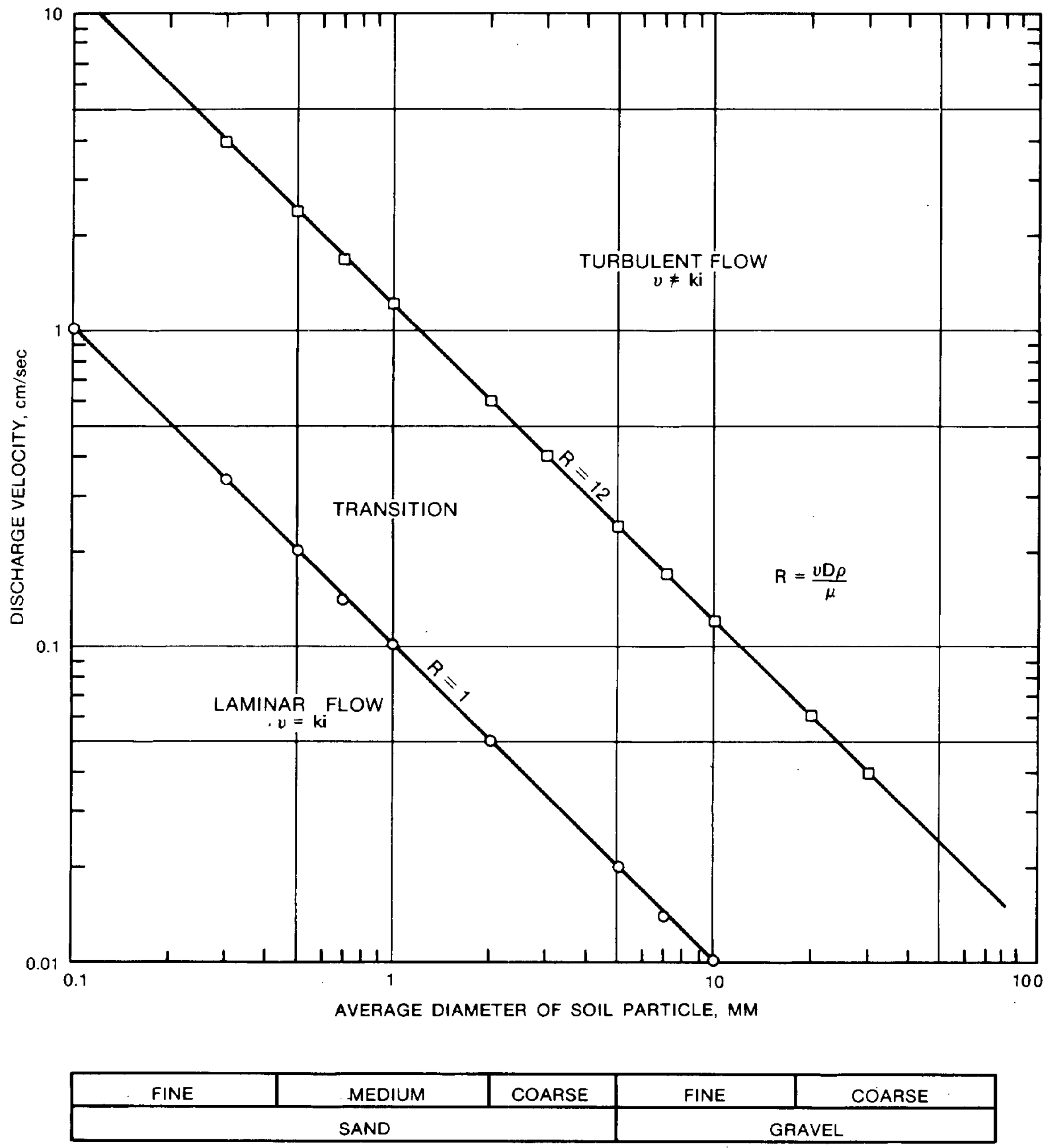


Figure 2-3. Boundary between laminar and turbulent flow determined using Reynolds number for temperature of 20° C (prepared by WES)

2-3. Coefficient of Permeability.

a. Darcy's (Engineer's) Coefficient of Permeability. The coefficient of permeability used in seepage analysis for dams is called the Darcy's or engineer's coefficient and is given by (Cedergren 1977)

$$k = \frac{v}{i} \tag{2-8}$$

or since $Q = vA$

$$k = \frac{Q}{iAt} \tag{2-9}$$

or using $q = \frac{Q}{t}$

$$k = \frac{q}{iA} \tag{2-10}$$

The coefficient of permeability is defined as the rate of discharge-of water at a temperature of 20° C under conditions of laminar flow through a unit cross-sectional area of a saturated soil medium. The coefficient of permeability has the dimensions of a velocity and is usually expressed in centimeters per second. Permeability computed on the basis of Darcy's law is limited to the conditions of laminar flow and complete saturation of the soil. Under conditions of partial saturation, the flow is in a transient state and is time dependent. To analyze natural flow conditions which depart from the Darcy flow condition, it is sometimes necessary to apply Darcy's law in conditions where it is not strictly valid. When this is done, the effects of turbulent flow and partial saturation on the permeability must be recognized and taken into consideration (Cedergren 1975).

b. <u>Intrinsic (Specific) Permeability</u>. The coefficient of permeability of a soil material varies for different pore fluids depending upon their density and viscosity as follows:

$$k = k_o \frac{\gamma}{\mu} \tag{2-11}$$

where

k_o = intrinsic permeability

γ = unit weight of pore fluid

μ = viscosity of pore fluid

The intrinsic permeability has the dimensions of length squared and is expressed in square centimeters or Darcy's (equal to 1.01 x 10^{-8} cm^2). Figure 2-4 is a chart for the conversion of permeability values from one set of

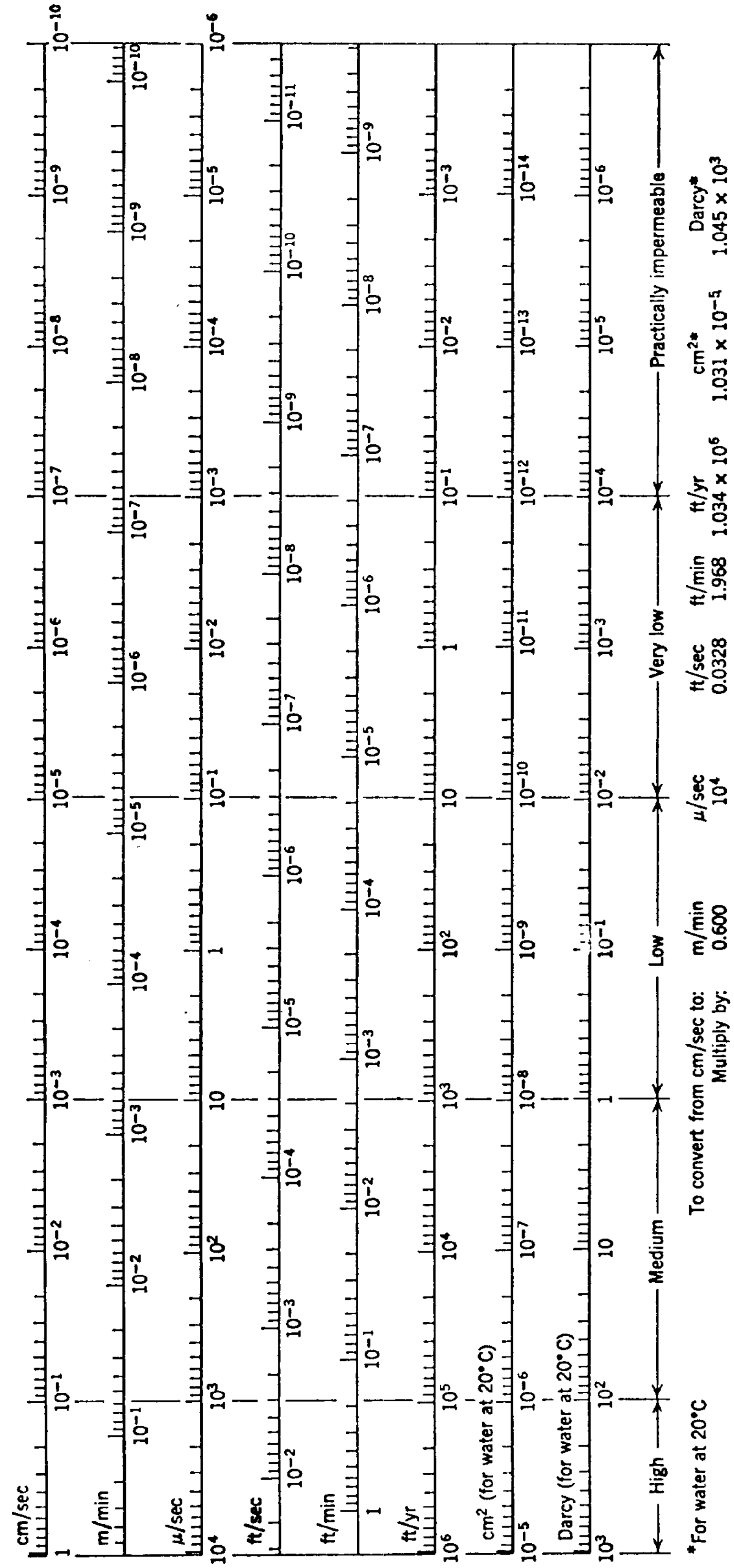


Figure 2-4. Permeability conversion chart (courtesy of John Wiley & Sons, Inc.[201])

units to another (Lohman et al. 1972). Substituting equation 2-11 into equation 2-1

$$v = k_o \frac{\gamma}{\mu} i \tag{2-12}$$

Equation 2-12 may be used when dealing with more than one fluid or with temperature variations. This is widely used in the petroleum industry where the presence of gas, oil, and water occur in multiphase flow systems (Freeze and Cherry 1979, and Bureau of Reclamation 1977). In seepage analysis for earth dams where we are primarily interested in the flow of water subject to small changes in temperature, this refinement is seldom required.

c. Transmissivity Factor. In order to describe the flow characteristics of an aquifer (saturated permeable geologic unit that can transmit significant quantities of water under ordinary hydraulic gradients), C. V. Thesis introduced the term transmissivity which is defined as (Bureau of Reclamation 1977)

$$T = kt \tag{2-13}$$

where

T = transmissivity factor

k = average permeability

t = aquifer thickness

Transmissivity represents the rate of discharge for a gradient of unity through a vertical strip of aquifer one unit wide and has dimensions of length squared per unit time and is usually expressed in square feet per day.

2-4. Factors Influencing Permeability.

a. Range of Values of Permeability. No other property of soil exhibits a wider range of values (up to ten orders of magnitude) or shows greater directional (anisotropy) and spatial variability in a given deposit as does the coefficient of permeability. The approximate range in coefficients of permeability for soils and rocks is shown in figure 2-5 (Milligan 1976). Within the range, extreme variations of permeability in situ are possible due to the degree of stratification or heterogeneity of the soil deposit.

b. Variation of In Situ Permeability. Natural soil deposits are generally stratified in structure. Water-deposited soils are laid down in 'horizontal layers and are often more permeable in the horizontal than vertical direction. Windblown sands and silts are generally more permeable vertically than horizontally due to the presence of continuous vertical root holes. An important example of stratification is openwork gravel which may occur in

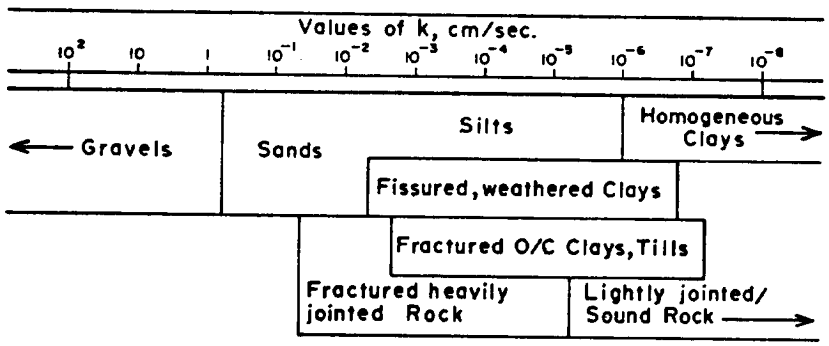
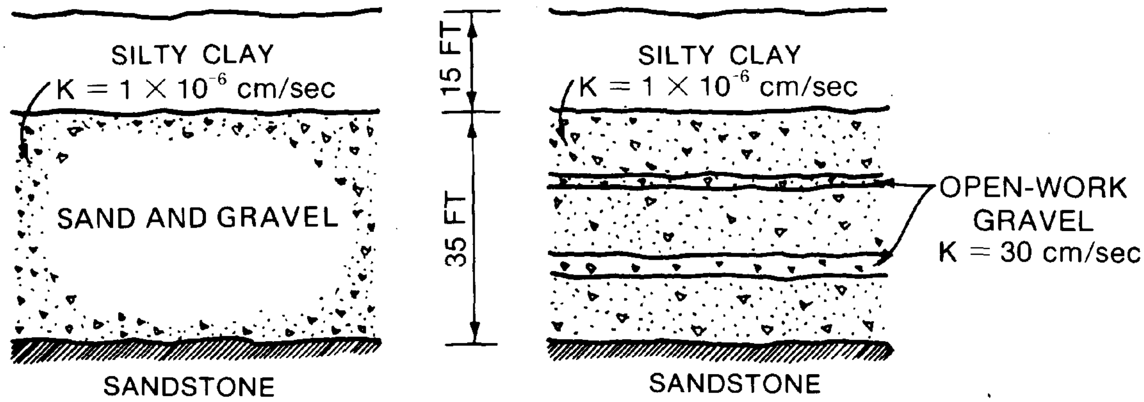


Figure 2-5. Approximate range in coefficient of permeability of soils and rocks (from Milligan[224])

ordinary gravel or soil and have tremendous influence on the watertightness of dam foundations and abutments as shown in figure 2-6 (Cedergren 1977). Figure 2-6a shows a soil profile surmised from several drill holes. The grain size analysis of soil samples taken at frequent intervals erroneously indicated that the deposit was composed of relatively uniform sandy gravels. Laboratory permeability tests on disturbed samples produced coefficients of permeability of about 1×10^{-6} cm/sec. Using this value of permeability, the probable seepage loss beneath the proposed dam was estimated to be 3 cu ft/day, which is an insignificant quantity. However, the design engineer had observed many openwork streaks in which the fines fraction of the material was almost completely absent along the banks of the river and noted that the ground-water table was level for several hundred feet away from the river and fluctuated rapidly with changes in river stage. Field pumping tests were conducted which indicated somewhat variable permeabilities but none approaching the magnitude of openwork gravels. Based upon the available data, the dam was designed with a cutoff trench to bedrock. During the excavation of the cutoff trench, streaks of openwork gravel were found throughout the foundation. A revised seepage computation based on a permeability of 30 cm/sec indicated that without the cutoff trench, the theoretical underseepage would be about 1,000,000 cu ft/day. If openwork gravel or other important discontinuities in earth dam foundations remain undetected, serious problems from excessive seepage and hydrostatic pressures will develop. This example illustrates the potential serious effects of deviations between the design assumptions and the as-built dam (Cedergren 1977). Also, thin continuous seams of cohesive soil can drastically alter the vertical flow through what would otherwise be a highly permeable site.



a. Soil profile surmised from drill holes with estimated quantity of seepage under dam equal to 3 cu ft/day

b. Soil profile revealed by cutoff trench with estimated quantity of seepage under dam equal to 1,000,000 cu ft/day (without cutoff trench)

Figure 2-6. Influence of openwork gravel on underseepage (courtesy of John Wiley & Sons, Inc. [155])

c. Properties of the Seepage Fluid. The properties of the seepage fluid which influence the permeability of soils are the temperature, density, viscosity, and chemical composition.

(1) As shown in table 2-1, for the range of temperatures ordinarily encountered in seepage analysis of dams (0° C to 40° C) the density of water is nearly constant (varies less than 1 percent).

Table 2-1. Properties of Water[a]

Temperature °C	Temperature °F	Density kg/m^3	Viscosity kg/m sec
0	32	999.8	1.787×10^{-3}
5	41	999.9	1.519
10	50	999.7	1.307
15	59	999.1	1.139
20	68	998.2	1.002
25	77	997.0	0.890
30	86	995.6	0.798
35	95	994.0	0.719
40	104	992.2	0.653

[a]Prepared by WES.

(2) The viscosity varies up to 63.5 percent over the range of temperatures ordinarily encountered in seepage analysis of dams. As indicated in equation 2-11, the permeability is inversely proportional to the viscosity of the water. As given in table 2-1, the viscosity of water decreases as temperature increases. Therefore, the coefficient of permeability of the soil increases as the temperature of the water increases. Permeability tests are run at the most convenient temperature and reported at 20° C.

(3) The total dissolved salts (TDS) present in the seepage water may influence the permeability of the soil, particularly for cohesive soils (Quirk and Schofield 1955 and Cary, Walter, and Harstad 1943). Available data indicate that cohesive soils may be two to three orders of magnitude more permeable to seepage water containing moderate amounts of dissolved salts (less than 300 parts per million by weight) than the distilled water (Carry, Walter, and Harstad 1943).

d. Degree of Saturation. The degree of saturation of a soil

$$S = \frac{V_w}{V_v} \times 100 \text{ percent} \tag{2-14}$$

where

S = degree of saturation

V_w = volume of water

V_v = volume of voids

has an important influence on permeability. A decrease in the degree of saturation causes a decrease in the permeability as shown in figure 2-7 (Lambe 1951). When the degree of saturation is less than 85 percent, much of the air would be continuous throughout the soil voids and Darcy's law would not hold. When the degree of saturation is greater than 85 percent, most of air present in the soil is in the form of small occluded bubbles and Darcy's law will be approximately valid. The ratio of the permeability of the unsaturated sand to the saturated sand at the same void ratio is given as (Scott 1963 and Parker and Thornton 1976)

$$\frac{k_{us}}{k} = 1 - m\left(1 - \frac{S}{100}\right) \quad 100 \geq S \geq 80 \tag{2-15}$$

where

k_{us} = unsaturated permeability

m = constant with values between 2 (uniform grain size) and 4 (well-graded materials)

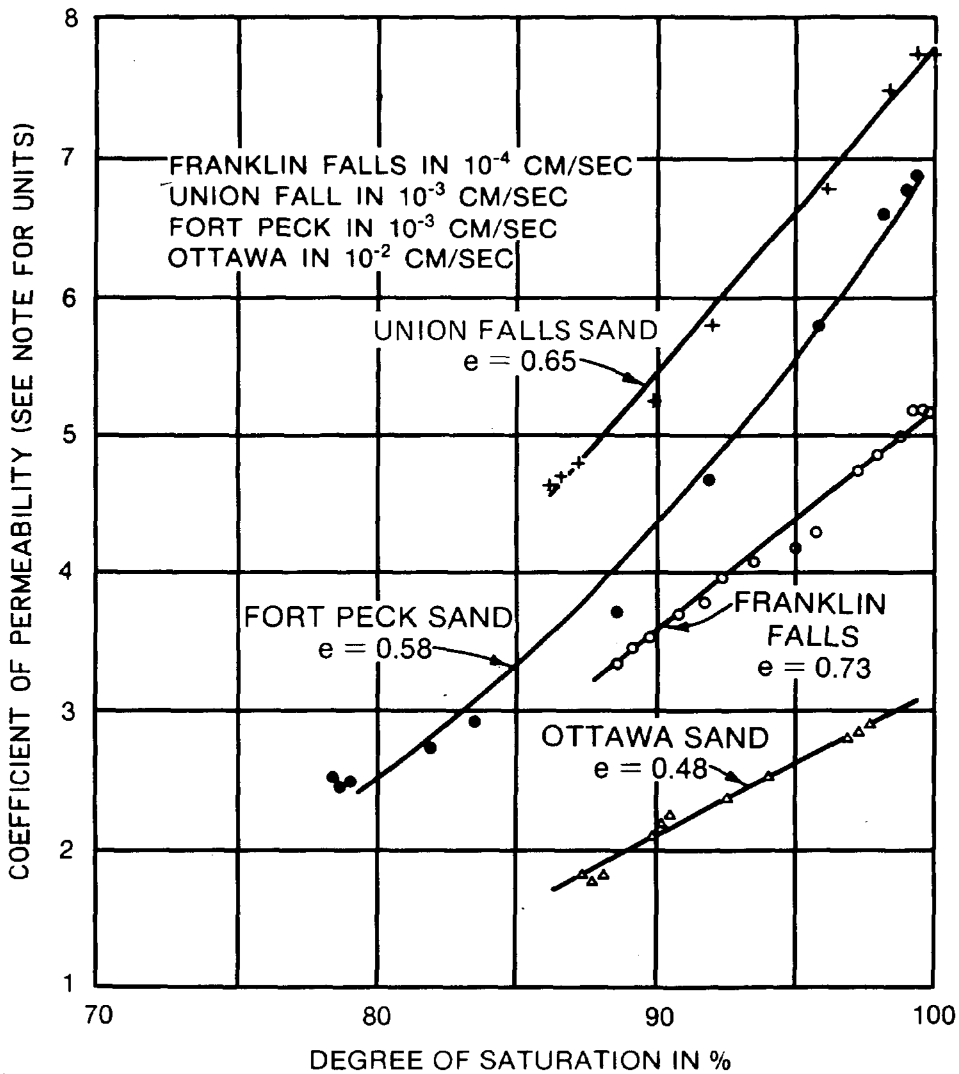


Figure 2-7. Permeability versus degree of saturation for various sands (courtesy of John Wiley & Sons, Inc.[200])

e. Hydraulic Gradient. The hydraulic gradient

$$i = \frac{H}{L} \tag{2-16}$$

where

H = head loss

L = length over which head loss occurs

at which permeability is measured can have a significant influence on the coefficient of permeability computed from Darcy's law under certain conditions. The maximum hydraulic gradient for which laminar flow occurs for a particular soil at a given density may be determined in the laboratory by plotting the discharge velocity

$$v = \frac{Q}{At} \tag{2-17}$$

versus the hydraulic gradient as shown in figure 2-8. A straight line relationship indicates laminar flow

$$k = \frac{v}{i} \tag{2-18}$$

while deviations from the straight line at high gradients indicate turbulent flow. Darcy's law for a fine sand, as shown in figure 2-8, is valid only for the hydraulic gradient less than 2 for the loose state and 4.5 for the dense state. For soils larger than a fine sand, Darcy's law is valid for progressively smaller hydraulic gradients (Burmister 1948 and Burmister 1955).

f. Particle Size. For cohesive soils, the permeability increases with increases in clay mineral size and increase in void ratio (ratio of the volume of voids to the volume of solid particles in the soil mass) as shown in figure 2-9 (Yong and Warkentin 1966).(1) For cohesionless soils, the size and shape of the soil particles influence the permeability. Allan Hazan conducted tests on filter sands for use in waterworks and found that for uniform loose clean sands the permeability was given by (Taylor 1948)

$$k = 100\ D_{10}^2 \tag{2-19}$$

where

k - coefficient of permeability in cm per second

D_{10} = particle size in cm at which 10 percent of the material is finer by weight (also known as Hazen's effective size)

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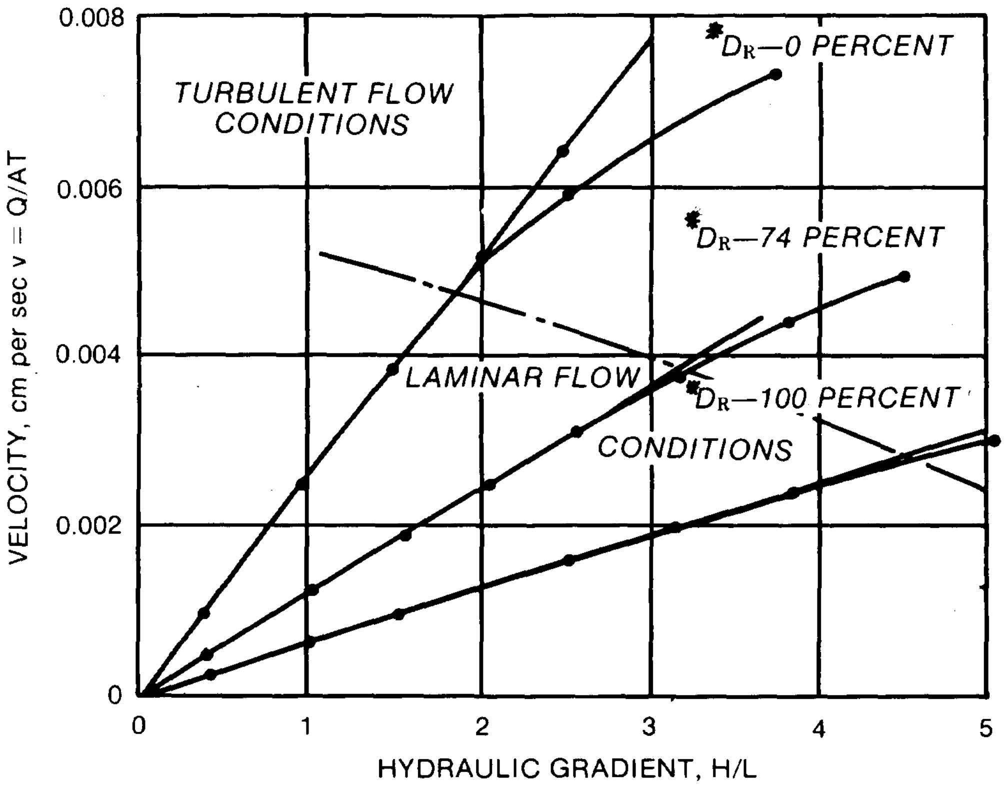
(1) As shown in table 2-2, the exchangeable cation present influences the permeability of clay minerals at constant void ratio (Scott 1963). The permeabilities are much smaller when the exchangeable cation is sodium which is one of the reasons why sodium montmorillonite is used to seal reservoirs.



$$* \quad D_R = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

Figure 2-8. Determination of maximum hydraulic gradient for which laminar flow occurs for a fine sand (courtesy of American Society for Testing and Materials[147])

Hazen's experiments were made on sands for which 0.1 mm $\leq D_{10} \leq$ 0.3 mm and the uniformity coefficient, $C_u < 5$, where

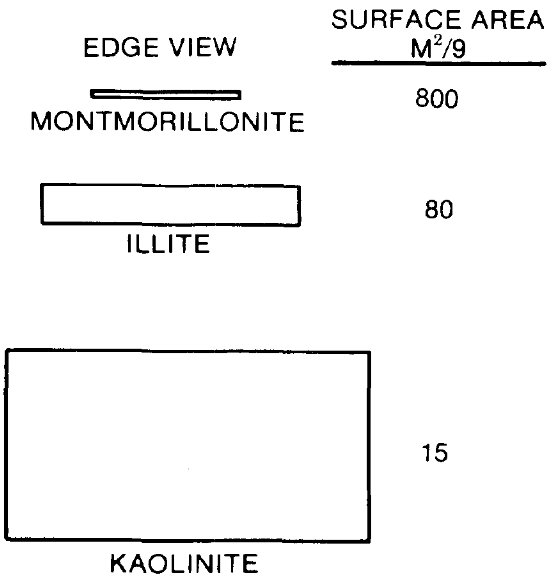
$$C_u = \frac{D_{60}}{D_{10}} \tag{2-20}$$

where

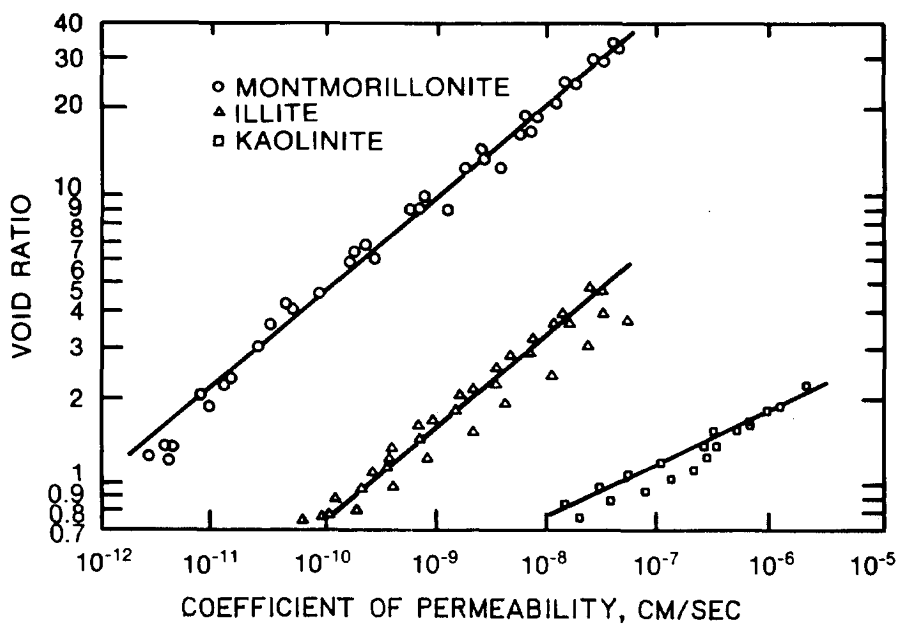
C_u = uniformity coefficient

D_{60} = particle size at which 60 percent of the material is finer by weight

The coefficient 100 is an average of many values which ranged from 41 to 146, but most of the values were from 81 to 117. Equation 2-19 makes no allowance for variations in shape of the soil particles or void ratio.



a. Edge view sketch to show relative size and shape of clay particles (dimension not shown in length)



b. Permeability versus void ratio for various clay minerals

Figure 2-9. Influence of particle size and void ratio on permeability of clay minerals (courtesy of Macmillan[294])

Table 2-2. Coefficients of Permeability for Different Exchange Cations and Void Ratios for Two Clay Minerals[a]

Clay Mineral	Exchangeable Cation	Void Ratio	Coefficient of Permeability cm/sec
Montmorillonite	Na	15	8×10^{-8}
		3	1.5×10^{-8}
	K	11	5×10^{-8}
		7	8×10^{-9}
	Ca	8	1×10^{-5}
		4	1×10^{-7}
	H	9	2×10^{-6}
		3	1×10^{-7}
Kaolinite	Na	1.6-0.5	1.5×10^{-6} to 8×10^{-7}
	K	1.6-1.1	3×10^{-6} to 9×10^{-7}
	Ca	1.6-1.3	1×10^{-5} to 1.5×10^{-6}
	H	1.4-1.0	1×10^{-5} to 1.5×10^{-6}

Permeabilities are obtained by falling-head test on samples in consolidation apparatus. Results indicate the following:

For montmorillonite at void ratio 8 the order of permeability in terms of the exchangeable ion present is

$$K < Na < H < Ca$$

for kaolinite at void ratio 1.5 the order is

$$Na < K < Ca < H$$

For compacted soils it is also observed that the permeability is much lower ($\times 10^{-1}$ to 10^{-2}) in soils compacted slightly wet of optimum than in soils compacted dry of optimum; it is thought that this occurs because of the parallel arrangement of clay platelets in the wetter material after compaction.

[a]Courtesy of Addison-Wesley Publishing Company, Inc.[251]

g. Particle Shape and Surface Roughness. Cohesionless soil particles have different particle shapes and surface roughness dependent on the distance they have been transported by flowing water from the place of original erosion. As shown in table 2-3, the measured permeability is several orders of magnitude lower for angular sand particles with rough surfaces than for rounded sand particles with smooth surfaces (Burmister 1948). For uniform cohesionless soils, crushing of particles during compaction with resulting decrease in permeability occurs to a higher degree in soils with angular shapes and rough surfaces than in soils with rounded shapes and smooth surfaces. Crushing of particles during compaction leads to an increase in the amount of silt-sized particles (smaller than No. 200 sieve or 0.074 mm) which results in lower permeability. For this and other reasons (cementation in limestones and arching due to particle angularity) crushed rock is generally not used for filters in earth dams. Also, table 2-3 compares the measured permeability with the permeability computed from equation 2-19 developed by Hazen for uniform loose clean sands. The agreement between measured and computed permeability is within one order of magnitude for uniform sands and glass spheres. Therefore, Hazen's equation should be used only for uniform sands (sphericity and roundness $\geq$ 0.90). The sphericity and roundness may be estimated for sands using figure 2-10 (Krumbein and Sloss 1951).

h. Void Ratio. The permeability increases as the void ratio increases.

$$e = \frac{V_v}{V_s} \tag{2-21}$$

where

e = void ratio

V_s = volume of solids

There are considerable laboratory test data, shown in figure 2-11, which indicate that a plot of void ratio versus log of coefficient of permeability is frequently a straight line (Lambe and Whitman 1969).

i. Amount and Type of Fines Present. The permeability of sands and gravels varies significantly with the amount and type of fines (material smaller than the No. 200 sieve) (Barber and Sawyer 1952; Fenn 1966; Younger and Lim 1972; Strohm, Nettles, and Calhoun 1967; Nettles and Calhoun 1967, and Loudon 1952). As shown in figure 2-12a, the addition of 2.5 percent, by dry weight, silt fines to concrete sand results in an order of magnitude decrease in permeability (Barber and Sawyer 1952). The addition of 6.5 percent silt fines to concrete sand decreases the permeability two orders of magnitude. Similar results are obtained by the addition of somewhat larger amounts of clay and limestone fines to concrete sand. As shown in figure 2-12b, the addition of 2.0 percent silt fines to a sand-gravel mixture results in an order of magnitude decrease in permeability (Barber and Sawyer 1952). The addition of 4.2 percent silt fines to sand-gravel mixture decreases the permeability two orders of magnitude. Similar results are obtained by the addition of

Table 2-3. Influence of Particle Shape and Surface Roughness on Permeability of Sand(1)

Type of Material	Surface Roughness	Sphericity	Roundness	C_u(2)	C_c(3)	Void Ratio	Relative Density percent	Coefficient of Permeability cm/sec: Measured	Coefficient of Permeability cm/sec: Computed-Hazen(4)
Crushed Sudeten granite	Very rough	0.68	0.10	3.09	1.54	0.543	79	5.00×10^{-5}	1.21×10^{-2}
Nysd Klodzkd River sand	Very rough	0.75	0.15	5.47	2.20	0.451	79	5.19×10^{-6}	5.63×10^{-3}
Odra River sand	Rough to smooth	0.84	0.55	2.58	0.87	0.595	58	9.41×10^{-4}	1.44×10^{-2}
Odra River sand	Rough to smooth	0.87	0.65	2.42	0.83	0.608	63	5.44×10^{-3}	1.44×10^{-2}
Baltic Beach sand	Smooth	0.87	0.85	1.91	0.89	0.720	38	1.83×10^{-3}	1.32×10^{-2}
Glass spheres	Very smooth	1.00	1.00	1.38	0.92	0.691	67	6.45×10^{-3}	6.40×10^{-3}

(1) Courtesy of British Geotechnical Society[146].

(2) $$C_u = \frac{D_{60}}{D_{10}} .$$

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

(4) $k = 100\ D_{10}$.

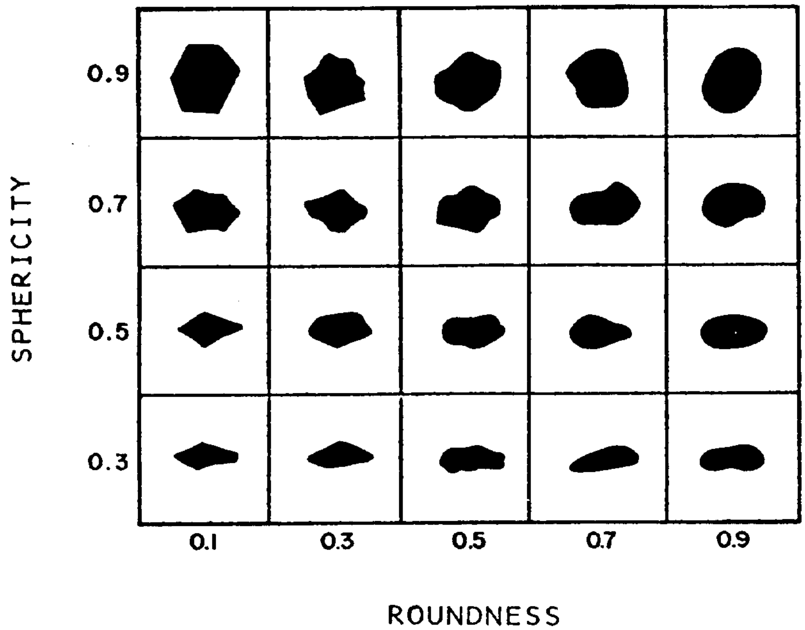


Figure 2-10. Krumbein and Sloss standard chart for visual estimation of sphericity and roundness of cohesionless soils (courtesy of W. H. Freeman and Company[198])

somewhat smaller amounts of clay and larger amounts of limestone, respectively, to a sand-gravel mixture. As shown in figure 2-12c, the addition of about 1 percent calcium montmorillonite fines to a uniform fine sand results in an order of magnitude decrease in permeability, while over 10 percent kaolinite fines would be required for a similar reduction in permeability (Fenn 1966).

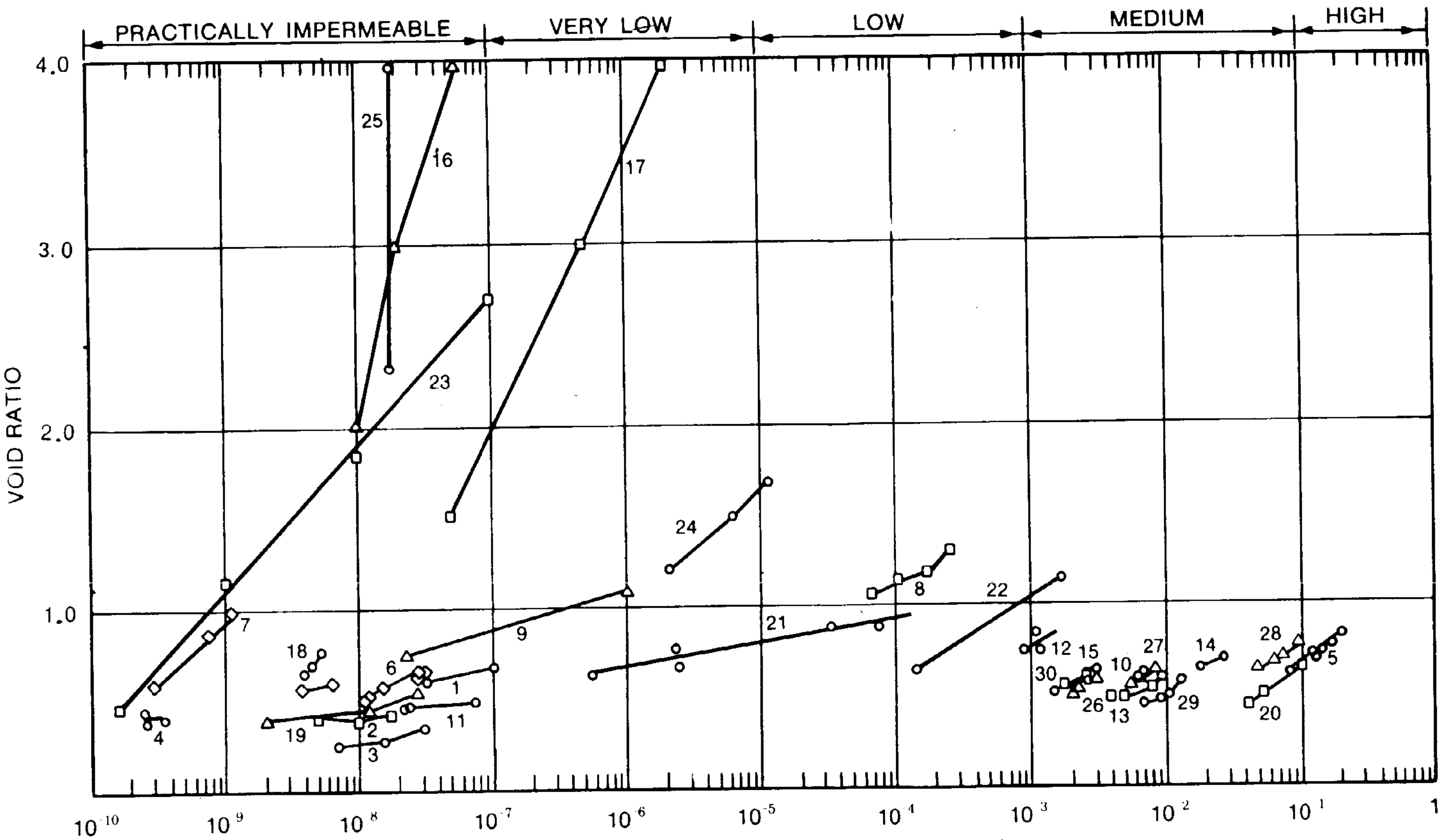
j. Summary of Factors Influencing Permeability. The significant influence that various factors exert on the permeability emphasizes the importance of duplicating field conditions when determining permeability in the laboratory.

2-5. Indirect Methods for Determining Permeability.

a. Hazen's Equation. For uniform loose clean sands, classified SP in the Unified Soil Classification System (U. S. Army Engineer Waterways Experiment Station 1960), the permeability may be estimated from the previously given Hazen's equation (Taylor 1948)

$$k = 100\, D_{10}^2 \qquad (2\text{-}22)$$

where k is in cm per second and D_{10} is in cm.

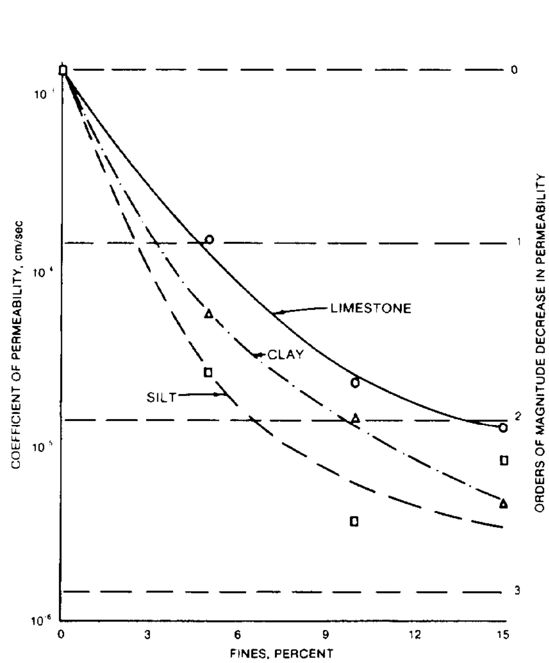


SOIL IDENTIFICATION CODE

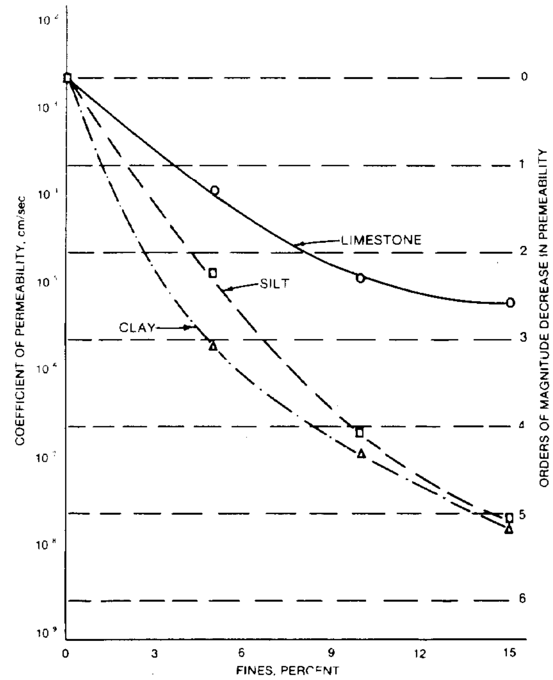
1. COMPACTED CALICHE
2. COMPACTED CALICHE
3. SILTY SAND
4. SANDY CLAY
5. BEACH SAND
6. COMPACTED BOSTON BLUE CLAY
7. VICKSBURG BUCKSHOT CLAY
8. SANDY CLAY
9. SILT—BOSTON
10. OTTAWA SAND
11. SAND—GASPEE POINT
12. SAND—FRANKLIN FALLS
13. SAND—SCITUATE
14. SAND—PLUM ISLAND
15. SAND—FORT PECK
16. SILT—BOSTON
17. SILT—BOSTON
18. LOESS
19. LEAN CLAY
20. SAND—UNION FALLS
21. SILT—NORTH CAROLINA
22. SAND FROM DIKE
23. SODIUM—BOSTON BLUE CLAY
24. CALCIUM KAOLINITE
25. SODIUM MONTMORILLONITE

26.-30. SAND DAM FILTER

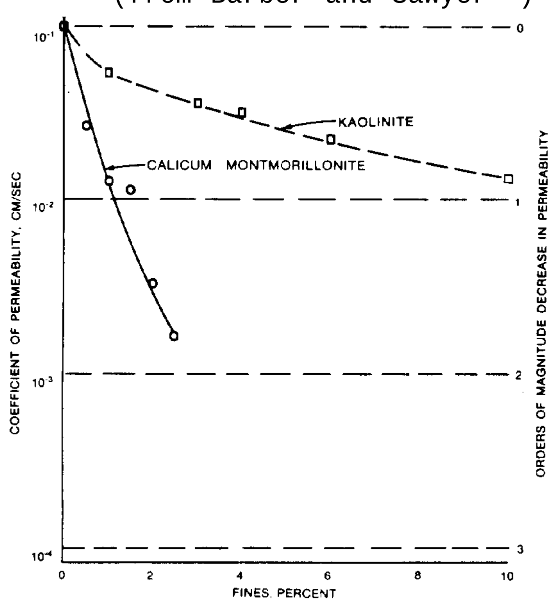
Figure 2-11. Relationship between void ratio and log of permeability (courtesy of Wiley[201])



a. Effect of fines on permeability of concrete sand (from Barber and Sawyer[137])



b. Effect of fines on permeability of sand-gravel mixture (from Barber and Sawyer[137])



c. Effect of fines on permeability of uniform fine sand (from Fenn[171])

Figure 2-12. Influence of type and amount of fines on permeability of concrete sand, sand-gravel mixture, and uniform fine sand (prepared by WES)

b. Masch and Denny Method. For uniform or nonuniform dense clean sands, classified SP or SW in the Unified Soil Classification System (U. S. Army Engineer Waterways Experiment Station 1960), the permeability may be estimated from an empirical method developed by Masch and Denny 1966, and Denny 1965. The gradation curve is plotted in Krumbein 4 units (Krumbein and Pettijohn 1938) (using the chart in figure 2-13) as shown in figure 2-14 where

$$\phi = -\log_2 d = -\frac{\log_{10} d}{\log_{10} 2} = -3.322 \log_{10} d \qquad (2\text{-}23)$$

where

ϕ = phi scale units used to describe grain size distribution

d = grain size diameter in mm

The inclusive standard deviation is used as a measure of the spread of the gradation curve where (Masch and Denny 1966)

$$\sigma_I = \frac{d_{16} - d_{84}}{4} + \frac{d_5 - d_{95}}{6.6} \qquad (2\text{-}24)$$

where

σ_I = inclusive standard deviation

d_{16} = grain size in ϕ units at which 16 percent is finer

d_{84} = grain size in ϕ units at which 84 percent is finer

d_5 = grain size in ϕ units at which 5 percent is finer

d_{95} = grain size in ϕ units at which 95 percent is finer

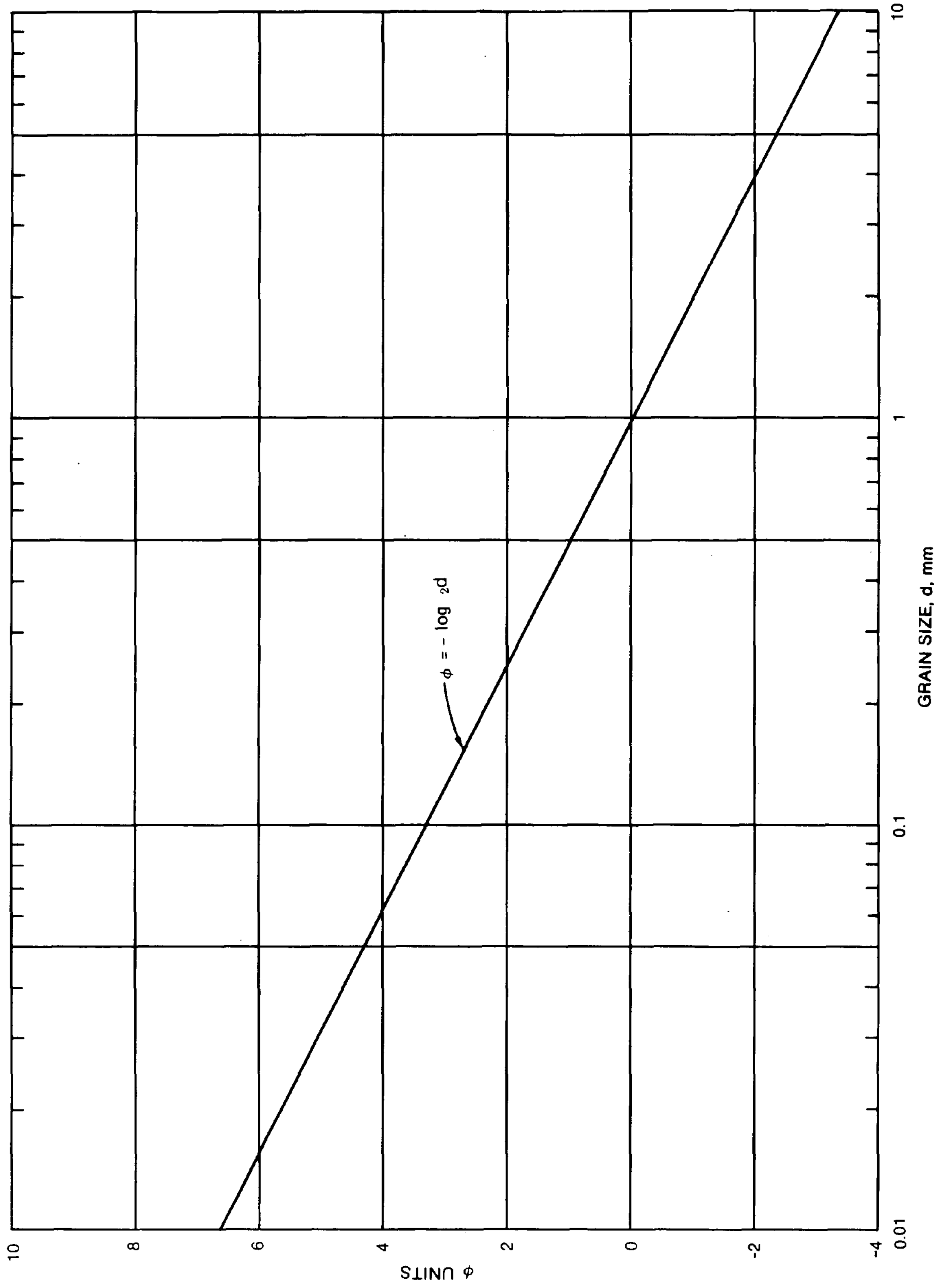
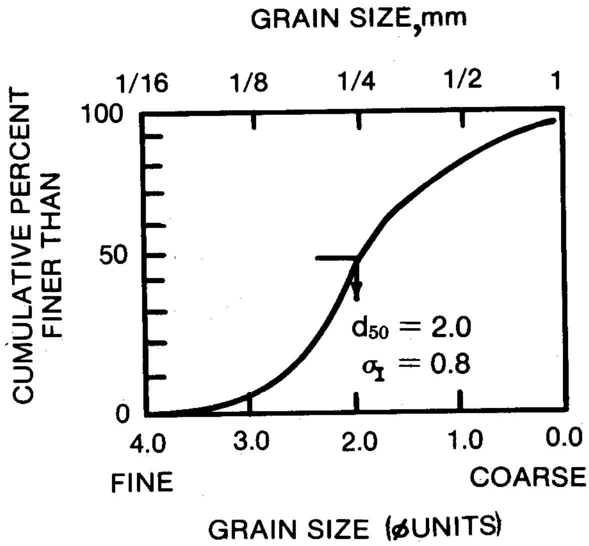
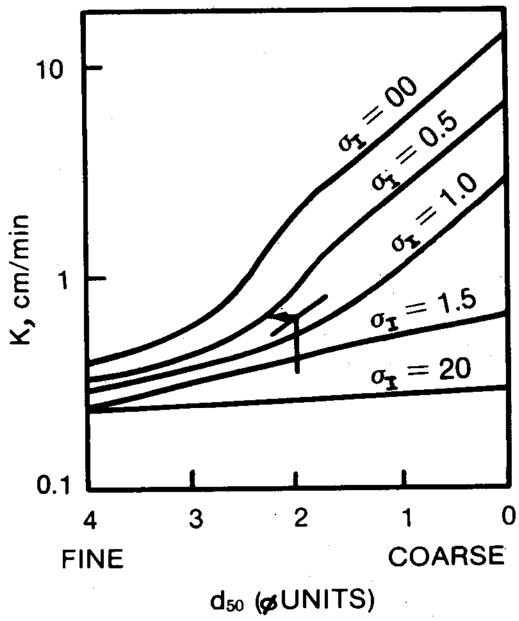


Figure 2-13. Conversion chart for ϕ and grain size in mm, for the range 0.01 to 10 mm (prepared by WES)



a. Grain size distribution



b. Coefficient of permeability versus median grain size

Figure 2-14. Masch and Denny relationship for permeability as a function of median grain size and inclusive standard deviation (courtesy of Prentice-Hall[175])

The median grain size, d_{50} in ϕ units, is determined from the gradation curve as shown in figure 2-14a. Then knowing σ_I and d_{50}, the coefficient of permeability in cm per minute can be obtained from figure 2-14b (Freeze and Cherry 1979).

c. Kozeny-Carman Equation. For uniform loose to dense clean sands classified SP in the Unified Soil Classification System (U. S. Army Engineer Waterways Experiment Station 1960), the permeability may be estimated using the Kozeny-Carman equation (Loudon 1952 and Perloff and Baron 1976)

$$k = \frac{1}{C_s T_o^2 S_s^2} \frac{\gamma_w}{\mu} \frac{e^3}{1 + e} \tag{2-25}$$

where

k = coefficient of permeability

γ_w = unit weight of fluid

e = void ratio

C_s = shape factor corresponding to a particular flow channel

T_o = tortuosity factor related to the degree of sinuous flow

S_s = specific surface (surface area of solids/volume of solids)

μ = coefficient of viscosity of fluid

For sands and silt-sized (finer than 0.074 mm and coarser than 0.005 mm) particles $C_s T_o^2 = 5$ is a good approximation (Perloff and Baron 1976). The specific surface may be obtained from (Loudon 1952)

$$S_s = A(X_1 S_1 + X_2 S_2 + \ldots + X_n S_n) \tag{2-26}$$

where

S_s = specific surface

A = angularity factor

X_1 = percentage of total soil sample between adjacent sieves expressed as a decimal

S_1 = specific surface of spheres uniformly distributed in size between the mesh sizes of adjacent sieves

The angularity factor, A , which varies from 1.0 for glass spheres to 1.8 for crushed glass, may be determined by microscopic examination of the soil or estimated from table 2-4 (Loudon 1952). The specific surface of spheres, S_i , between the mesh sizes d_x and d_y is (Loudon 1952)

$$S_i = \frac{6}{\sqrt{d_x d_y}} \tag{2-27}$$

Specific surfaces of spheres lying between selected U. S. standard sieves is given in table 2-5.

d. Correlation of In Situ Horizontal Permeability and Hazen's Effective Grain Size. For natural fine to medium, relatively uniform sands, classified SP or SW in the Unified Soil Classification System (U. S. Army Engineer Waterways Experiment Station 1960), in the middle and lower Mississippi River Valley, the in situ horizontal permeability may be estimated from the Hazen's effective size as shown in figure 2-15 (U. S. Army Engineer Waterways Experiment Station 1956a). The relationship given in figure 2-15 should not be used outside the geographic area for which it was developed. A similar relationship between transmissivity and median grain size of sands is available for the Arkansas River Valley (Bedinger 1961).

Table 2-4. Angularity Factor for Soil Grains[a]

Type of Material	Description	Angularity Factor
Glass sphere	Well rounded	1.0
Natural sand	Rounded	1.1
	Subrounded	1.2
	Subangular	1.3
	Angular	1.4
Crushed rock	Quartzite	1.5
Crushed rock	Basalt	1.6
Crushed glass	Pyrex	1.8

(a) Courtesy of the Institution of Civil Engineering[210].

Table 2-5. Specific Surface of Spheres Lying Between Selected U. S. Standard Sieve Sizes[a]

U. S. Standard Sieve Numbers	Specific Surface[b] 1/cm
4 to 6	382
6 to 8	538
8 to 10	696
10 to 16	985
16 to 20	1524
20 to 30	2178
30 to 40	3080
40 to 50	4318
50 to 70	6089
70 to 100	8574
100 to 140	12199
140 to 200	17400

(a) Prepared by WES.

(b) $S_i = \dfrac{6}{\sqrt{d_x d_y}}$

e. Computation of Permeability from Consolidation Test. The coefficient of permeability of normally consolidated clays and silts can be computed from the consolidation test using the relationship (Lambe 1951 and Olson and Daniel 1979)

$$k = \frac{C_v a_v \gamma_w}{1 + e_o} \qquad (2\text{-}28)$$

where

c_v = coefficient of consolidation

a_v = coefficient of compressibility

e_o = initial void ratio

2-6. Laboratory Methods for Determining Permeability.

a. General. Laboratory tests described in EM 1110-2-1906 can be used to determine the coefficient of permeability of a soil, Unless otherwise required, the coefficient of permeability shall be determined using deaired distilled water and completely saturated soil specimens. The apparatus used

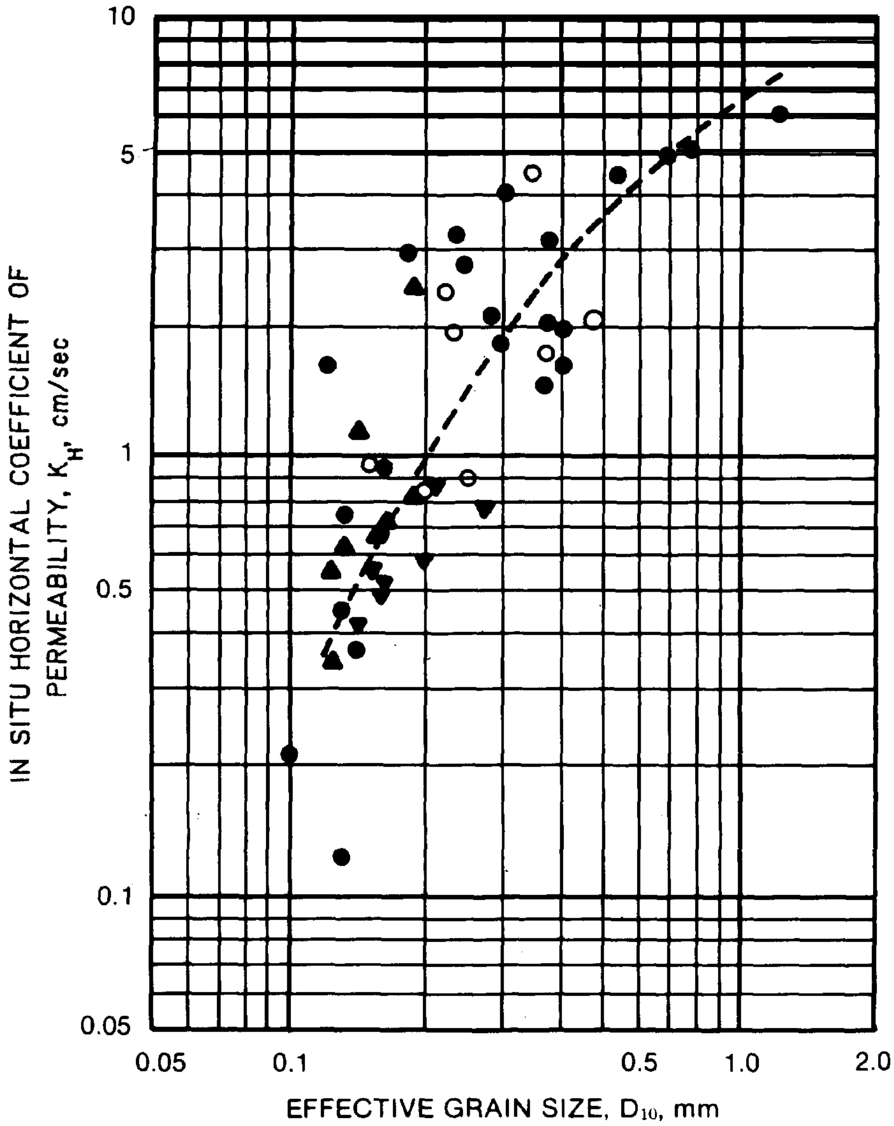


Figure 2-15. Relationship between in situ horizontal permeability and effective size (prepared by WES[120])

for permeability testing may vary depending upon whether the sample is fine-grained or coarse-grained, undisturbed, remolded, or compacted, and saturated or unsaturated. The permeability of remolded coarse-grained soils is determined in permeameter cylinders, while the permeability of undisturbed coarse-grained soils in a vertical direction can be determined using the sampling tube as a permeameter. Samples which have become segregated or contaminated with drilling mud during sampling operations will not give reliable results. The permeability of remolded coarse-grained soils is generally used to approximate the permeability of undisturbed coarse-grained soils in a horizontal direction. Usually the laboratory permeability of remolded coarse-grained soils is considerably less than the horizontal permeability of the coarse-grained soil in the field, so the approximation may not be conservative. Pressure cylinders and consolidometers are used for fine-grained soils in the remolded or undisturbed state. Fine-grained soils can be tested with the specimen oriented to obtain the permeability in either the vertical or horizontal direction.

b. Possible Errors. There are several possible errors in determining permeability in the laboratory (FM 1110-2-1906; Olson and Daniel 1979; and Mitchell, Guzikowski, and Villet 1978).

(1) Use of samples that are not representative of actual field conditions. This can be minimized by thorough field investigation, attention to details (take undisturbed samples from test fills for determination of permeability of embankment materials, sampling along faults, fissures, clay seams, and sand partings for determination of permeability of the dam foundation), and by the use of large samples.

(2) Orientation of the in situ stratum to the direction of seepage flow is seldom duplicated in the laboratory. This can be overcome by obtaining the permeability of the soil (embankment material and/or foundation) in both the vertical and horizontal direction.

(3) Incorrect hydraulic gradient used in the laboratory test. The hydraulic gradient used in the laboratory should cover the range of expected hydraulic gradient in situ. Where possible the hydraulic gradient should be selected so that the flow is laminar (straight line relationship between discharge versus hydraulic gradient) and Darcy's law will be applicable. It is usually not practical to achieve laminar flow for coarser soils, and the laboratory test should be run at the hydraulic gradient anticipated in the field.

(4) Air dissolved in the 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 with increasing time. Permeability tests on saturated specimens should show no significant decrease in permeability with time if properly deaired distilled water is used. However, if such a decrease in permeability occurs, 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.

(5) Leakage along the sides of the permeameter can result in an increased permeability. One major advantage of the triaxial compression chamber for permeability tests is that the specimen is confined by a flexible membrane which is pressed tightly against the specimen by the chamber pressure thus reducing the possibility for leakage along the sides.

2-7. Origin, Occurrence, and Movement of Ground Water.

a. Hydrologic Cycle. Precipitation, runoff, storage, and evaporation of the earth's water follow an unending sequence called the hydraulic cycle, as shown in figure 2-16. Radiation from the sun evaporates water from the oceans into the atmosphere. The moisture is condensed and rises to form cloud formations. From these clouds, the earth receives precipitation of rain,

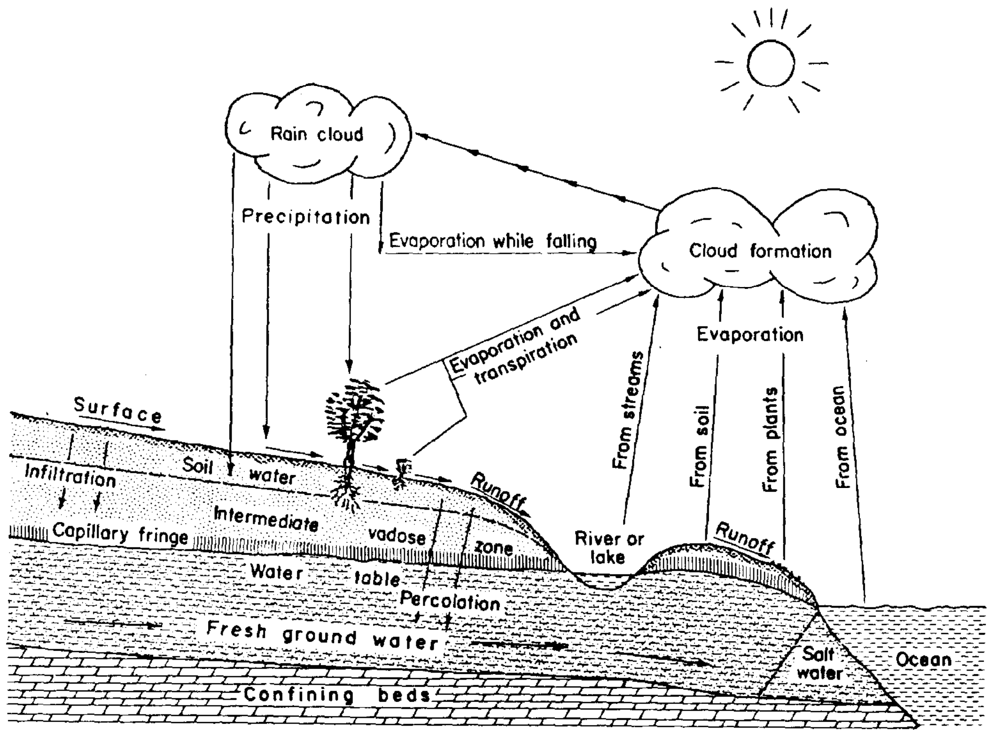


Figure 2-16. Schematic diagram of the hydrologic cycle (courtesy of Johnson Division, Universal Oil Products[189])

snow, sleet, or hail which runs into lakes and streams or seeps into the soil and thence into the underlying rock formations. The percolating water moves through the saturated subsurface materials and may reappear at the surface, at a lower elevation than the level where it entered the ground, in the form of springs and seeps which maintain the flow of streams in dry periods (TM 5-545; Bureau of Reclamation 1977; and Johnson Division, Universal Oil Products 1972).

b. Water Table. The surface below which the soil or rock is saturated is the water table, as shown in figure 2-17. The water table is not a level surface but varies in shape and slope depending upon the variations in permeability and areas of recharge and discharge. In general, the water table reflects the surface topography but with less relief. Ground water is said to be perched if it is separated from the main water table by unsaturated materials, as shown in figure 2-17. An aquifer is a saturated permeable geologic unit that can transmit significant quantities of water under ordinary hydraulic gradients. An unconfined aquifer is one that does not have a confining layer overlying it as shown in figure 2-17. The water table, or upper surface of the saturated ground water is in direct contact with the atmosphere through the open pores of the overlying material and movement of the ground water is in direct response to gravity. The aquifer may be a layer of gravel or sand, permeable sedimentary rocks such as sandstones or limestones, a rubbly zone between lava flows, or even a large body of massive rock, such as fractured

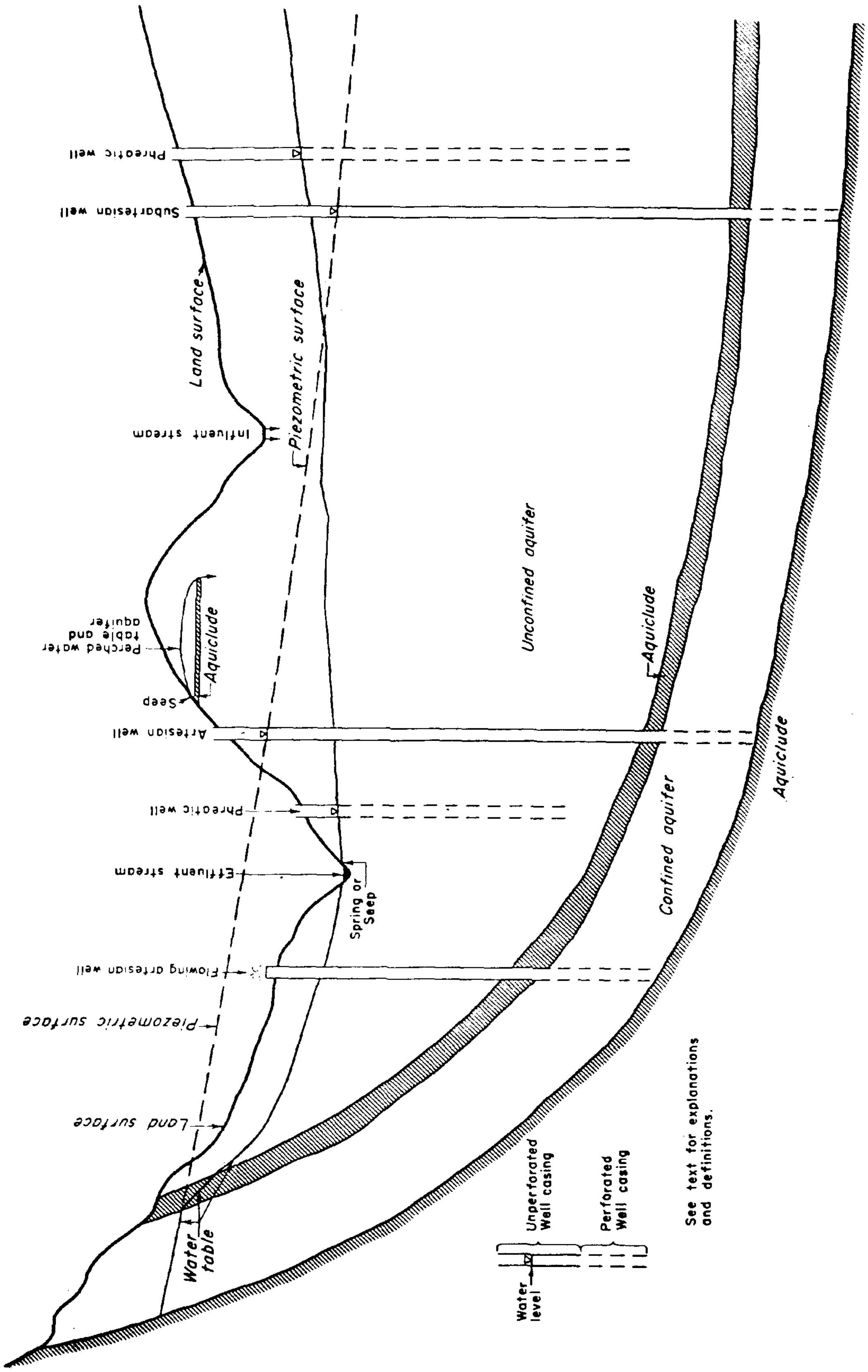


Figure 2-17. Groundwater relationships (courtesy of Soil Conservation Service[68])

granite, which has sizable openings. An aquiclude is a saturated geologic unit that is incapable of transmitting significant quantities of water under ordinary hydraulic gradients as shown in figure 2-17. A confined or artesian aquifer has an overlying confining layer of lower permeability than the aquifer and has only an indirect or distant connection with the atmosphere as. shown in figure 2-17. The water in the artesian aquifer is under pressure and if the aquifer is penetrated by a tightly cased well or a piezometer, the water will rise above the bottom of the confining layer to an elevation at which it is in balance with atmospheric pressure. If this elevation is greater than that of the land surface at the well, artesian water will flow from the well as shown in figure 2-17. The imaginary surface, conforming to the elevations to which water will rise in wells penetrating an artesian aquifer, is the piezometric surface as shown in figure 2-17 (Soil Conservation Service 1978, TM 5-545, Bureau of Reclamation 1977, Freeze and Cherry 1979, and Anonymous 1980).

2-8. Field Methods for Determining Permeability.

a. General. In sands it is difficult to obtain undisturbed soil samples for laboratory testing and the structure (void ratio, stratification, etc.) has an important influence on-permeability. Therefore, field tests for determining permeability are necessary. Because sampling operations do not necessarily indicate the relative perviousness of foundations containing large amounts of gravelly materials, field pumping tests are required to determine the foundation permeability for dams where positive measures are not proposed to completely cut off underseepage in the gravelly formations.

b. Test Pits and Bore Hole Tests.. In sands and gravels above the ground-water level, field tests are normally carried out by measuring the downward seepage from test pits or shallow boreholes (Cedergren 1977). Below the ground-water table information about the order of magnitude and variability of the coefficient of permeability may be obtained by conducting falling head permeability tests in the exploratory boring as drilling proceeds. The hole is cased from the ground surface to the top of the zone to be tested and extends without support for a suitable depth below the casing. If the pervious stratum is not too thick, the uncased hole is extended throughout the full thickness, otherwise the uncased hole penetrates only a part of the pervious stratum. Water is added to raise the water level in the casing and then the water level descends toward its equilibrium position. The elevation of the water level is measured as a function of time and the coefficient of permeability is calculated (Terzaghi and Peck 1967).

$$k = \frac{1}{C} \frac{A \frac{\Delta h}{\Delta t}}{r'_o h'_m} \tag{2-29}$$

where

k = coefficient of permeability

A = inside cross-sectional area of casing

Δh = drop in water level in casing during time interval At

Δt = time interval

C = dimensionless quantity depending on shape of cylindrical hole and depth of penetration into pervious zone (see figure 2-18)

r'_o = mean radius of hole below casing

h'_m = mean distance during time interval At from water level in casing to equilibrium water level in pervious zone

The falling head field permeability test often gives an observed permeability that is too low because silt particles which are suspended in the water may form a filter skin over the walls and bottom of the hole in the pervious material. The results of such tests are little more than an indication of the order of magnitude of the in situ permeability. More reliable data are obtained from field pumping tests.

c. Field Pumping Tests. The most reliable method for determining in situ permeability is a field pumping test on a test well which fully penetrates the aquifer. The test procedures for equilibrium (steady-state flow) and nonequilibrium (transient flow) are given in Appendix III to TM 5-818-5 and Civil Works Engineer Letter 63-16 (U. S. Army Corps of Engineers 1963). The ratio of the horizontal to vertical permeability can be determined from specially conducted field pumping tests (Mansur and Dietrich 1965).

2-9. Chemical Composition of Ground Water and River (or Reservoir) Water.

a. Ground Water. The chemical composition of the ground water is important because some ground waters are highly corrosive to metal screens, pipes, and pumps, or may contain dissolved minerals or carbonates which form incrustations in wells or filters and, with time, cause clogging and reduced efficiency of the dewatering or drainage system. Indications of corrosive and incrusting waters are given in table 2-6 (TM 5-818-5; and Johnson Division, Universal Oil Products 1972). General information concerning ground-water properties is available in an Atlas (Pettyjohn et al. 1979). Sampling, sample preservation, and chemical analysis of ground water is covered in handbooks (Moser and Huibregtse 1976, and Environmental Protection Agency 1976).

b. River (or Reservoir) Water. The total amount of cations (calcium, magnesium, potassium, and sodium) in the river water (for dams not yet constructed) and in the reservoir water (for existing dams) significantly influences the erosion through a possible crack in the core of the dam (Perry 1975). Usually, as the total amount of cations in the eroding water decreases, the erodibility of the soil increases. For dams constructed of dispersive clay, the susceptibility of the dam to piping depends, in part, upon the total amount of cations in the seepage water (Perry 1979).

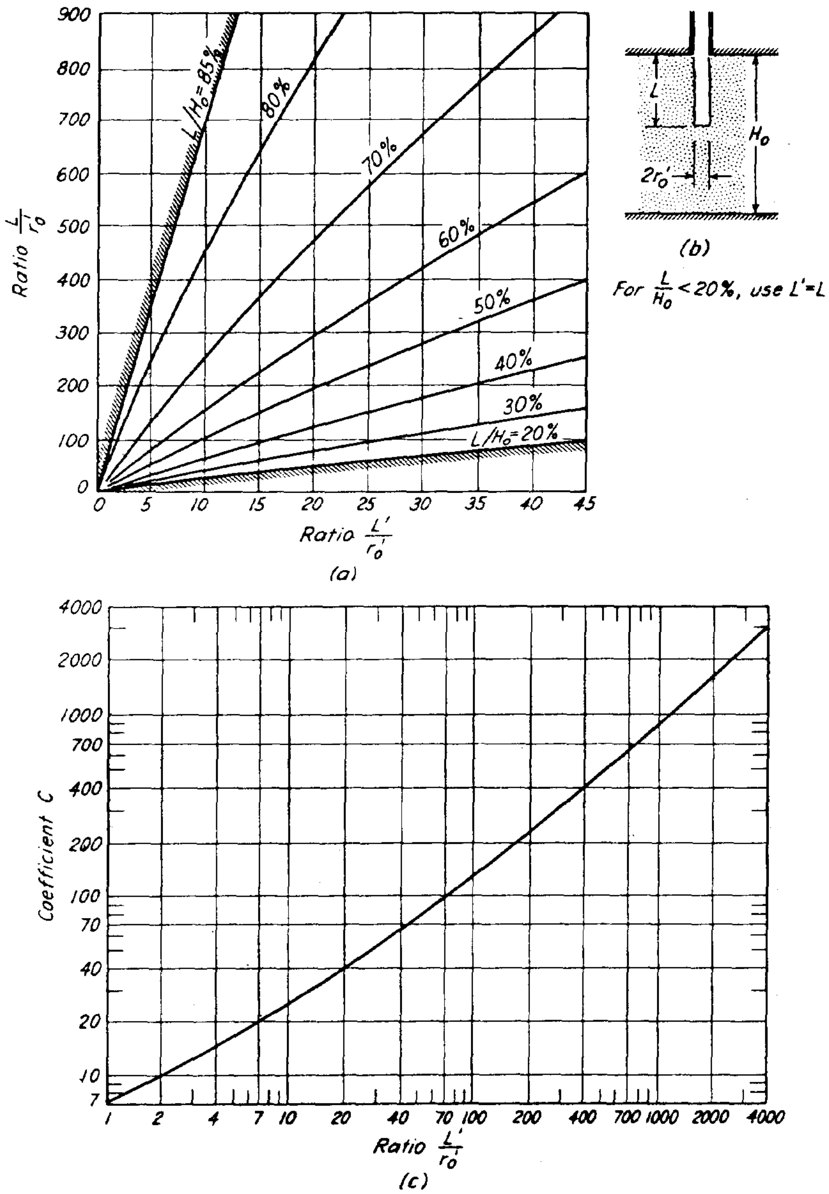


Figure 2-18. Field permeability test in bore hole (courtesy of John Wiley and Sons[175])

Table 2-6. Indicators of Corrosive and Incrusting Waters[(a)]

Indicators of Corrosive Water	Indicators of Incrusting Water
1. A pH less than 7	1. a pH greater than 7
2. Dissolved oxygen in excess of 2 ppm[(b)]	2. Total iron (Fe) in. excess of 2 ppm
3. Hydrogen sulfide (H_2S) in excess of 1 ppm, detected by a rotten egg odor	3. Total manganese (Mn) in excess of 1 ppm in conjunction with a high pH and the presence of oxygen
4. Total dissolved solids in excess of 1000 ppm indicates an ability to conduct electric current great enough to cause serious electrolytic corrosion	4. Total carbonate hardness in excess of 300 ppm
5. Carbon dioxide ($C0_2$) in excess of 50 pm	
6. Chlorides (CL) in excess of 500 ppm	

(a) From TM 5-818-5[1].

(b) ppm = parts per million.