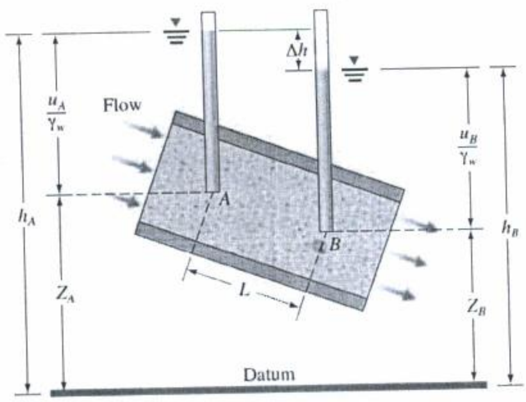
B-46

MATERY OF SOIL MECHANIS -1

(MODUL-2)

MEKANIKA TANAH 1



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MATERI

Tegangan Effektif Tanah

Rembesan dan Jaringan Aliran

Pemadatan Tanah

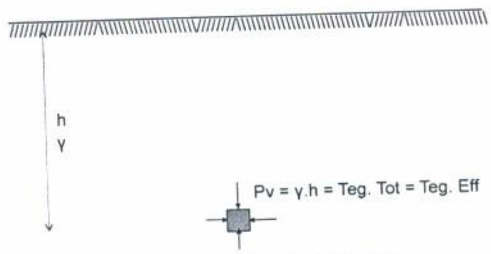
California Bearing ratio

Tegangan Efektif

Beberapa hal yang perlu diketahui :

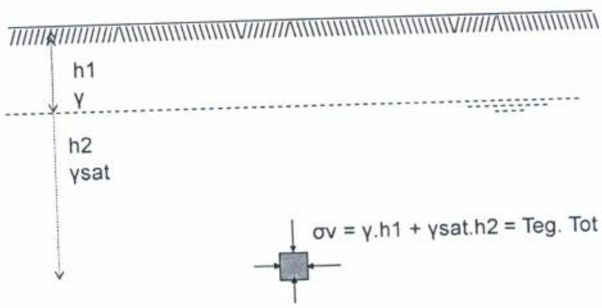
- Parameter tanah (γ , γsat , Gs , e , w)
- Hubungan antara parameter
- Kondisi permukaan air tanah
- Tegangan Total (σt)
- Tekanan Air Pori (u)
- Tegangan Efektif (σ' = σt - u)

TEGANGAN EFFEKTIF
(Tanpa Muka Air Tanah)



Pv = Teg. Akibat berat sendiri tanah arah vertikal

TEGANGAN EFFEKTIF
(Dengan Muka Air Tanah)



σ = Teg. Total
U = γw . h2
σ' = γ.h1 + γsat.h2 – γw.h2

PERMEBILITAS DAN REMBESAN
(HIDRAULIC CONDUCTIVITY AND SEEPAGE)

Soils have interconnected voids through which water can flow from points of high energy to points of low energy. The study of the flow of water through porous soil media is important in soil mechanics. It is necessary for estimating the quantity of underground seepage under various hydraulic conditions, for investigating problems involving the pumping of water for underground construction, and for making stability analyses of earth dams and earth-retaining structures that are subject to seepage forces.

HYDRAULIC CONDUCTIVITY

Bernoulli's Equation

From fluid mechanics we know that, according to Bernoulli's equation, the total head at a point in water under motion can be given by the sum of the pressure, velocity, and elevation heads, or

$$h = \underbrace{\frac{u}{\gamma_w}}_{\text{Pressure head}} + \underbrace{\frac{v^2}{2g}}_{\text{Velocity head}} + \underbrace{Z}_{\text{Elevation head}} \qquad (5.1)$$

where

h = total head
u = pressure
v = velocity
g = acceleration due to gravity
γ_w = unit weight of water

Note that the elevation head, Z, is the vertical distance of a given point above or below a datum plane. The pressure head is the water pressure, u, at that point divided by the unit weight of water, γ_w.

If Bernoulli's equation is applied to the flow of water through a porous soil medium, the term containing the velocity head can be neglected because the seepage velocity is small. Then the total head at any point can be adequately represented by

$$h = \frac{u}{\gamma_w} + Z \tag{5.2}$$

Figure 5.1 shows the relationship among the pressure, elevation, and total heads for the flow of water through soil. Open standpipes called *piezometers* are installed at points A and B. The levels to which water rises in the piezometer tubes situated at points A and B are known as the *piezometric levels* of points A and B, respectively. The pressure head at a point is the height of the vertical column of water in the piezometer installed at that point.

The loss of head between two points, A and B, can be given by

$$\Delta h = h_A - h_B = \left(\frac{u_A}{\gamma_w} + Z_A\right) - \left(\frac{u_B}{\gamma_w} + Z_B\right) \tag{5.3}$$

The head loss, Δh, can be expressed in a nondimensional form as

$$i = \frac{\Delta h}{L} \tag{5.4}$$

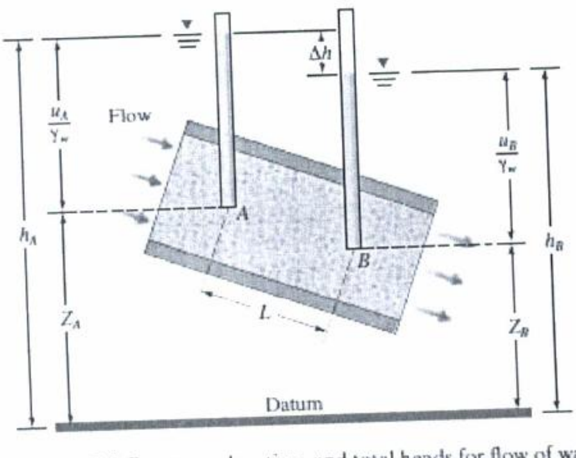


Figure 5.1 Pressure, elevation, and total heads for flow of water through soil

Darcy's Law

In 1856, Henri Philibert Gaspard Darcy published a simple empirical equation for the discharge velocity of water through saturated soils. This equation was based primarily on Darcy's observations about the flow of water through clean sands and is given as

$$v = ki \tag{5.6}$$

where

v = *discharge velocity*, which is the quantity of water flowing in unit time through a unit gross cross-sectional area of soil at right angles to the direction of flow

k = hydraulic conductivity (otherwise known as the coefficient of permeability)

$$q = vA = A_v v_s \tag{5.7}$$

where

v_s = *seepage velocity*

A_v = area of void in the cross section of the specimen

However,

$$A = A_v + A_s \tag{5.8}$$

where A_s = area of soil solids in the cross section of the specimen. Combining Eqs. (5.7) and (5.8) gives

$$q = v(A_v + A_s) = A_v v_s$$

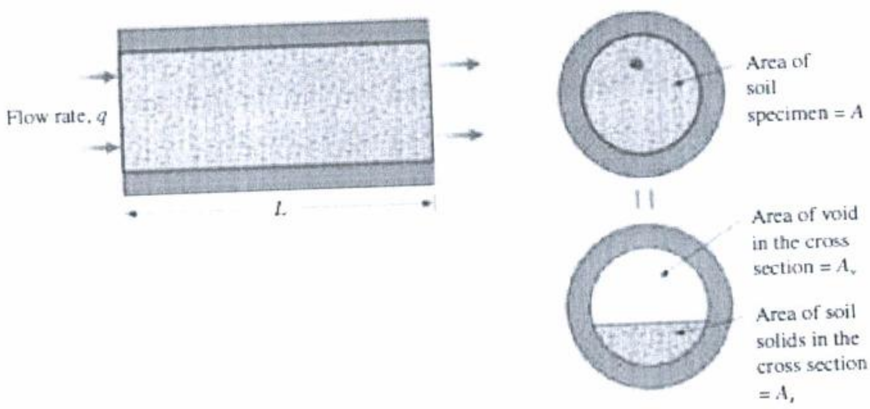


Figure 5.3 Derivation of Eq. (5.10)

Hydraulic Conductivity

The hydraulic conductivity of soils depends on several factors: fluid viscosity, pore-size distribution, grain-size distribution, void ratio, roughness of mineral particles, and degree of soil saturation. In clayey soils, structure plays an important role in hydraulic conductivity. Other major factors that affect the hydraulic conductivity of clays are the ionic concentration and the thickness of layers of water held to the clay particles.

The value of hydraulic conductivity, k, varies widely for different soils. Some typical values for saturated soils are given in Table 5.1. The hydraulic conductivity of unsaturated soils is lower and increases rapidly with the degree of saturation.

Table 5.1 Typical values of hydraulic conductivity for saturated soils

Soil type	k (cm /sec)
Clean gravel	100–1
Coarse sand	1.0–0.01
Fine sand	0.01–0.001
Silty clay	0.001–0.00001
Clay	<0.000001

The hydraulic conductivity of a soil is also related to the properties of the fluid flowing through it by the following equation:

$$k = \frac{\gamma_w}{\eta}\overline{K} \quad (5.11)$$

where

γ_w = unit weight of water
η = viscosity of water
$\overline{K}$ = absolute permeability

The *absolute permeability*, $\overline{K}$, is expressed in units of length squared (that is, cm^2).

Laboratory Determination of Hydraulic Conductivity

Two standard laboratory tests are used to determine the hydraulic conductivity of soil: the constant head test and the falling head test. The constant head test is used primarily for coarse-grained soils. For fine-grained soils, however, the flow rates through the soil are too small and, therefore, falling head tests are preferred. A brief description of each follows.

Constant Head Test

A typical arrangement of the constant head permeability test is shown in Figure 5.4. In this type of laboratory setup, the water supply at the inlet is adjusted in such a way that the difference of head between the inlet and the outlet remains constant during the test period. After a constant flow rate is established, water is collected in a graduated flask for a known duration.

The total volume of water, Q, collected may be expressed as

$$Q = Avt = A(ki)t \quad (5.12)$$

where

A = area of cross section of the soil specimen
t = duration of water collection

Also, because

$$i = \frac{h}{L} \quad (5.13)$$

where L = length of the specimen, Eq. (5.13) can be substituted into Eq. (5.12) to yield

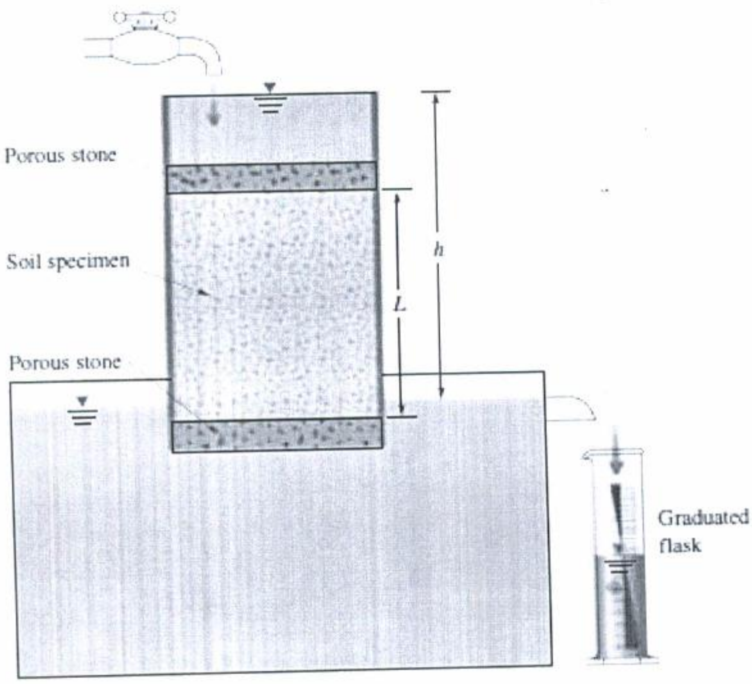


Figure 5.4 Constant head permeability test

$$Q = A\left(k\frac{h}{L}\right)t \tag{5.14}$$

or

$$k = \frac{QL}{Aht} \tag{5.15}$$

Falling Head Test

A typical arrangement of the falling head permeability test is shown in Figure 5.5. Water from a standpipe flows through the soil. The initial head difference, h_1, at time $t = 0$ is recorded, and water is allowed to flow through the soil specimen such that the final head difference at time $t = t_2$ is h_2.

The rate of flow of the water, q, through the specimen at any time t can be given by

$$q = k\frac{h}{L}A = -a\frac{dh}{dt} \tag{5.16}$$

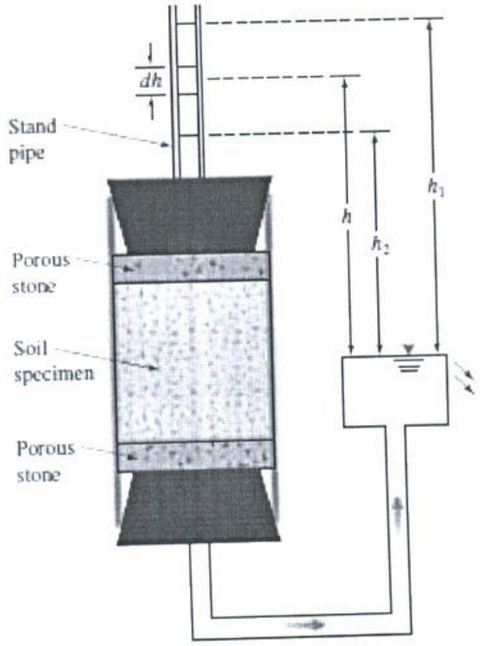


Figure 5.5
Falling head permeability test

where

a = cross-sectional area of the standpipe
A = cross-sectional area of the soil specimen

Rearranging Eq. (5.16) gives

$$dt = \frac{aL}{Ak}\left(-\frac{dh}{h}\right) \qquad (5.17)$$

Integration of the left side of Eq. (5.17) with limits of time from 0 to t and the right side with limits of head difference from h_1 to h_2 gives

$$t = \frac{aL}{Ak}\log_e\frac{h_1}{h_2}$$

or

$$k = 2.303\frac{aL}{At}\log_{10}\frac{h_1}{h_2} \qquad (5.18)$$

Equivalent Hydraulic Conductivity in Stratified Soil

Depending on the nature of soil deposit, the hydraulic conductivity of a given layer of soil may vary with the direction of flow. In a stratified soil deposit where the hydraulic conductivity for flow in different directions changes from layer to layer, an equivalent hydraulic conductivity determination becomes necessary to simplify calculations. The following derivations relate to the equivalent hydraulic conductivity for flow in vertical and horizontal directions through multilayered soils with horizontal stratification.

Figure 5.11 shows n layers of soil with flow in the *horizontal direction*. Let us consider a cross-section of unit length passing through the n layer and perpendicular to the direction of flow. The total flow through the cross-section in unit time can be written as

$$\begin{aligned} q &= v \cdot 1 \cdot H \\ &= v_1 \cdot 1 \cdot H_1 + v_2 \cdot 1 \cdot H_2 + v_3 \cdot 1 \cdot H_3 + \cdots + v_n \cdot 1 \cdot H_n \end{aligned} \tag{5.29}$$

where

v = average discharge velocity
$v_1, v_2, v_3, \ldots, v_n$ = discharge velocities of flow in layers denoted by the subscripts.

If $k_{H_1}, k_{H_2}, k_{H_3}, \ldots, k_{H_n}$ are the hydraulic conductivity of the individual layers in the horizontal direction, and $k_{H(eq)}$ is the equivalent hydraulic conductivity in the horizontal direction, then from Darcy's law

$$v = k_{H(eq)} i_{eq};\ v_1 = k_{H_1} i_1;\ v_2 = k_{H_2} i_2;\ v_3 = k_{H_3} i_3;\ \ldots;\ v_n = k_{H_n} i_n$$

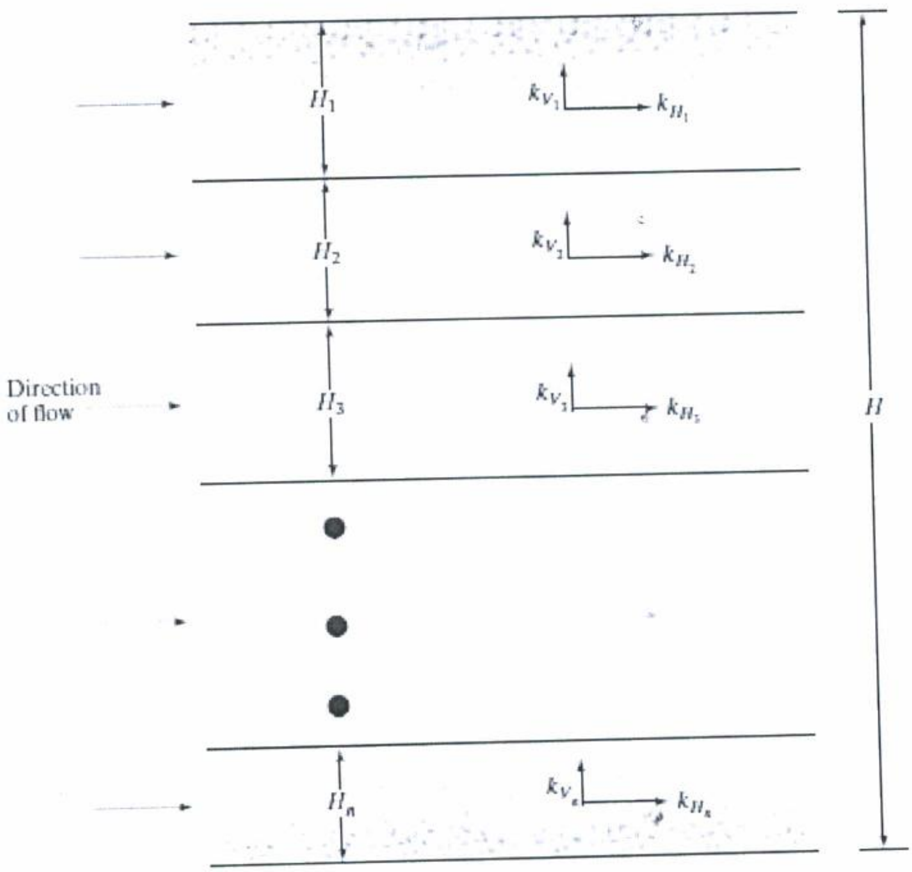


Figure 5.11 Equivalent hydraulic conductivity determination—horizontal flow in stratified soil

Substitution of the preceding relations for velocities in Eq. (5.29) and noting $i_{eq} = i_1 = i_2 = i_3 = \cdots i_n$ results in

$$k_{H(eq)} = \frac{1}{H}(k_{H_1}H_1 + k_{H_2}H_2 + k_{H_3}H_3 + \cdots + k_{H_n}H_n) \tag{5.30}$$

Figure 5.12 shows n layers of soil with flow in the vertical direction. In this case, the velocity of flow through all the layers is the same. However, the total head loss, h, is equal to the sum of the head loss in each layer. Thus

$$v = v_1 = v_2 = v_3 = \cdots = v_n \tag{5.31}$$

and

$$h = h_1 + h_2 + h_3 + \cdots + h_n \tag{5.32}$$

Using Darcy's law, Eq. (5.31) can be rewritten as

$$k_{V(eq)}\frac{h}{H} = k_{V_1}i_1 = k_{V_2}i_2 = k_{V_3}i_3 = \cdots = k_{V_n}i_n \tag{5.33}$$

where $k_{V1}, k_{V2}, k_{V3}, \ldots, k_{Vn}$ are the hydraulic conductivities of the individual layers in the vertical direction and $k_{V(eq)}$ is the equivalent hydraulic conductivity.

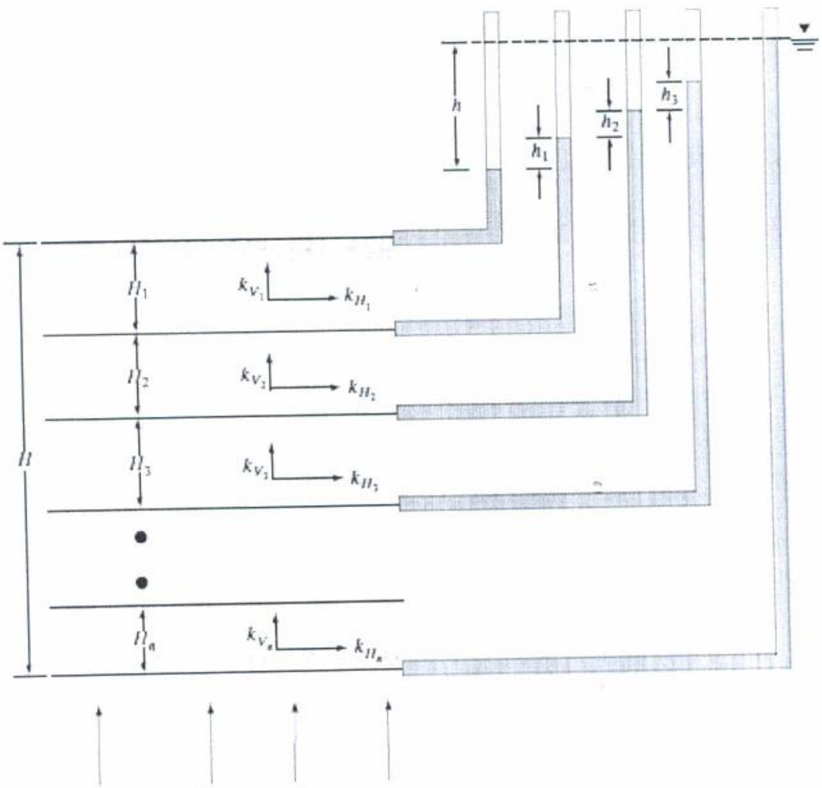


Figure 5.12 Equivalent hydraulic conductivity determination—vertical flow in stratified soil

Again, from Eq. (5.32)

$$h = H_1 i_1 + H_2 i_2 + H_3 i_3 + \cdots + H_n i_n \tag{5.34}$$

Solution of Eqs. (5.33) and (5.34) gives

$$k_{V(eq)} = \frac{H}{\left(\frac{H_1}{k_{V_1}}\right) + \left(\frac{H_2}{k_{V_2}}\right) + \left(\frac{H_3}{k_{V_3}}\right) + \cdots + \left(\frac{H_n}{k_{V_n}}\right)} \tag{5.35}$$

Penentuan Koefisien Rembesan dengan Pumping Test

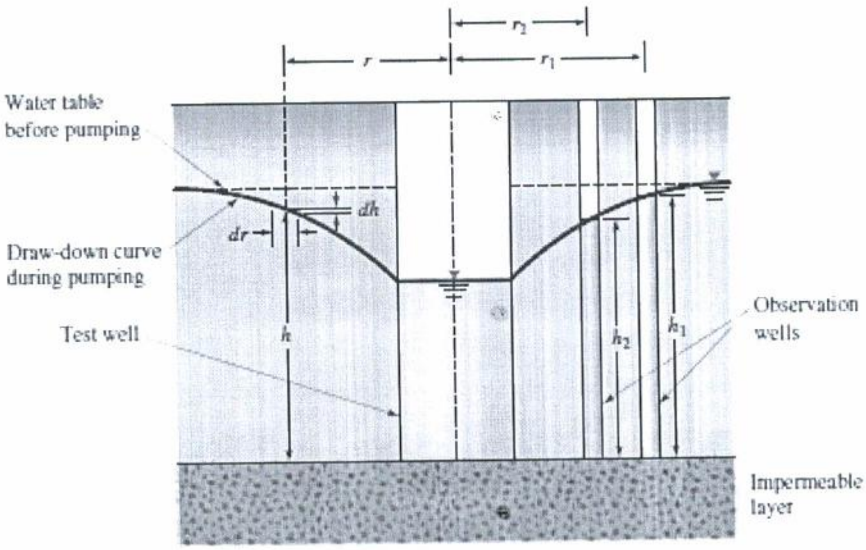


Figure 5.13 Pumping test from a well in an unconfined permeable layer underlain by an impermeable stratum

unconfined and underlain by an impermeable layer. During the test, water is pumped out at a constant rate from a test well that has a perforated casing. Several observation wells at various radial distances are made around the test well. Continuous observations of the water level in the test well and in the observation wells are made after the start of pumping, until a steady state is reached. The steady state is established when the water level in the test and observation wells becomes constant. The expression for the rate of flow of groundwater, q, into the well, which is equal to the rate of discharge from pumping, can be written as

$$q = k\left(\frac{dh}{dr}\right)2\pi r h \tag{5.36}$$

or

$$\int_{r_2}^{r_1} \frac{dr}{r} = \left(\frac{2\pi k}{q}\right)\int_{h_2}^{h_1} h\,dh$$

Thus,

$$k = \frac{2.303 q \log_{10}\left(\frac{r_1}{r_2}\right)}{\pi(h_1^2 - h_2^2)} \tag{5.37}$$

From field measurements, if q, r_1, r_2, h_1, and h_2 are known, then the hydraulic conductivity can be calculated from the simple relationship presented in Eq. (5.37).

The average hydraulic conductivity for a confined aquifer can also be determined by conducting a pumping test from a well with a perforated casing that penetrates the full depth of the aquifer and by observing the piezometric level in a number of observation wells at various radial distances (Figure 5.14). Pumping is continued at a uniform rate q until a steady state is reached.

Because water can enter the test well only from the aquifer of thickness H, the steady state of discharge is

$$q = k\left(\frac{dh}{dr}\right)2\pi r H \tag{5.38}$$

or

$$\int_{r_2}^{r_1} \frac{dr}{r} = \int_{h_2}^{h_1} \frac{2\pi k H}{q}\, dh$$

This gives the hydraulic conductivity in the direction of flow as

$$k = \frac{q \log_{10}\left(\frac{r_1}{r_2}\right)}{2.727\, H(h_1 - h_2)} \tag{5.39}$$

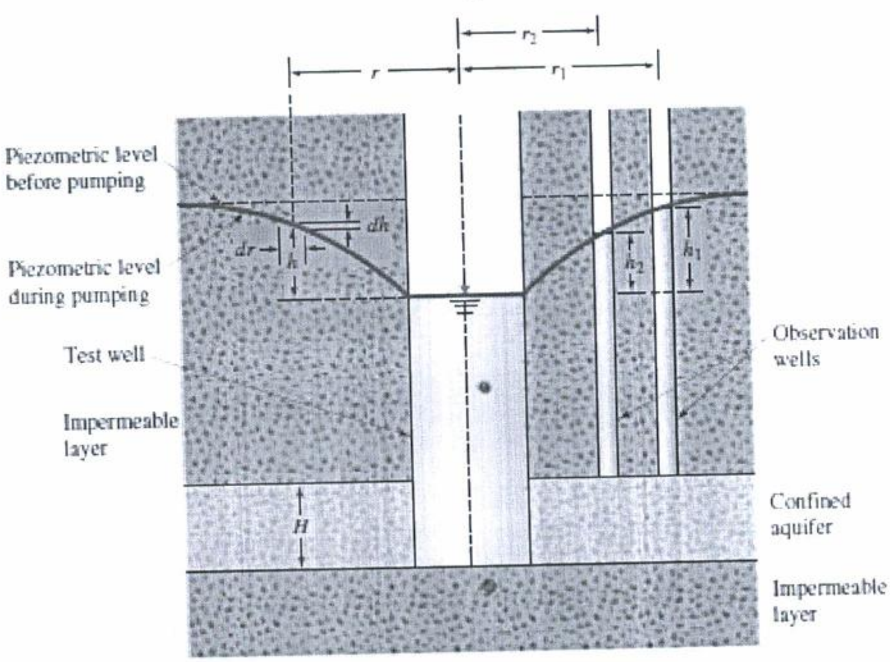


Figure 5.14 Pumping test from a well penetrating the full depth in a confined aquifer

SEEPAGE

In the preceding sections of this chapter, we considered some simple cases for which direct application of Darcy's law was required to calculate the flow of water through soil. In many instances, the flow of water through soil is not in one direction only, and it is not uniform over the entire area perpendicular to the flow. In such cases, the groundwater flow is generally calculated by the use of graphs referred to as *flow nets*. The concept of the flow net is based on *Laplace's equation of continuity*, which governs the steady flow condition for a given point in the soil mass. The following sections explain the derivation of Laplace's equation of continuity and its application to drawing flow nets.

Laplace's Equation of Continuity

To derive the Laplace differential equation of continuity, we consider a single row of sheet piles that have been driven into a permeable soil layer, as shown in Figure 5.15a. The row of sheet piles is assumed to be impervious. The steady-state flow of water from the upstream to the downstream side through the permeable layer is a two-dimensional flow. For flow at a point A, we consider an elemental soil block. The block has dimensions dx, dy, and dz (length dy is perpendicular to the plane of the paper); it is shown in an enlarged scale in Figure 5.15b. Let v_x and v_z be the components of the discharge velocity in the horizontal and vertical directions, respectively. The rate of flow of water into the elemental block in the horizontal direction is equal to $v_x\, dz\, dy$, and in the vertical direction it is $v_z\, dx\, dy$. The rates of outflow from the block in the horizontal and vertical directions are

$$\left(v_x + \frac{\partial v_x}{\partial x}dx\right)dz\,dy$$

and

$$\left(v_z + \frac{\partial v_z}{\partial z}dz\right)dx\,dy$$

respectively. Assuming that water is incompressible and that no volume change in the soil mass occurs, we know that the total rate of inflow should equal the total rate of outflow. Thus,

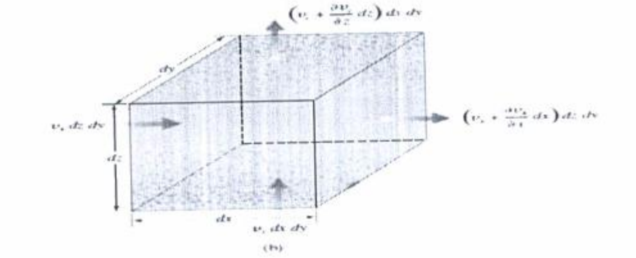
$$\left[\left(v_x + \frac{\partial v_x}{\partial x}dx\right)dz\,dy + \left(v_z + \frac{\partial v_z}{\partial z}dz\right)dx\,dy\right] - [v_x\,dz\,dy + v_z\,dx\,dy] = 0$$

or

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_z}{\partial z} = 0 \qquad (5.40)$$

With Darcy's law, the discharge velocities can be expressed as

$$v_x = k_x i_x = k_x\left(-\frac{\partial h}{\partial x}\right) \qquad (5.41)$$



and

$$v_z = k_z i_z = k_z\left(-\frac{\partial h}{\partial z}\right) \tag{5.42}$$

where k_x and k_z are the hydraulic conductivities in the vertical and horizontal directions, respectively.

From Eqs. (5.40), (5.41), and (5.42), we can write

$$k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \tag{5.43}$$

If the soil is isotropic with respect to the hydraulic conductivity—that is, $k_x = k_z$—the preceding continuity equation for two-dimensional flow simplifies to

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0 \tag{5.44}$$

Flow Nets

The continuity equation [Eq. (5.44)] in an isotropic medium represents two orthogonal families of curves: the flow lines and the equipotential lines. A *flow line* is a line along which a water particle will travel from the upstream to the downstream side in the permeable soil medium. An *equipotential line* is a line along which the potential head at all points is equal. Thus, if piezometers are placed at different points along an equipotential line, the water level will rise to the same elevation in all of them. Figure 5.16a demonstrates the definition of flow and equipotential lines for flow in the permeable soil layer around the row of sheet piles shown in Figure 5.15 (for $k_x = k_z = k$).

A combination of a number of flow lines and equipotential lines is called a *flow net*. Flow nets are constructed to calculate groundwater flow in the media. To complete the graphic construction of a flow net, one must draw the flow and equipotential lines in such a way that the equipotential lines intersect the flow lines at right angles and the flow elements formed are approximate squares.

Figure 5.16b shows an example of a completed flow net. Another example of a flow net in an isotropic permeable layer is shown in Figure 5.17. In these figures, N_f is the number of flow channels in the flow net, and N_d is the number of potential drops (defined later in this chapter).

Drawing a flow net takes several trials. While constructing the flow net, keep the boundary conditions in mind. For the flow net shown in Figure 5.16b, the following four boundary conditions apply:

1. The upstream and downstream surfaces of the permeable layer (lines *ab* and *de*) are equipotential lines.
2. Because *ab* and *de* are equipotential lines, all the flow lines intersect them at right angles.

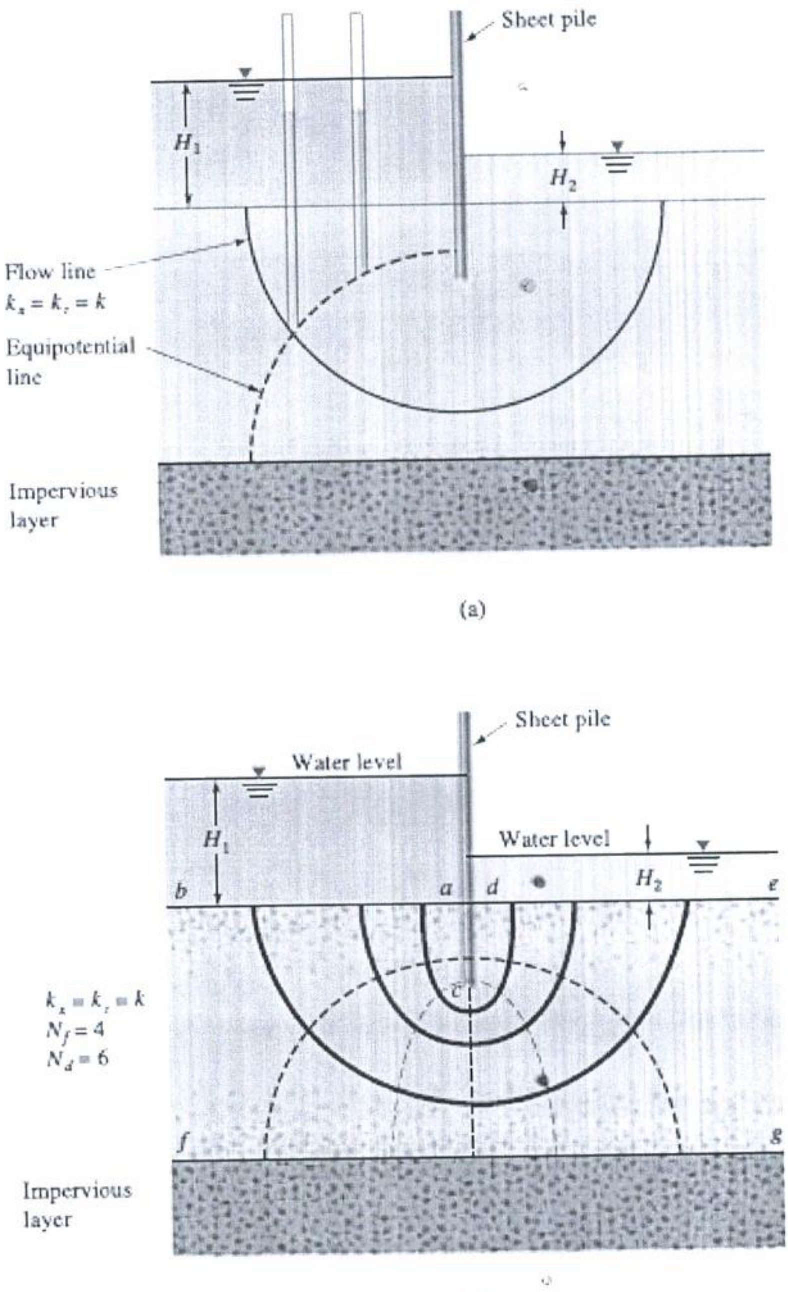


Figure 5.16 (a) Definition of flow lines and equipotential lines; (b) completed flow net

3. The boundary of the impervious layer—that is, line fg—is a flow line, and so is the surface of the impervious sheet pile, line acd.
4. The equipotential lines intersect acd and fg at right angles.

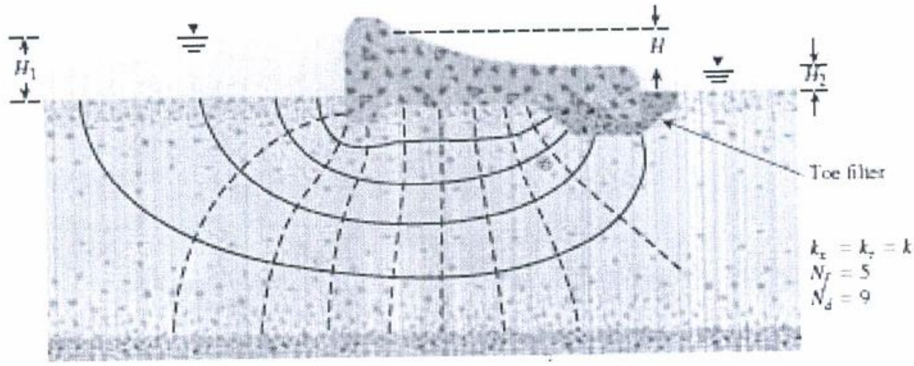


Figure 5.17 Flow net under a dam with toe filter

Seepage Calculation from a Flow Net

In any flow net, the strip between any two adjacent flow lines is called a *flow channel*. Figure 5.18 shows a flow channel with the equipotential lines forming square elements. Let $h_1, h_2, h_3, h_4, \ldots, h_n$ be the piezometric levels corresponding to the equipotential lines. The rate of seepage through the flow channel per unit length (perpendicular to the vertical section through the permeable layer) can be calculated as follows: Because there is no flow across the flow lines,

$$\Delta q_1 = \Delta q_2 = \Delta q_3 = \cdots = \Delta q \tag{5.45}$$

From Darcy's law, the flow rate is equal to kiA. Thus, Eq. (5.45) can be written as

$$\Delta q = k\left(\frac{h_1 - h_2}{l_1}\right)l_1 = k\left(\frac{h_2 - h_3}{l_2}\right)l_2 = k\left(\frac{h_3 - h_4}{l_3}\right)l_3 = \cdots \tag{5.46}$$

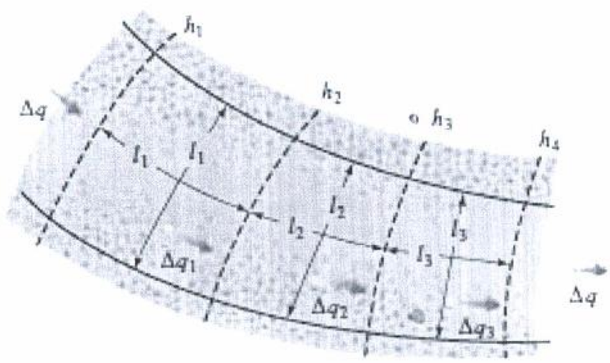


Figure 5.18 Seepage through a flow channel with square elements

Equation (5.46) shows that if the flow elements are drawn as approximate squares, then the drop in the piezometric level between any two adjacent equipotential lines is the same. This is called the *potential drop*. Thus,

$$h_1 - h_2 = h_2 - h_3 = h_3 - h_4 = \cdots = \frac{H}{N_d} \quad (5.47)$$

and

$$\Delta q = k\frac{H}{N_d} \quad (5.48)$$

where

H = head difference between the upstream and downstream sides
N_d = number of potential drops

In Figure 5.16b, for any flow channel, $H = H_1 - H_2$ and $N_d = 6$.

If the number of flow channels in a flow net is equal to N_f, the total rate of flow through all the channels per unit length can be given by

$$q = k\frac{HN_f}{N_d} \quad (5.49)$$

Although drawing square elements for a flow net is convenient, it is not always necessary. Alternatively, one can draw a rectangular mesh for a flow channel, as shown in Figure 5.19, provided that the width-to-length ratios for all the rectangular elements in the flow net are the same. In this case, Eq. (5.46) for rate of flow through the channel can be modified to

$$\Delta q = k\left(\frac{h_1 - h_2}{l_1}\right)b_1 = k\left(\frac{h_2 - h_3}{l_2}\right)b_2 = k\left(\frac{h_3 - h_4}{l_3}\right)b_3 = \cdots \quad (5.50)$$

If $b_1/l_1 = b_2/l_2 = b_3/l_3 = \cdots = n$ (i.e., the elements are not square), Eqs. (5.48) and (5.49) can be modified:

$$\Delta q = kH\left(\frac{n}{N_d}\right) \quad (5.51)$$

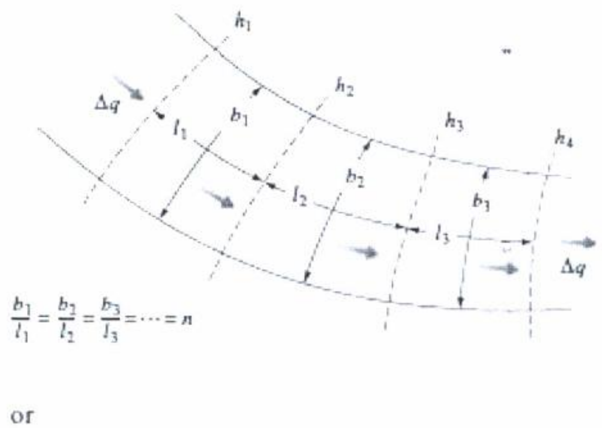


Figure 5.19 Seepage through a flow channel with rectangular elements

or

$$q = kH\left(\frac{N_f}{N_d}\right)n \quad (5.52)$$

Figure 5.20 shows a flow net for seepage around a single row of sheet piles. Note that flow channels 1 and 2 have square elements. Hence, the rate of flow through these two channels can be obtained from Eq. (5.48):

$$\Delta q_1 + \Delta q_2 = k\frac{H}{N_d} + k\frac{H}{N_d} = 2k\frac{H}{N_d}$$

However, flow channel 3 has rectangular elements. These elements have a width-to-length ratio of about 0.38; hence, from Eq. (5.51), we have

$$\Delta q_3 = kH\left(\frac{0.38}{N_d}\right)$$

So, the total rate of seepage can be given as

$$q = \Delta q_1 + \Delta q_2 + \Delta q_3 = 2.38\frac{kH}{N_d}$$

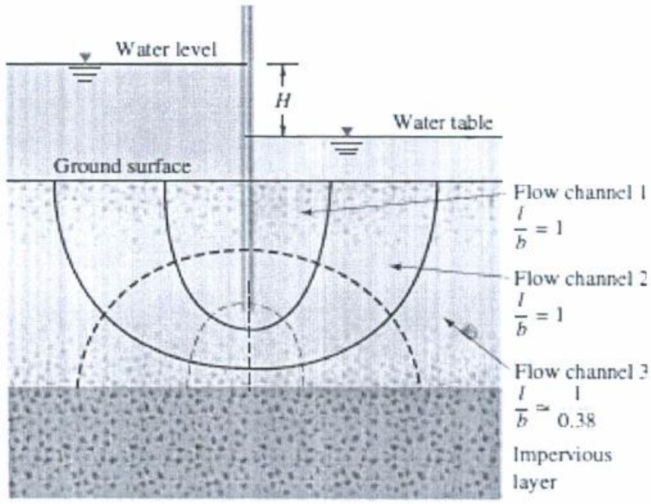


Figure 5.20 Flow net for seepage around a single row of sheet piles

PEMADATAN TANAH (SOIL COMPACTION)

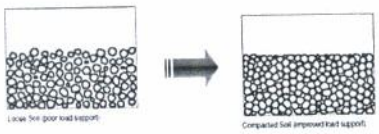
SOIL COMPACTION

Tujuannya :

- Mencapai kepadatan yang lebih tinggi
- Angka pori yang lebih rendah
- Rembesan yang lebih kecil
- Kuat geser yang lebih tinggi
- Daya dukung yang lebih tinggi
- Stabilitas yang baik

Soil Compaction

Sebuah proses dimana partikel-partikel tanah tersusun secara lebih rapat dengan berkurangnya volume pori, sebagai hasil dari pemakaian beban seperti rolling (gilas), tamping (menumbuk) atau getaran. Proses ini meliputi keluarnya udara dari pori tanpa perubahan yang signifikan terhadap kadar air.



COMPACTION CURVE

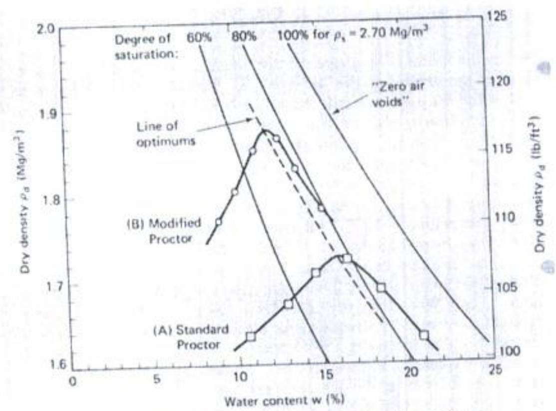


Fig. 5.1 Standard and modified Proctor compaction curves for Crosby B till.

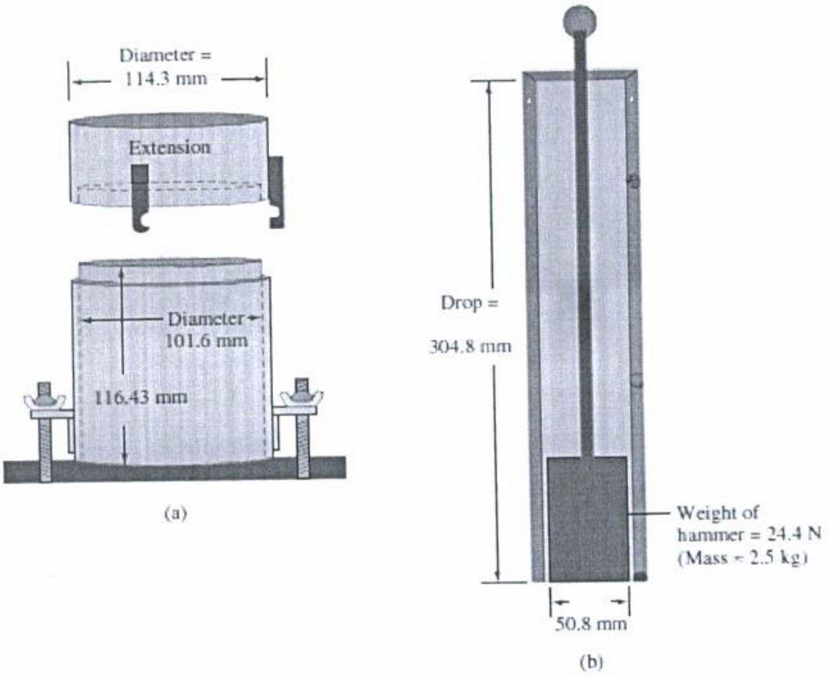


Figure 4.2 Standard Proctor test equipment: (a) mold; (b) hammer

For each test, the moisture content of the compacted soil is determined in the laboratory. With known moisture content, the dry unit weight γ_d can be calculated as

$$\gamma_d = \frac{\gamma}{1 + \frac{w\,(\%)}{100}} \tag{4.2}$$

$$\gamma_{zav} = \frac{G_s \gamma_w}{1 + e}$$

where

γ_{zav} = zero-air-void unit weight
γ_w = unit weight of water
e = void ratio
G_s = specific gravity of soil solids

For 100% saturation, $e = wG_s$, so

$$\gamma_{zav} = \frac{G_s \gamma_w}{1 + wG_s} = \frac{\gamma_w}{w + \frac{1}{G_s}}$$

where w = moisture content.

Effect of Compaction Effort

The compaction energy per unit volume, E, used for the standard Proctor test described in Section 4.2 can be given as

$$E = \frac{\begin{pmatrix}\text{number}\\ \text{of blows}\\ \text{per layer}\end{pmatrix} \times \begin{pmatrix}\text{number}\\ \text{of}\\ \text{layers}\end{pmatrix} \times \begin{pmatrix}\text{weight}\\ \text{of}\\ \text{hammer}\end{pmatrix} \times \begin{pmatrix}\text{height of}\\ \text{drop of}\\ \text{hammer}\end{pmatrix}}{\text{volume of mold}} \qquad (4.7)$$

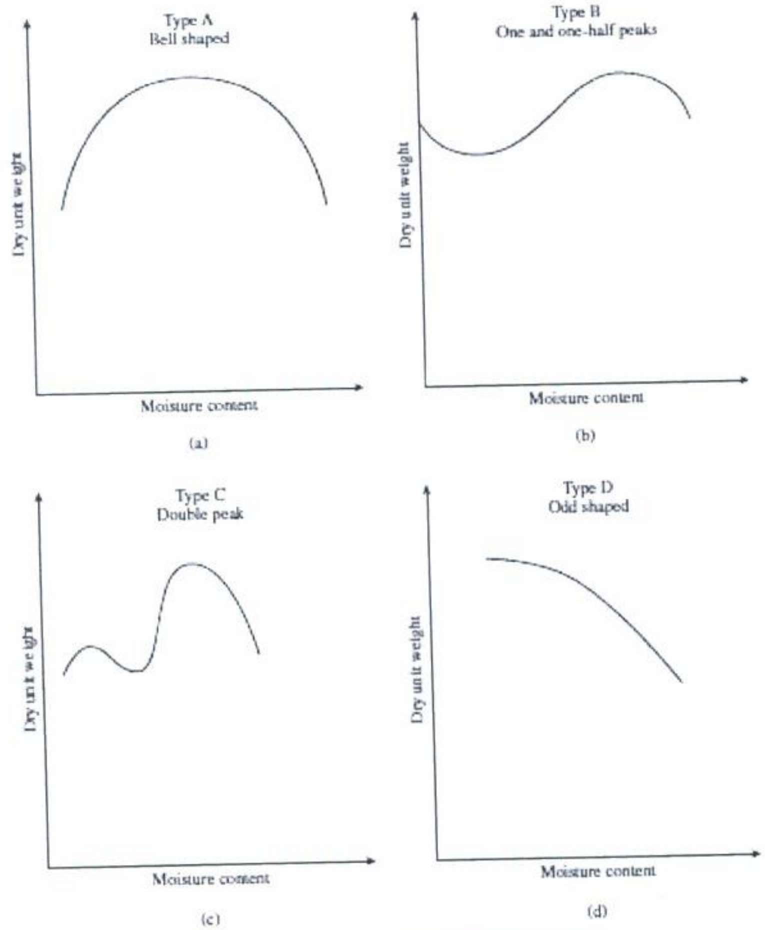


Figure 4.5 Various types of compaction curves encountered in soils

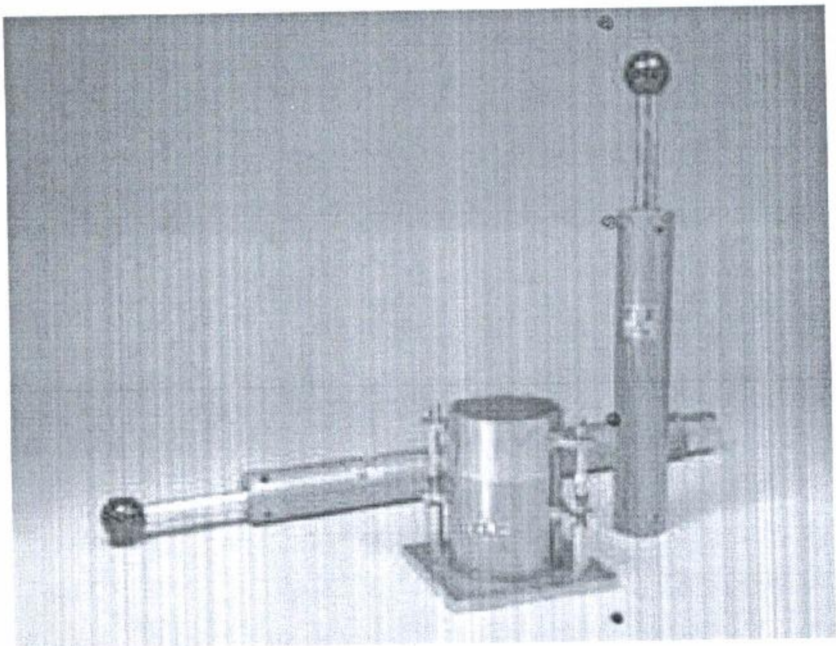


Figure 4.7 Hammer used for the modified Proctor test. (Courtesy of ELE International)

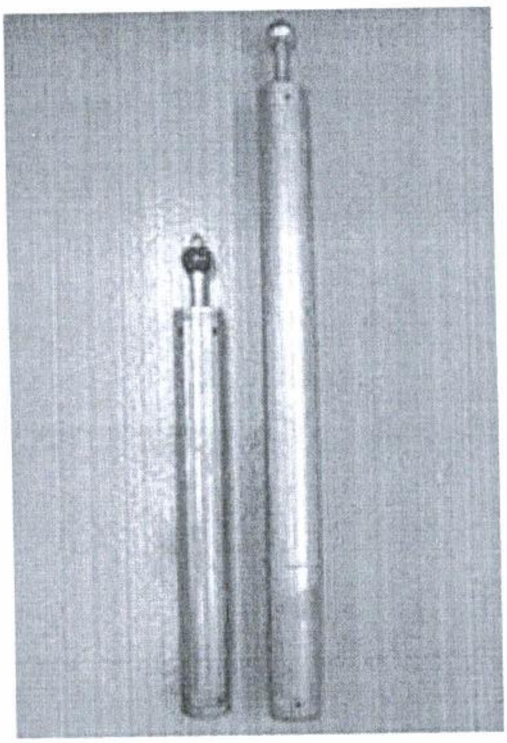


Figure 4.8 Comparision of standard (left) and modified (right) Proctor hammers (Courtesy of Braja Das)

Table 4.2 Specifications for standard Proctor test (Based on ASTM Test Designation 698)

Item	Method A	Method B	Method C
Diameter of mold	101.6 mm	101.6 mm	152.4 mm
Volume of mold	943.3 cm³	943.3 cm³	2124 cm³
Weight of hammer	24.4 N	24.4 N	24.4 N
Height of hammer drop	304.8 mm	304.8 mm	304.8 mm
Number of hammer blows per layer of soil	25	25	56
Number of layers of compaction	3	3	3
Energy of compaction	591.3 kN-m/m³	591.3 kN-m/m³	591.3 kN-m/m³
Soil to be used	Portion passing No. 4 (4.57 mm) sieve. May be used if 20% *or less* by weight of material is retained on No. 4 sieve.	Portion passing 9.5-mm sieve. May be used if soil retained on No. 4 sieve *is more* than 20%, and 20% *or less* by weight is retained on 9.5-mm sieve.	Portion passing 19-mm sieve. May be used if *more than* 20% by weight of material is retained on 9.5-mm sieve, and *less than* 30% by weight is retained on 19-mm sieve.

Table 4.3 Specifications for modified Proctor test (Based on ASTM Test Designation 1557)

Item	Method A	Method B	Method C
Diameter of mold	101.6 mm	101.6 mm	152.4 mm
Volume of mold	943.3 cm³	943.3 cm³	2124 cm³
Weight of hammer	44.5 N	44.5 N	44.5 N
Height of hammer drop	457.2 mm	457.2 mm	457.2 mm
Number of hammer blows per layer of soil	25	25	56
Number of layers of compaction	5	5	5
Energy of compaction	2696 kN-m/m³	2696 kN-m/m³	2696 kN-m/m³
Soil to be used	Portion passing No. 4 (4.57 mm) sieve. May be used if 20% *or less* by weight of material is retained on No. 4 sieve.	Portion passing 9.5-mm sieve. May be used if soil retained on No. 4 sieve *is more* than 20%, and 20% *or less* by weight is retained on 9.5-mm sieve.	Portion passing 19-mm sieve. May be used if *more than* 20% by weight of material is retained on 9.5-mm sieve, and *less than* 30% by weight is retained on 19-mm sieve.

Field Compaction

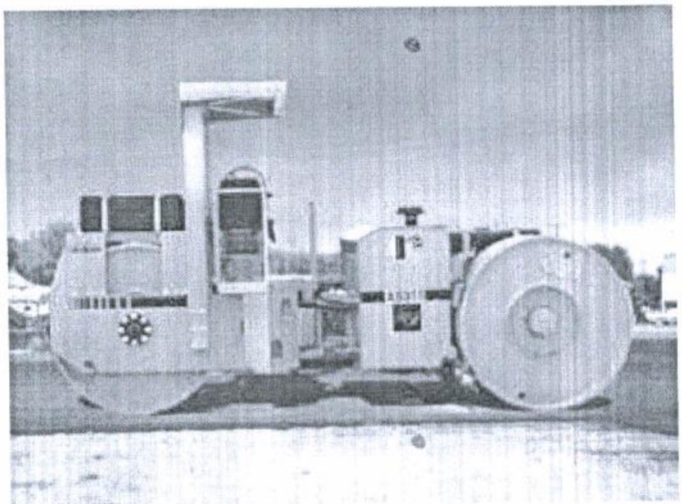
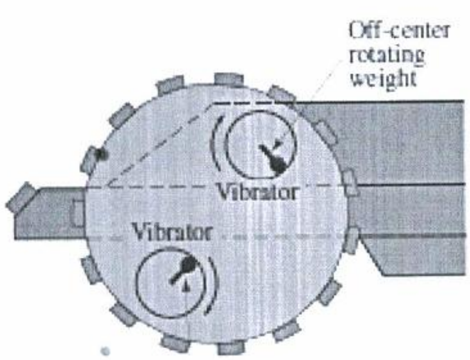


Figure 4.11 Smooth-wheel roller (Courtesy of Ingram Compaction, LLC)

Figure 4.13 Sheepsfoot roller (Courtesy of David A. Carroll, Austin, Texas)



PENGARUH PENAMBAHAN ENERGI PEMADATAN

- Kepadatan semakin tinggi
- Kadar air optimum semakin kecil.
- Semakin tinggi daya dukung CBR

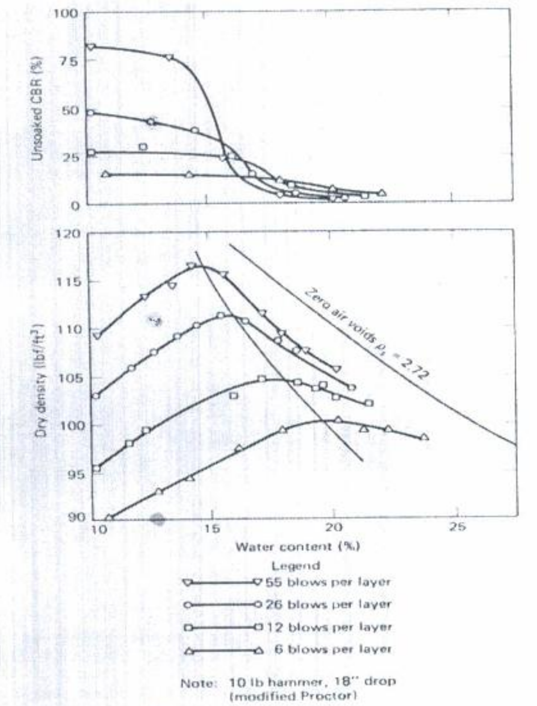


Fig. 5.8 Strength as measured by the CBR and the dry density versus water content for laboratory impact compaction (after Turnbull and Foster, 1956).

PENGARUH FREKWENSI GETAR TERHADAP KEPADATAN

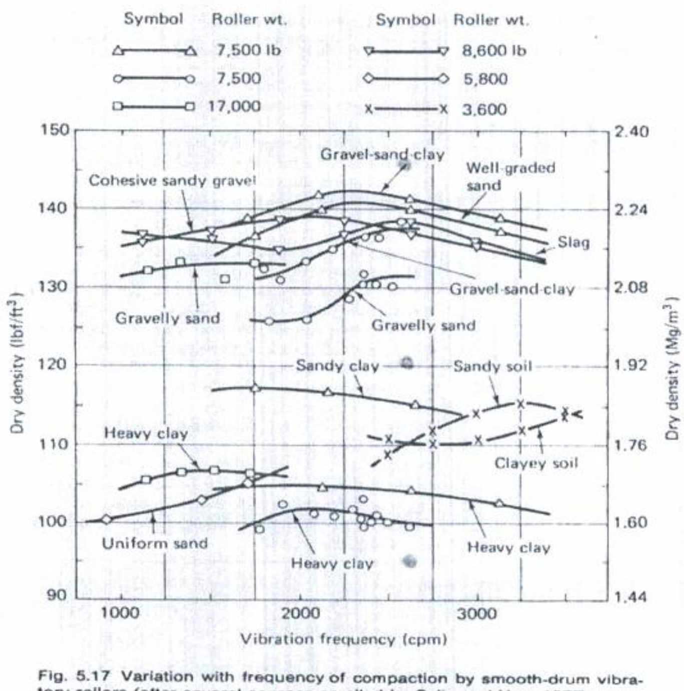


Fig. 5.17 Variation with frequency of compaction by smooth-drum vibratory rollers (after several sources as cited by Selig and Yoo, 1977).

PENGARUH KECEPATAN LINTAS PEMADAT TERHADAP KEPADATAN

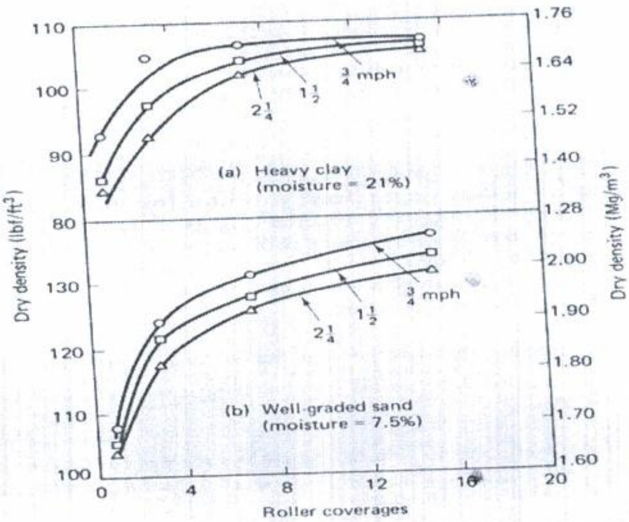


Fig. 5.18 Effect of roller travel speed on amount of compaction with 7700 kg (17,000 lb) towed vibratory roller (after Parsons, et al., 1962, as cited by Selig and Yoo, 1977).

DYNAMIC COMPACTION PADA TANAH GRANULAR

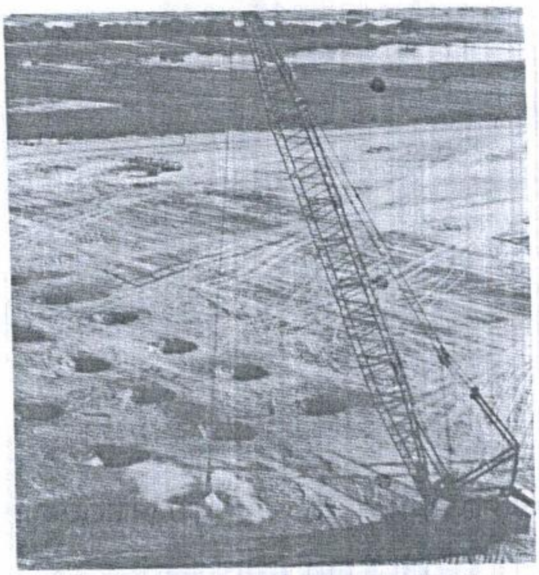
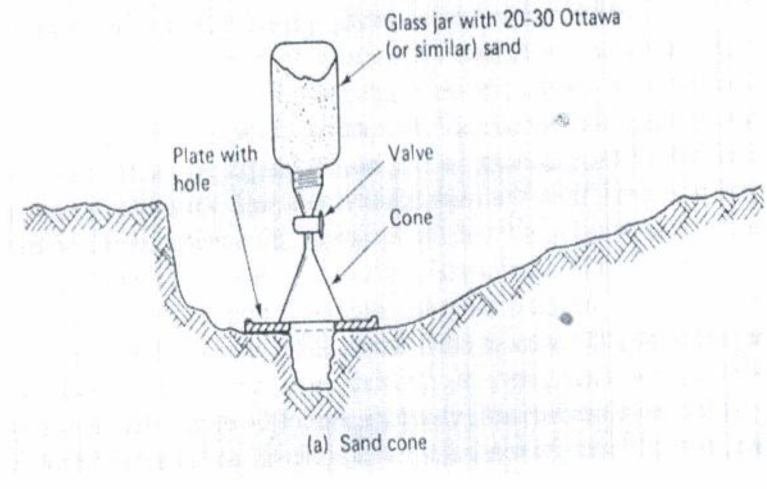


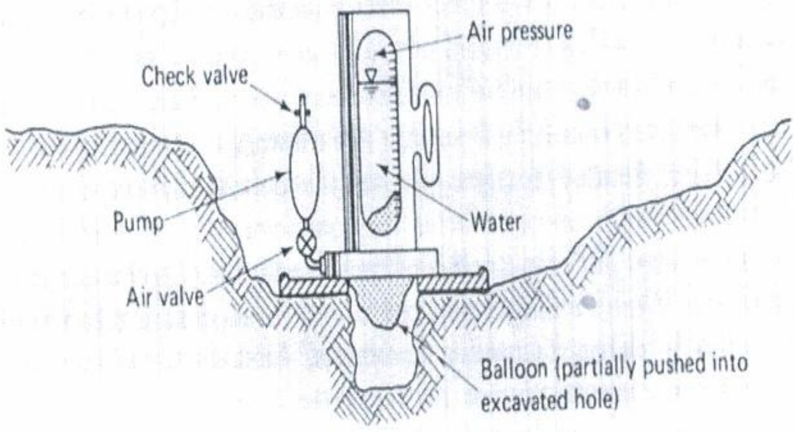
Fig. 5.21 Dynamic compaction at a site in Bangladesh. The 100 ton crane is dropping a 16 metric ton weight 30 m (courtesy of S. Varaksin, Techniques Louis Ménard, Longjumeau, France).

FIELD DENSITY BY SAND CONE TEST



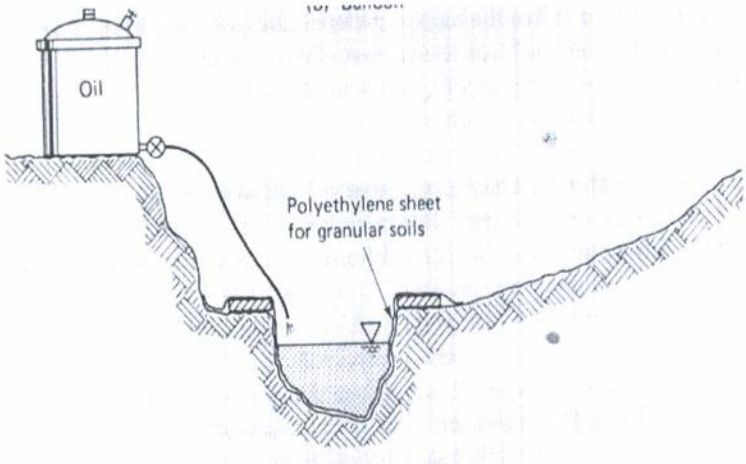
(a) Sand cone

FIELD DENSITY BY RUBBER BALLOON TEST



(b) Balloon

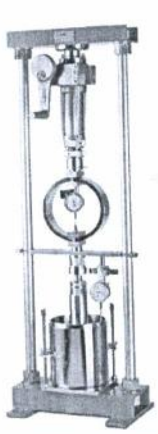
FIELD DENSITY TEST BY OIL METHODE

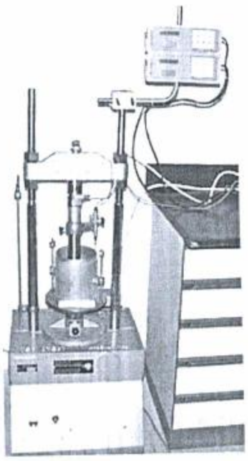


(c) Oil (or water) method

CALIFORNIA BEARING RATIO

California Bearing Ratio (CBR)

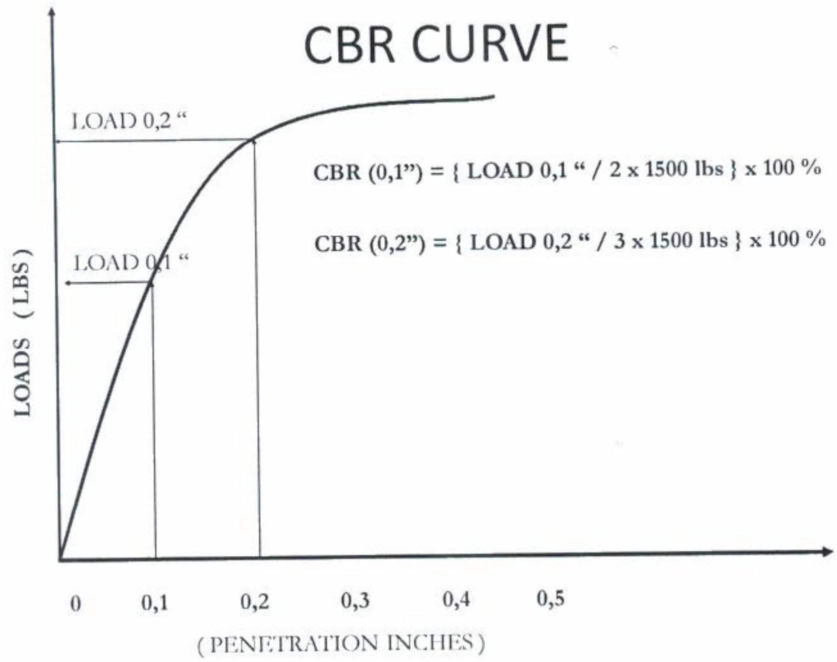




$$CBR = \frac{\text{Test unit stress}}{\text{Standard unit stress}} * 100$$

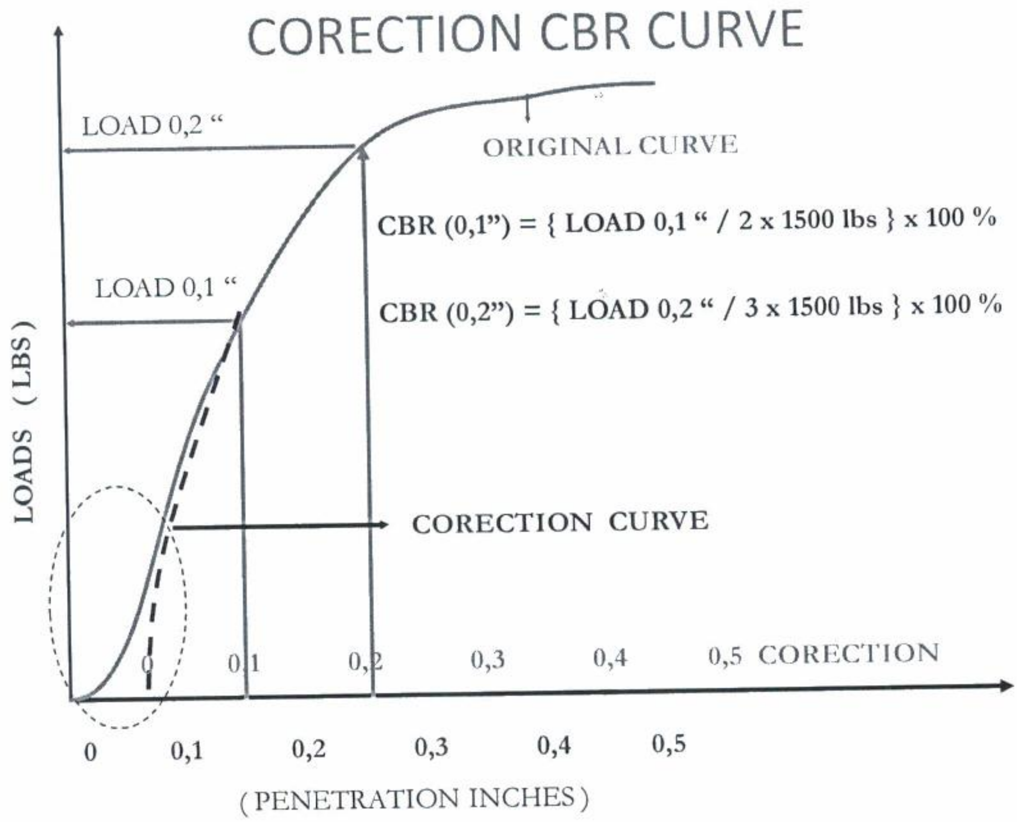
CBR	General Rating	Uses
0-3	Very poor	Sub-grade
3-7	Poor to fair	Sub-grade
7-20	Fair	Sub-base
20-50	Good	Base of sub-base
> 50	Excellent	Base

CBR CURVE

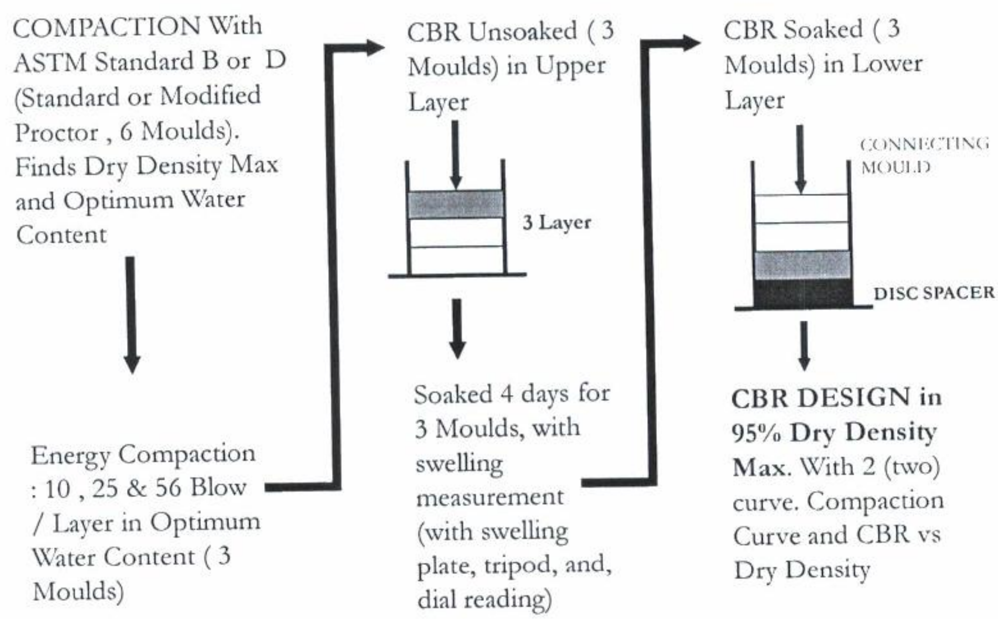


CBR VALUE

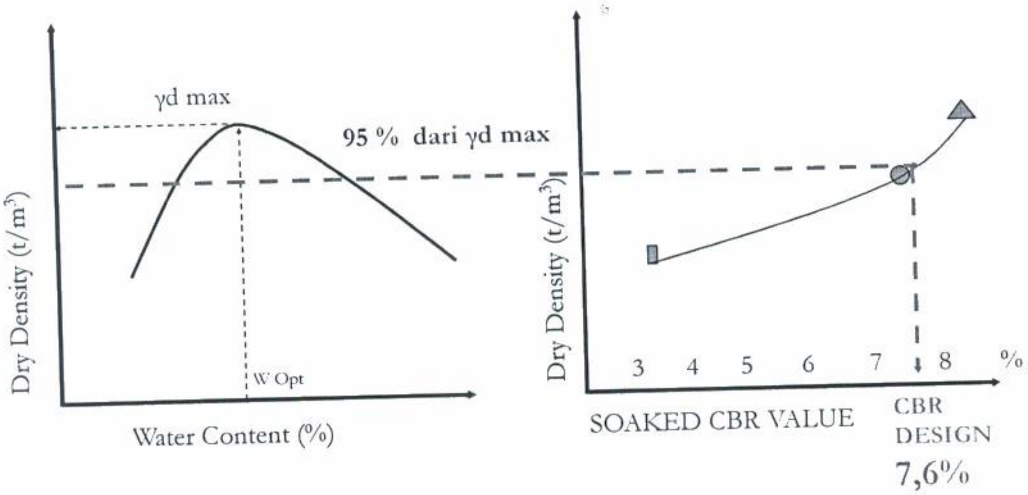
- Nilai CBR ditentukan dengan membandingkan CBR (0,1") dengan CBR (0,2")
- Jika CBR (0,1") > CBR (0,2"), maka CBR yang diambil CBR (0,1")
- Jika CBR (0,1") < CBR (0,2"), maka uji CBR diulang.
- Jika hasil uji ulang CBR (0,1") > CBR (0,2"), maka CBR yang diambil CBR (0,1")
- Jika hasil uji ulang CBR (0,1") < CBR (0,2"), maka CBR yang diambil CBR (0,2")



CBR DESIGN BY ASSHTO METHODE



CBR DESIGN BY ASSHTO METHODE



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