SOIL-WATER PROCESSES AND ENVIRONMENTAL SOIL PHYSICS

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1.0 SOIL AS A THREE PHASE SYSTEM

A natural system may consist of one or more substances and one or more phases. A system consisting of a single substance is called monophasic if in all its parts, the properties are similar. As example of such a system is a body of water consisting entirely of uniform ice. Such a system is homogeneous.

A system of uniform chemical composition may also be heterogeneous if the substance exhibits different properties in various regions of the system. A region inside a system which is internally uniform in physical properties is called a phase. A mixture of ice and water is chemically uniform, but physically heterogeneous consisting of two phases. The three phases in nature are solid, liquid and gaseous. A system including several substances can be monophasic or polyphasic.

In a heterogeneous system, the properties differ not only between one phase and another, but also between the internal parts of each phase or phases. Disperse systems are those in which atleast one of the phases is sub-divided into minute particles which together exhibit a very large surface area. Examples of a disperse system are the colloidal soils, emulsions, aerosols etc.

The soil is a heterogeneous, polyphasic, particulate, disperse and porous system in which the interfacial area per unit volume can be enormously large. The disperse nature of soil gives rise to such phenomena as swelling, shrinkages, dispersion, aggregation, adhesion, adsorption, ion exchange etc. The three phases of nature exist in the soil as well: the solid phase, consisting of soil particles, the liquid phase, consisting of soil water; and the gaseous phase, consisting of soil air.

The soil is thus an exceedingly complex system. Its solid matrix consists of particles differing in chemical and mineralogical composition as well as in size, shape and orientation. The mutual arrangement of these particles in the soil determines the characteristics of the pore spaces in which water and air are transmitted or retained. The water and air also vary in composition, both is time and in space.

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2.0 SOIL PARTICLES SIZES

Since the soils are formed by the weathering of rocks, the soil grains will consist of the basic rock forming minerals or their products after chemical alteration. If rocks are only physically degraded by the motion of ice, water or air, the soil grains will have the same composition as the parent rock; the size, shape and texture of the mineral grains will depend primarily on the history of degradation, transportation and deposition. Engineering studies are mainly concerned with the mechanical behaviour of soil masses.

The range of particle sizes in engineering soils is very large. Several systems are in vogue for the classification of particle sizes. As a general guide, individual sand-sized and coarser particles are visible by the naked eye, individual silt-sized particles are visible through an optical microscope while individual clay-sized particles can be seen only with electronic microscope. A standard particle size classification is given in Table-1.

Table 1: Particle sizes classification (BS 1377: 1975)

Description	**Diameter (mm)**
Cobbles	
Gravel	> 60
Coarse	60-20
Medium	20-6
Fine	6-2
Sand	
Coarse	3-0.6
Medium	0.6-0.2
Fine	0.2-0.06
Silt	
Coarse	0.06-0.02
Medium	0.02-0.006
Fine	0.006-0.002
Clay	< 0.002

2.1 Particle Size Distribution Curve

The distribution of particle sizes in a soil sample is found by sieving and by sedimentation. The coarse particles are separated by passing the sample through a set of sieves and weighing the fraction retained on each sieve. The smallest sieve has an aperture size of approximately 0.07 mm, corresponding roughly to the division between silt and sand sizes. Silt and clay particles are separated by sedimentation.

The distribution of particle sizes in a soil sample is shown as a grading curve, which is usually S-shaped (Fig.1). If the grading curve is flat and the soil sample contains a wide variety of particle sizes, the soil is known as “well graded”; if the curve is steep and one size predominates, the soil is "poorly graded". The grading of a soil often reflects its origin. Soils deposited by rivers or by wind tend to be poorly graded while boulder clays and glacial tills deposited by ice tend to be well graded.

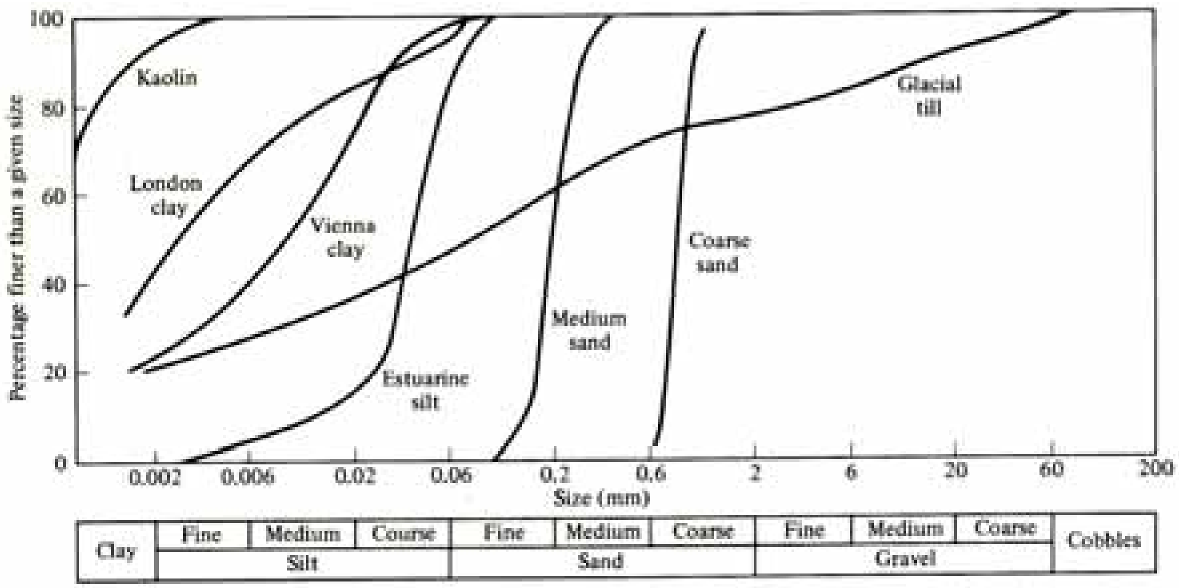

Fig. 1. grading curves plotted on a particle size distribution chart

Certain properties of clean sand have been related to particle diameters. The effective size of a sand is taken as the particle size corresponding to the 10% passing size from the grain size curve, and is indicated as D_{10}. It is this size that is related to permeability and capillarity. Another relation, the ratio D_{60}/D_{10}, is termed as **uniformity coefficient (Cu)**. It provides a comparative indication of the range of particle sizes in the soil. Sand having a wide range of particle sizes is considered well graded and has Cu value greater than 10. A uniform soil has Cu value less than 5.

2.2 Specific Surface

The specific surface of a soil can be defined as the total surface area of the particles per unit mass (a_m) or per unit volume of particles (a_v) or per unit volume of dry soil (a_b):

$$a_m = \frac{A_s}{M_s}$$

$$a_v = \frac{A_s}{V_s} \tag{1}$$

$$a_b = \frac{A_s}{V_t}$$

where, A_s is the total surface area of a mass particles M_s having a volume V_s and contained in a bulk volume V_t of the soil.

The specific surface is commonly expressed in terms of m^2 /gm, or m^2/cm^3. It depends upon the sizes of soil particles. In sand, the specific surface may be less than 1 m^2/gm, whereas in clay, the specific surface may be as high as several terms or hundreds m^2/gm.

The specific surface area also depends upon the shape of the particles. Flattened or elongated particles possess a greater specific surface area per unit mass than spherical or cubical particles of the same average mass. Since clay particle are generally platy, they contribute more to the overall specific area of the soil.

Therefore, the total specific surface of a soil, consisting of both external and internal surfaces, depends on the type of clay as well as on its total amount. The specific surface of soil is a highly pertinent property to provide basis for evaluating and predicting the soil behaviour. The specific surface often correlates with soil properties as cation. exchange capacity, availability of certain nutrients, swelling, retention of water at high suction, and certain mechanical properties such as plasticity and strength. The measurement of specific surface, though not common, may prove to be a more meaningful and pertinent index for characterizing a soil than the percentage of sand, silt and clay.

It can be shown that, for particles of nearly equal dimensions, such as most sand and silt grains, the measure of their particle sizes can be used to calculate approximate specific surface by:

$$a_m = \frac{6}{\rho_s} \frac{\sum d_i^2}{\sum d_i^3} \tag{2}$$

where, $6/\rho_s \approx 2.3$, if particle density ρ_s is taken as 2.65 gm/cm^3 . For platy particles, the approximate expression for specific surface is:

$$a_m \approx \frac{2}{\rho_s} \frac{0.75}{l} \text{ cm}^2/\text{gm} \tag{3}$$

if ρ_s = 2.65 gm/cm^3 . The specific surface area of clay can be estimated if the thickness of its platelets (l) is known.

3.0 STRESS STRAIN RELATIONSHIPS

When a body or mass is subjected to external loading, various combinations of internal normal and shear stresses are developed at different points within the body or mass. Generally, data concerning internal stress conditions are used to determine deformation or settlement of soil mass and to check the possibility of soils failures as its strength is exceeded. The stress at a point analysis is also applicable for determining the weakest plane or potential plane of failure in soils and indicating the magnitude of the stresses that act on this plane. This type of analysis has particular application for slope stability and foundation studies, since stability failures in soil masses occur when shear strength of the soil gets exceeded.

Stress is defined as an intensity of loading and strain as a measure of deformation. For soils, when the particle sizes may be relatively large, one has to make allowance for pressure that may exist in the pore water and which may modify the stress in the soil. Consideration of pore pressure is an important component of effective stress.

3.1 Normal Stress and Strain

Fig 2 (a) shows a small cube of material of cross sectional area ∂A and of height ∂Z. A load ∂F_N is applied across the cube in a direction normal to the area ∂A and along the element height ∂Z so that new length becomes ($\partial Z + \partial L$) as shown in Fig. 1(b). The normal stress and strain can be defined as:

$$\sigma = - \lim_{\partial A \to 0} \frac{\partial F_N}{\partial A}$$

$$\in = - \lim_{\partial Z \to 0} \frac{\partial L}{\partial Z} \qquad (4)$$

Tensile stresses occur only rarely in soils and so the negative signs have been introduced in above equations in order that compressive stresses and compressive strains are positive quantities.

3.2 Shear Stress and Strain

If a load ∂F_s is applied across the area in the plane of ∂A, the cube will distort as initially vertical fibres rotate as shown in Fig. 1(c). The shear stress τ and the shear strain ν are defined by :

$$\tau = - \lim_{\partial A \to 0} \frac{\partial F_s}{\partial A} \qquad (5)$$

$$\nu = - \lim_{\partial Z \to 0} \frac{\partial X}{\partial Z}$$

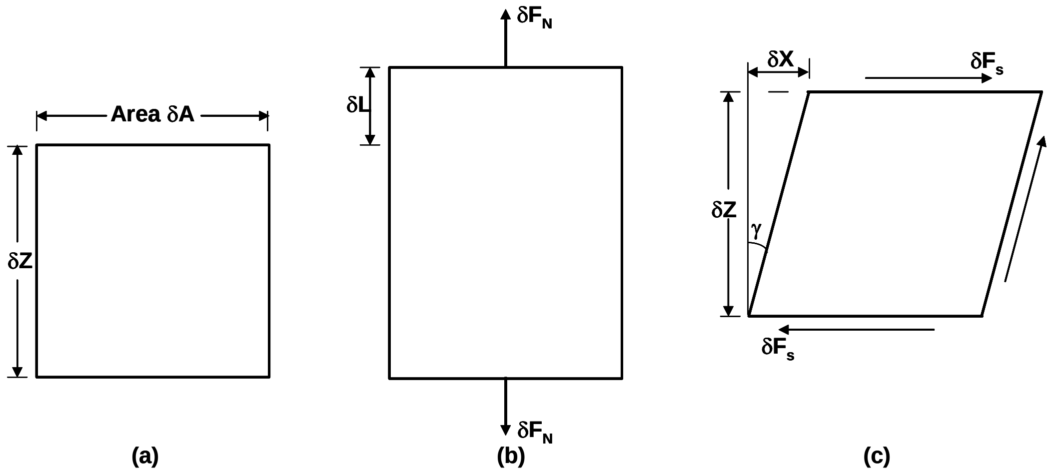

Fig. 2. (a) Stress and strain in an element (b) Normal stress and strain (c) shear stress and strain

3.3 Hook's Law

Hook's law of elasticity states that the strain is linearly proportional to the force causing the strain (that is the stress applied on this body). It is expressed as :

$$\frac{\text{Stress}}{\text{Strain}} = \frac{\sigma}{\in} = \text{constant} = E \qquad (6)$$

where, E is the modulus of elasticity or Young's Modulus in kg/cm^2 . The constant relationship between stress and strain applies only to stress below the proportional limit. E is a measure of stiffness of a body. The Young's Modulus helps in studying the behaviour of a material under load. For instance, it can be used to predict the amount by which a wire will extend under tension, or to predict the load at which a thin column will buckle under compression. For many materials, Young's modulus is a constant over a range of strains. Such materials are called linear and are said to obey Hook's law. The materials in which mechanical properties are the same in all directions are called as isotropic.

3.4 Stress at a Point

The stresses acting on any plane passing through a point consist of a normal stress (compression or tension) and a shearing stress. Depending on the type of external loading causing the stress conditions, it is possible for the shear or the normal stress, or both, to be zero on some planes. In soil problems, most external loadings are compressive. When the loading is compressive, normal stresses that develop on any plane would almost have a value other than zero; some shear stress would act on all planes with the exception of two planes, where it will be zero.

If the combination of normal and shear stresses acting on any two mutually perpendicular planes (orthogonal planes) are known, the combination of stresses acting on any other plane through the same point can be determined.

In analyzing stress at a point, it is convenient to assume an incremental element that represents the stress conditions at the point and show the known stresses acting on it (Fig. 3)

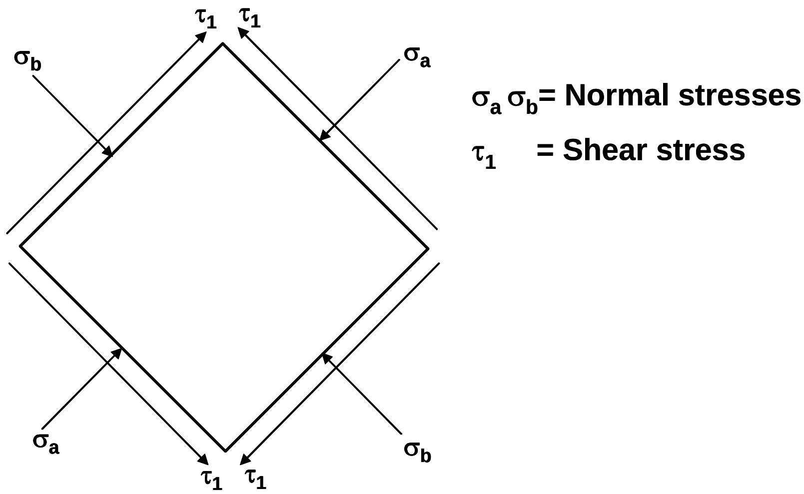

Fig. 3. Incremental element with stresses for analysis of 'stress at a point'

For equilibrium, the sum of forces acting in any direction must equate to zero, and the rotational moments about any axis caused by forces, similarly, must be equal to zero. To satisfy the later requirement, shear stress acting on orthogonal planes must be equal in magnitude.

Consider all the planes that can be passed through a point. On one particular plane, the shear stress will be zero, whereas the normal stress will be the maximum possible value of all the normal stresses acting through that point. On a plane perpendicular to the plane just referred to, the shear stress will also be zero, but the normal stress that acts will be the least of all the normal stresses acting through the point. These maximum and minimum normal stresses are called as principal stresses. The planes on which they act are called as principal planes. The shear stress on a principal plane is always zero.

The principal stresses act in the vertical and horizontal directions and are easily calculated. For instance, where the incremental element represents a point within a soil mass where the ground surface is horizontal, the vertical stress is due to the weight of soil over- burden at that point. The horizontal stress is proportionate to the vertical stress. Thus, the magnitude of principal stresses and the orientation of planes on which the stresses act are known.

If the major and minor principal stresses are σ_1 and σ_2, respectively, the magnitude of a normal stress σ_n and shear stress τ_n on any other plane can be determined.

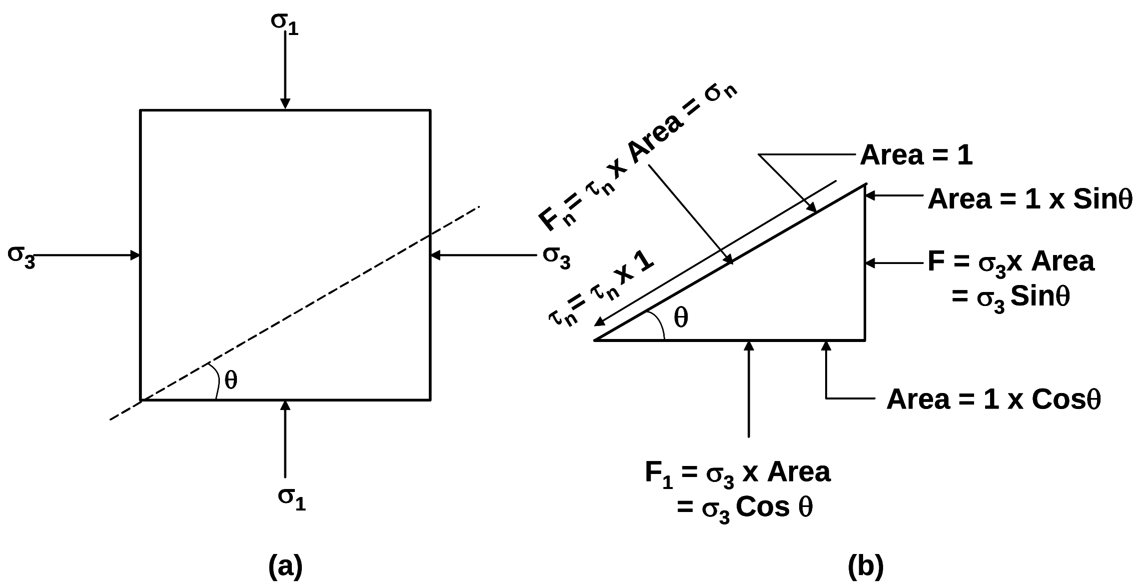

Fig. 4. Steps in determining stresses on a random plane through a point in terms of known principal stresses

Referring to Fig. 4(a) & (b) if the major and minor principal stresses are σ_1 and σ_3, respectively, the magnitude of a normal stress σ_n and shear stress τ_n on any other plane can be determined. Referring to the stressed element in Fig. 2(a) and letting the area on the cut plane be unity [Fig. 4(b)], it is apparent that the area of the plane on which σ_1 acts becomes 1 x Cos θ, and the area on which σ_3 acts becomes 1 x Sin θ. The total normal force on the cut plane is σ_n.

To determine σ_n and τ_n in terms of σ_1 and σ_3, the forces acting on the horizontal and vertical planes are resolved into components parallel and perpendicular to the cut plane. Summing the forces parallel to σ_n gives:

$$\sigma_n = \sigma_1 \cos\theta \cos\theta + \sigma_3 \sin\theta \sin\theta$$

$$= \sigma_1 \cos^2\theta + \sigma_3 \sin^2\theta \qquad (7)$$

$$= \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta$$

Summing the forces parallel to the cut plane in the direction of τ_n gives:

$$\tau_n = \sigma_1 \cos\theta \ \sin\theta + \sigma_3 \sin\theta \cos\theta$$

$$= (\sigma_1 - \sigma_3)\ (\cos\theta \sin\theta) \qquad (8)$$

$$= \left(\frac{\sigma_1 - \sigma_3}{2}\right) \sin 2\theta$$

From Eq.8 it is clear that the maximum shear stress will occur on a plane that is 45° from the major principal plane (θ = 45°) and will have a magnitude equal to ½ ($\sigma_1 - \sigma_3$). On this plane, the normal stress is always ½ ($\sigma_1 + \sigma_3$) .

The above discussion applies to a two dimensional stress analysis. In reality, for soil problems, the study of stress at a point involves three dimensional analysis. An incremental element with three principal stresses (σ1, σ2 and σ3) is shown in Fig. 5.

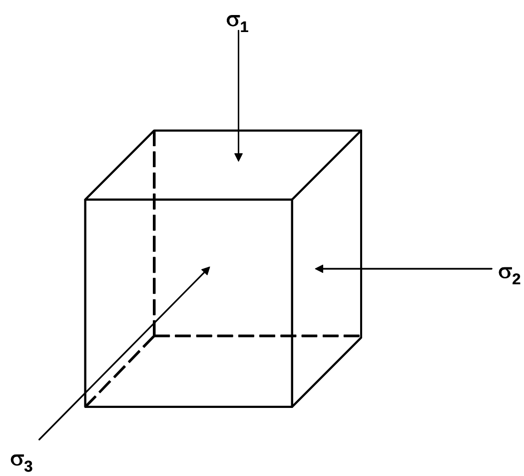

Fig. 5 Three dimensional incremental element showing principal stresses in mutually perpendicular directions

For many practical conditions of loading, the intermediate stress, σ_2 is equal to σ_1 or σ_3. The practical effects of intermediate stress are not clearly understood. It appears that there is some influence on the strength and stress-strain proportion of the material. However, for most practical problems, the degree of accuracy does not appear to be significantly diminished by neglecting the effect of the intermediate stress and working with the simpler two dimensional analysis.

3.5 Mohr's Circle

The values of normal stress and shear stress corresponding to differing values of θ can be determined by Eqs (1) and (2). If the combination of normal and shear stress resulting from each value of θ is plotted as a point on a coordinate system where the horizontal axis represents normal stress and the vertical axis represents shear stress, the locus of many plotted points will form a circle. Therefore, by working with simple properties of a circle, a graphical method for computing the normal and shear stresses on any plane, given the stresses on orthogonal planes, is easily developed. This method is referred to as the Mohr's circle for determining stresses.

In using Mohr's circle, a sign convention is required. For soils related problems, compressive stresses are conventionally assumed to be positive and shearing stresses that provide a clock wise couple are also considered positive.

Consider Fig. 6 (a), where σ_1 and σ_3 are the major and minor principal stresses. There is no shear stress acting on the planes. To construct Mohr's circle for this combination locate σ_1 = 2500 and σ_3 = 500 on the X-axis (since shear stress is zero).

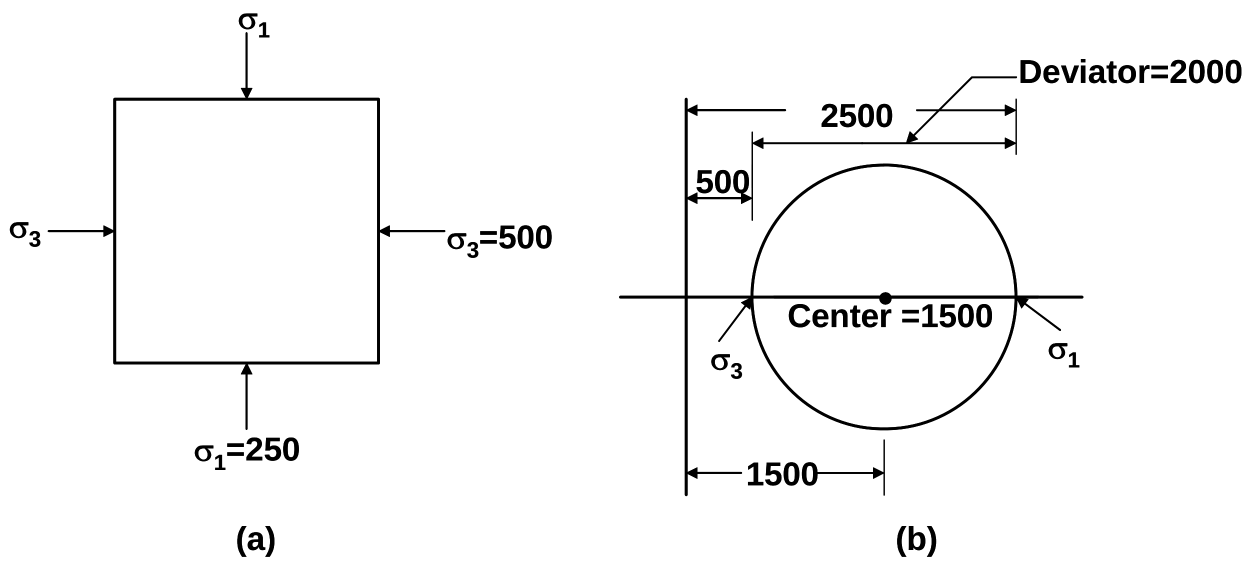

Fig. 6. Representation of principal stresses acting at a point (a) and the related Mohr's circle plot (b)

Next establish the location for the centre of the Mohr's circle, knowing that diameter of the circle has a value equal to σ_1 - σ_3. The numerical difference between σ_1 and σ_3 is called deviator stress. For this example deviatior stress is 2500-500 = 2000 (Fig. 6b)). The radius of the circle is then 1000, and the centre of the circle will plot at σ_3 + radius = 1500. After the centre of the circle and the diameter are established, the circle can be constructed; thereafter the stress combination on any plane can be determined.

The angular measurement to plane on which stresses are to be calculated, is marked on the Mohr's circle by starting with the point representing the major principal stress. If the angle to be measured is formed by two radii, the angle is measured at the centre of the circle. Because of the properties of a circle, the central angle on the circle must be twice the value of the angle θ measured at the original element.

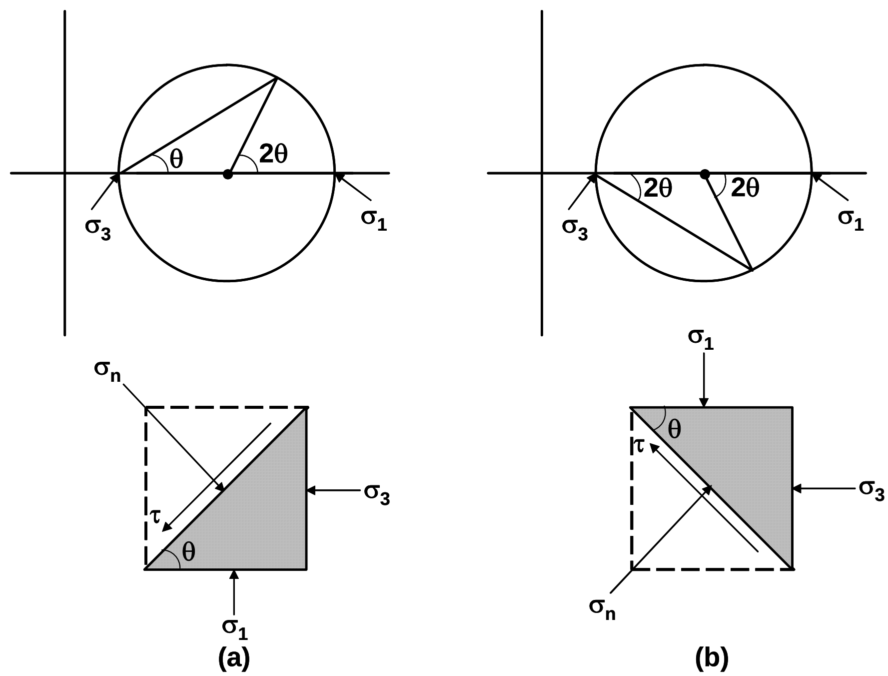

Fig. 7. Relation of portions of planes on the incremental elements to point on the Mohr's circle (a) angle measured counter clockwise (b) angle measured clockwise

Referring to Fig.7, the direction of measurement from the principal plane to the cut plane on the element, clockwise or counter clockwise, is the same direction as to be used on the Mohr's circle.

The normal stress coordinate is the value of the centre of the Mohr's circle plus or minus the horizontal projection of the of the radius. As shown in Fig.6, the normal stress would be:

$$\sigma_n = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \text{Cos } 2\theta \tag{9}$$

The shear stress coordinate is the vertical projection of the radius,

$$\tau = \frac{\sigma_1 - \sigma_2}{2} \text{ Sin } 2\,\theta \tag{10}$$

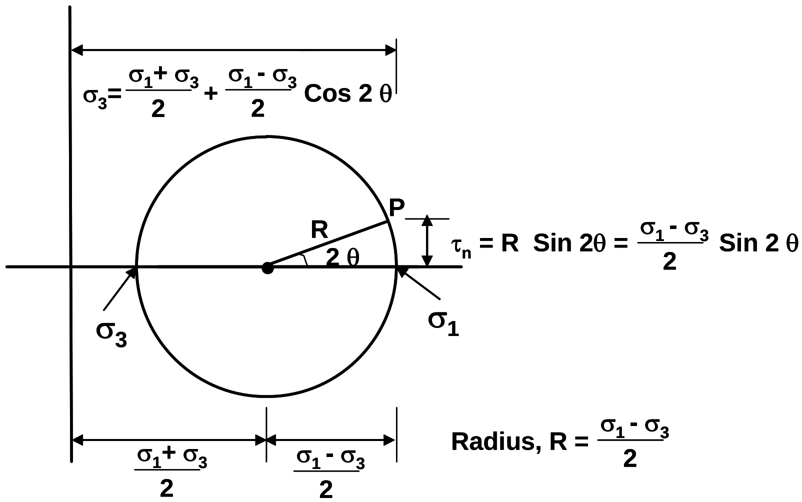

Fig. 8. General equations for normal and shear stresses on the Mohr's circle

As an illustrations, with σ_1 = 2500 and σ_3 = 500, assume that it is desired to find the stresses acting on a plane that is 60° clock wise from the major principal plane (Fig. 8).

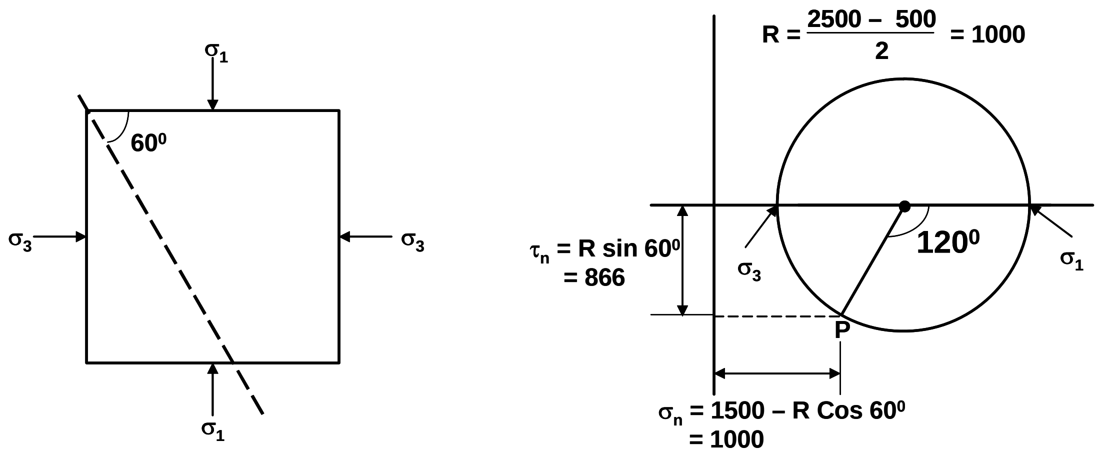

Fig. 9. Use of Mohr's circle to determine stresses on a plane 60° clockwise from the major principal plane

On the Mohr's circle, the central angle measured is 2θ = 120°. Mathematically, from Mohr's circle, the shear stress τ_n equal R x Sin 60° or 1000 x 0.866 = 866. The normal stress, σ_n is 1500 – R x Cos 60° or 1000 as shown in Fig. 9.

If the stress combination is to be determined on a plane measured with reference to the minor principal plane, the related angular measurement on the Mohr's circle is made from the minor principal stress, σ_3.

The Mohr's circle method can similarly be used for determining principal stresses if the stress conditions for any two orthogonal planes other than the principal planes are known. For an element where the normal and shear stresses are as shown in Fig.10, the Mohr's circle construction proceeds as follows:

1. On the Mohr's circle coordinate, establish the points σ_a and σ_b.
2. Connect these points, thereby establishing the diameter of the Mohr's circle, and determine the coordinate for the centre of the circle.
3. Calculate the value for the radius of the Mohr's circle.
4. The value of σ_1 (the major principal stress) is the circle radius added to the value established for the centre of the circle. The value of σ_3 (the minor principal stress) is the radius subtracted from the coordinate for the centre of the circle.
5. The angle 2 θ on the Mohr's circle is obtained from simple geometry.
6. The orientation of the principal planes with respect to the original element is shown in Fig. 8.
7. After establishing principal planes, stresses on any other plane can be determined.

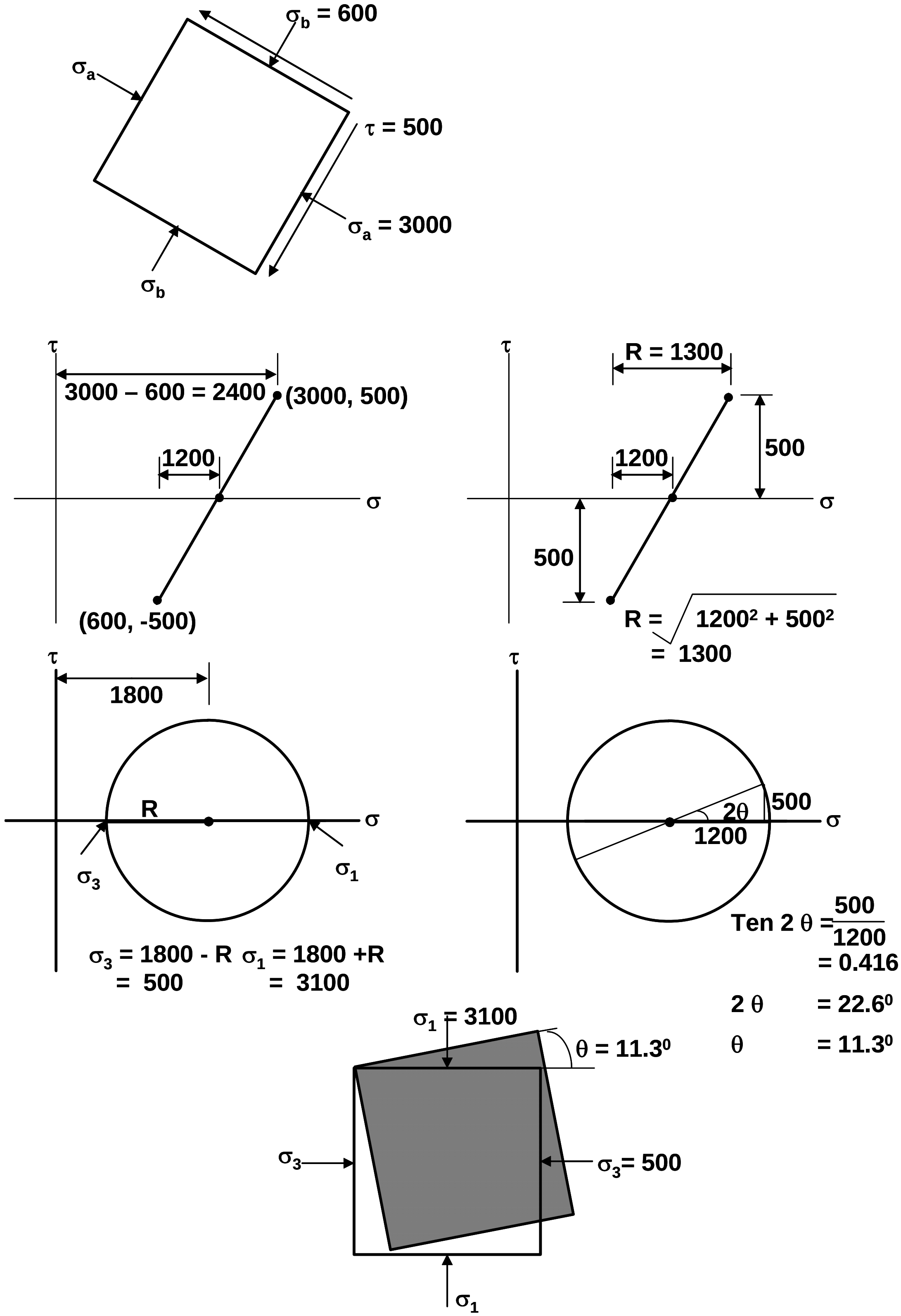

Fig. 10. Procedure for applying the Mohr's circle analysis to solve for principal stresses

4.0 SOIL CONSOLIDATION

The weight of any structure supported on the earth will result in stresses being imposed on the soils below the level of the base of the structure. The deformation in the soil occurring due to these stresses causes changes in the volume of the soil, which may cause settlement of the structure. Settlement is caused both by soil compaction and lateral yielding of the soils located under the loaded area. The settlement factor that is most significant for cohesive soils is different from the factor that is most significant for cohesionless soils. In cohesive soils, compression is the cause of most settlement, where as for cohesionless soils lateral yielding is the more significant cause.

4.1 Compressibility

Compressibility is the term applied to one dimensional volume changes that occur in cohesive soils that are subjected to compressive loading. This property of fine grained soils can be determined in the laboratory by consolidation test.

In soil compression, volume decrease is principally due to decrease in the void spaces between soil particles. Practically, there is no decrease in the actual volume of soil particles. The time required for volume changes to occur differs significantly for the coarse (cohesionless) and the fine grained (cohesive) soils. The cohesionless soils experience compression quickly after a loading is imposed. Conversely, clay soils require a significant period before full compression under an applied loading results. Relating compression with the time period necessary for the compression to occur (time rate of compression) is called consolidation.

4.2 Consolidation in Clay Soils

Compression of clay soils occurs gradually when new loading is applied. This happens due to the low permeability of these fine grained soils. For compression to occur in a soil with water filled in pore spaces, water in the voids must first be expelled for the decrease in void spaces to occur. As trapped water escapes, the pressure caused by the external loading is transferred from this water to the soil. The process of load transfer to the soil as pore water escapes is the consolidation process.

The rate of consolidation for a stratum of clay soil is affected by many factors such as :

1. The permeability of soil
2. The extent or thickness of the soil and the distance that pore water in the soil must travel to escape.
3. The void ratio of the soil
4. The ratio of new loading to original loading
5. The compression properties of the soil

The consolidation properties of soil can be defined by a term called ‘coefficient of consolidation C_v” which indicates how rapidly or slowly the process of consolidation takes place. The coefficient of consolidation is:

$$C_v = \frac{K\,(1+e)}{a_v\,\gamma_w} \tag{11}$$

where,
C_v = Coefficient of consolidation,
K = Coefficient of permeability (cm/sec),
a_v = Coefficient of compressibility,
γ_w = Unit weight of water (gm/cm^3), and
e = Void ratio.

It is a common practice to determine C_v directly from laboratory consolidation test. The consolidation test is a compression test, but with added provision of time reading for each applied load.

As previously indicated, the consolidation process requires expelling of pore water and adjusting of soil particles to a denser condition. Full consolidation (100 percent), or a condition of equilibrium is assumed to have occurred when the full loading of the structure is carried by the soil particles (no excess pressure in the pore water) and the total expected compression has taken place. The compression resulting after trapped pore water under pressure has escaped is "primary consolidation". Some additional long term compression termed as secondary compression also takes place as soil particles continue to adjust under the applied loading.

For given field conditions, where the distance that trapped pore water must travel to escape from the compressible layer (H_{dr} in Fig. 9) and C_v are known, the time period t required for a given percentage of consolidation to occur is:

$$t = \frac{T_v\,H_{dr}^2}{C_v} \tag{12}$$

where,

T_v = Time factor for consolidation due to vertical drainage,
H_{dr} = Drainage distance for escaping pore water, and
C_v = Coefficient of consolidation.

The value T_v is a time factor that is constant for a given percentage of consolidation and for a given type of sub-surface condition.

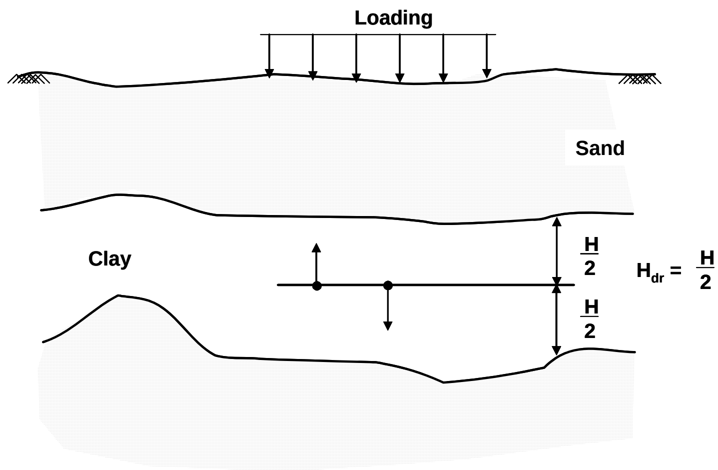

Fig. 11. Double drainage conditions in consolidation

For the condition of 'double drainage' (Fig.11), where pore water can move downward and upward to escape, the value of T_v for a given percentage of consolidation (U per cent) can be determined from Fig 12.

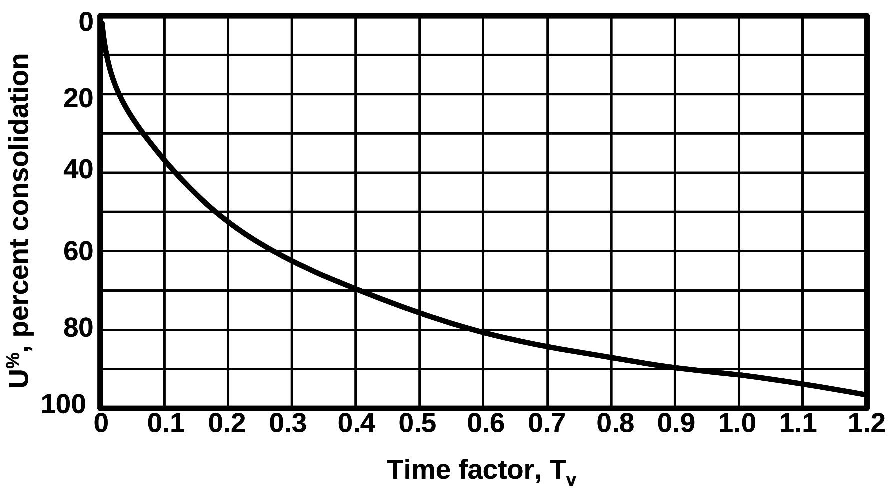

Fig. 12. Variation of time factor T_v with percentage of consolidation

For known Cv and Hdr, the time required for U% of consolidation to occur or the percentage of consolidation in a given time can be estimated by Eq(12). At the sites with compressible or weak sub-soils, where a structure is proposed, the soil must be improved so that its strength is adequate and the compressibility reduced to support the structure. The improvement of poor soils may possibly be accomplished by utilizing a surcharge programme. Surcharging of a soil, in its simplest form, is merely imposing an external loading for a long duration to cause desirable changes in the soil before a structure is erected and supported on the soil.

5.0 INFILTRATION

Infiltration rate is defined as the volume flux of water entering the soil profile per unit of soil surface area and time. Horton (1940) defined infiltration capacity as the maximum rate at which a given soil in a given condition can absorb rain as it falls. It is the infiltration capacity of the soil that determines for a given storm, the amount and time distribution of rainfall excess that is available for runoff and surface storage. More recently, Hillel (1971) defined the term infiltrability to designate infiltration flux when water at atmospheric pressure is made freely available at the soil surface. This single word replacement avoids the extensity-intensity contradiction in the term infiltration capacity and allows the use of the term infiltration rate in the ordinary literal sense under any set of circumstances, whatever the rate or pressure at which the water is supplied to the soil. For example, the infiltration rate can be expected to exceed infiltrability whenever, water is ponded over the soil to a depth sufficient to cause the pressure at the surface, to be significantly greater than atmospheric pressure. On the other hand, if water is applied slowly or at a sub-atmospheric pressure, the infiltration rate may well be smaller than the infiltrability.

5.1 Factors Affecting Infiltration

5.1.1 Soil properties

One of the most important variables controlling infiltration is the hydraulic conductivity (K_S). However, during early stages of infiltration, soil structure or pore size distribution has a significant effect. As the soil behind the wetting front approaches saturation, the hydraulic gradient approaches unity and the hydraulic conductivity begins to control the flow rate. In general, wider is the range of pore sizes, the more gradual is the change in the infiltration rate. Fig.13 shows the infiltration rates of six different soils as predicted from numerical solution of Richards equation for vertical infiltration from a ponded surface into deep homogenous profiles.

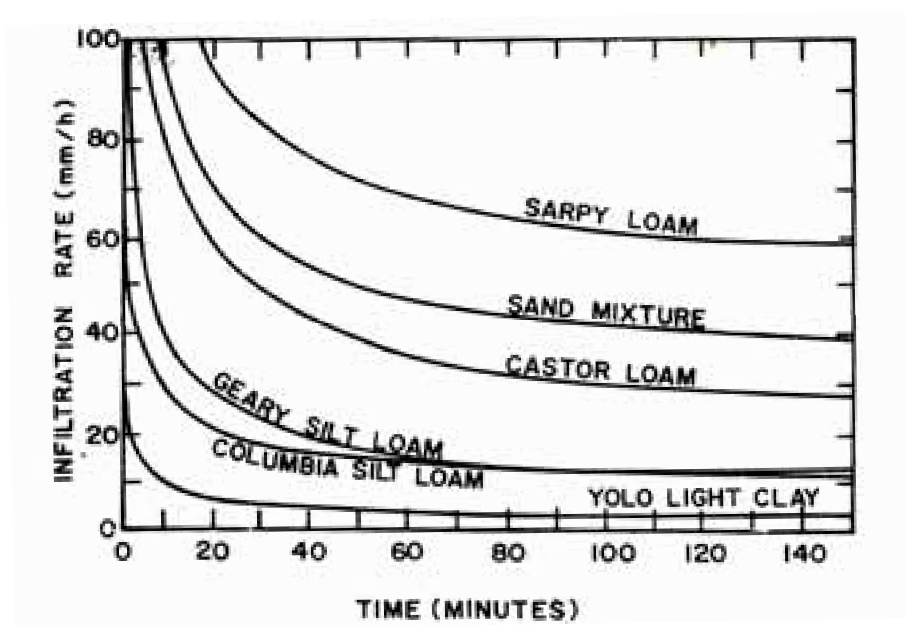

Fig. 13. Predicted infiltration rates for deep soils with a shallow ponded surface

5.1.2 Initial water content

Higher the initial water content, lower will be the infiltrability owing to smaller suction gradients. If infiltration continues indefinitely, the infiltration rate eventually approaches k_s regardless of the initial water content. (Fig. 14).

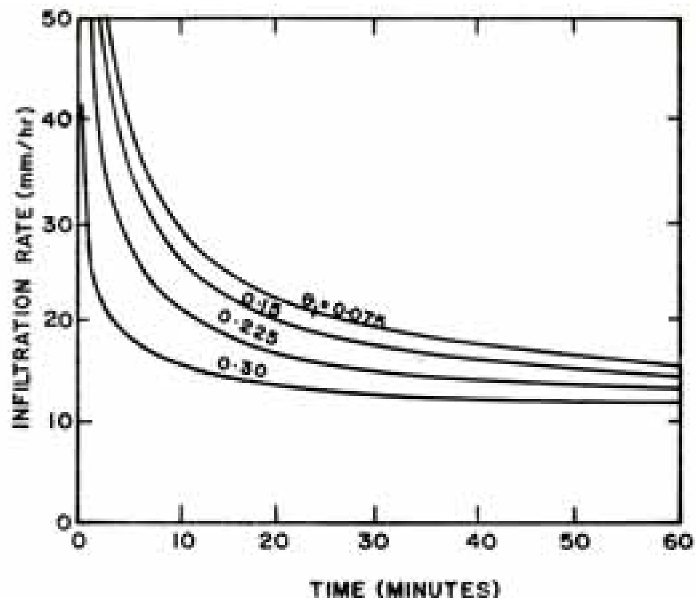

Fig. 14. Infiltration rate for different silty loamy soils for different initial soil moisture contents

5.1.3 Rainfall rates

Infiltration rate is a function of both rainfall rate and soil conditions. If the rainfall rate, R is less than K_s for a deep homogenous soil, infiltration may continue indefinitely at a rate equal to the rainfall rate without ponding at the surface. The water content of the soil in this case does not reach saturation at any point but approaches a limiting value which depends on rainfall intensity. However, for soils with restricting layers, infi!tration will not always continue indefinitely without surface ponding when $R < K_s$.

When R > Ks, the infiltration rate is initially controlled by the application rate till it falls below R. Subsequently, surface ponding begins and water becomes available for runoff.

5.1.4 Surface sealing and crusting

The changes in the hydraulic properties at the soil surface during the application of water strongly affect the infiltration rate. Horton (1939) attributed the exponential decay of infiltration rate with time to the slaking of aggregates and swelling of colloids which progressively seal the soil surface. Edward and Larson (1969) applied the theory of soil water movement to investigate the influence of surface seal development on infiltration of water into a tilled soil. Hillel and Gardener (1969,1970) while evaluating the effect of surface sealing on steady state and transient infiltration processes observed that infiltration into crusted profile may be approximated by assuming that water enters the soil layer below the crust at a nearly constant suction, the magnitude of which depends on the hydraulic properties of the underlying soil. Morin and Benyamini (1977) concluded that the crust formed by raindrop impact is the dominant factor influencing the infiltration capacity (Fig. 15).

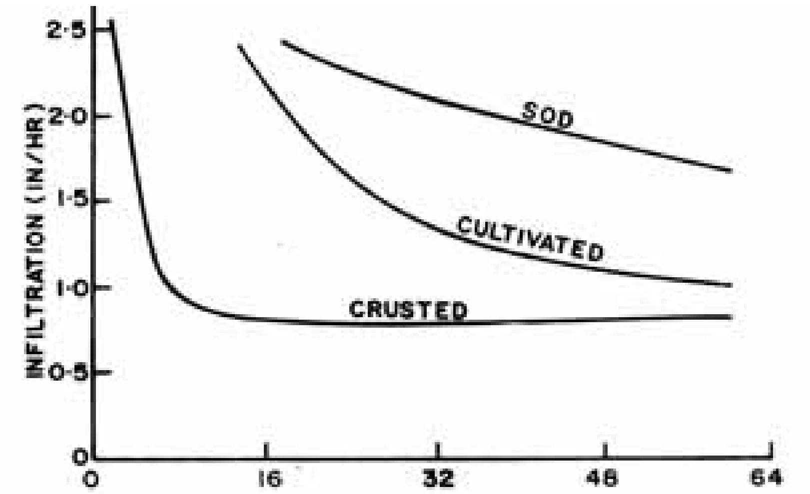

Fig. 15. Effect of surface sealing and crusting due to rainfall impact on infiltration rate

5.1.5 Layered soils

Normally, infiltration processes occur in soils which are uniform neither in texture nor in initial wetness. Takagi (1960) analysed steady state downfall of water through a two layered profile and found that if the upper layer is less pervious than the lower, negative pressures develop in the lower layer and these can remain constant throughout a considerable depth range.

5.1.6 Movement and entrapment of soil air

The assumption of constant air pressure during derivation of Richards equation is rarely satisfied in actual practice. It is assumed that because of small viscosity of air relative to that of water, air can escape through large pores that remain partially open during infiltration. However, in many cases the trapped air causes air pressure build up thereby reducing the infiltration rate. Slack (1978) observed that a total air entrapment of 10 percent can bring 80 to 90 percent reduction in hydraulic conductivity and, therefore, effects infiltration rate.

5.2 Infiltration Models

Numerous formulations have been proposed over the years to express infiltrabilty as a function of time or of the total quantity of water infiltrated into the soil. Some of them are entirely empirical while other are theoretically explained.

5.2.1 Empirical infiltration models

The approximate infiltration models have been developed by applying the principles governing soil water movement for simplified boundary and initial conditions. The parameters for such models are determined from soil water properties. For models which are strictly empirical in nature, parameters must be obtained from measured infiltration data or estimated using more approximate procedures.

(i) Kostiakov's Equation

Kostiakov (1932) proposed the simple form of infiltration equation given by:

$$f_p = Kt^{-n} \tag{13}$$

where,

f_p = Infiltration capacity of the soil, mm/sec;
t = Time after infiltration started, sec; and
K&n = Constants which depend upon the soil type and initial moisture content of the soil.

This equation implies that infiltration rate approaches zero as time tends to infinity rather than a constant non-zero f_p which is not correct. Moreover, the parameters of this equation must be evaluated from experimental data and thus have no physical interpretation.

(ii) Horton’s Equation

Horton (1939,1940) presented a three parameter infiltration equation which may be written as:

$$f_p = f_c + (f_o - f_c)e^{-kt} \tag{14}$$

where,

f_c = Final constant rate of infiltration capacity;
f_o = Initial rate of infiltration capacity;
f_p = Infiltration capacity at any time, t; and
K = A constant depending primarily upon soil and vegetation.

It represents the rate of decrease of infiltration capacity during the storm event. At t = 0, the infiltrability is not infinite but takes on the finite value f_o . The constant K determines how quickly f_p will decrease from f_o to f_c . This equation is also integrable to give cumulative infiltration F at any time, t. Though this equation has the advantage of simplicity, the assumption that continuous ponding occurs at the surface or that the rainfall rate is always greater than the infiltration rate does not, of course, always exists during a storm. Moreover, the three constants must be evaluated experimentally.

(iii) Smith's Equation

The limitation of both the Horton and Kostiakov equations is that they ignore the preponding infiltration and consider infiltration as a single stage event. Smith (1972) modified the Israelson and Hansen form of Kostiakov equation for two stage infiltration events by adding an empirical parameter whose value is equal to zero for initially ponded infiltration as:

$$f_p = f_c + C(t-t_o)^d \qquad (15)$$

where, t_o is an empirical constant which is equal to zero for initially ponded infiltration; and C & D are constant determined as the intercept and slope, respectively of a log-log plot of $(f_p - f_c)$ versus $(t-t_o)$.

(iv) Holtan's Equation

Holtan (1961) proposed an empirical equation for infiltration based on a storage concept as :

$$f_p = f_c + a\,(M-F)^n \qquad (16)$$

where, f_c is the equilibrium infiltration rate associated with the gravity force at field capacity soil moisture content and F is the cumulative infiltration up to a given time period since the beginning of infiltration. Holtan specified M as the water storage capacity of the soil but the meaning of M for a soil without an impending stratum was not made clear. Moreover, it is only defined for the range $0 < F < M$ since $f_p = f_c$ can only occur at the single point F = M. When F exceeds M, then the quantity (M-F) becomes either positive and increasing, negative and decreasing or imaginary depending upon whether n is even, odd or fractional, respectively. Thereafter, in addition to meeting the condition $0 < F < M$ on equation to be complete, one must also state,

$$f_p = f_c \text{ for } F > M \qquad (17)$$

Hence, the Holtan expression must be recognized as the two form mathematical specification represented by Equations (16) and (17). 'a' and 'n' are the parameters determined experimentally from infiltrometers plot data. 'n' was found to be 1.4 and 'a' was found to vary between 0.2 and 0.8 for the soil cover complexes studied and for various soil moisture contents at time, t.

The advantage of Holtan's model over Horton's model is that only one parameter *'a'* need to be estimated as compared to two, f_o and K in the Horton's model.

5.2.2 Physically based infiltration models

The models based upon soil physics approach are more complicated than the empirical models. They are derived by application of the principles of continuity of mass and soil water movement with certain simplifications and assumptions.

(i) The Green and Ampt approach

Green and Ampt (1911) proposed an approximate model of infiltration utilizing Darcy's Law originally from a ponded surface into a deep homogenous soil with a uniform initial water content. The equation may be written as :

$$f_p = K_h (Z + S_e + D)/D \tag{18}$$

where,

f_p = Infiltration capacity, mm/day;
K_h = Hydraulic conductivity of the transmission zone, mm/sec;
Z = Depth of water ponded on the surface;
S_e = Effective suction at the wetting front, and
D = Distance from the surface to the wetting front.

Considering cumulative infiltration as F = MD and neglecting ponding at the surface (Z=0), Equation (18) may be written as :

$$f_p = K_h + K_h MS_e/F \tag{19}$$

where, M denotes the initial soil water deficit which is the difference between initial and final volumetric water contents. The Green and Ampt's models is based upon the following assumptions:

1. There exists a distinct and precisely definable wetting front.
2. Tension or suction at the wetting front remains essentially constant with time and depth.
3. Behind the wetting front, the soil is uniformly wet and has constant hydraulic conductivity.
4. The wetting front is viewed as separating a uniformly wetted infiltrated zone from the non-infiltrated zone. This in turn supposes that k_h versus volumetric moisture content θ relationship is discontinuous at the wetting front.

These assumptions simplify the flow equation and make it amenable to analytical solution. In addition to uniform profiles for which it was originally derived, the Green and Ampt's equation has been satisfactorily used for profiles where hydraulic conductivity increases with depth (Bouwer, 1976), for soils with partially sealed surfaces (Hillel and Gardener, 1970) and for non-uniform initial water contents (Bouwer, 1969). Morel Seytoux and Khanji (1974) observed that Equation (19) may be used for simultaneous movement of both water and air. Whisler and Bouwer (1970) compared the Green and Ampt and Philip equations with field data and concluded that Green and Ampt formula was best for practical reasons.

(ii) Determination of parameters in the Green-Ampt equation

Bouwer (1960) suggested that hydraulic conductivity parameter in Green and Ampt model may be taken as half of the saturated value because of the entrapped air. He further suggested that effective suction at the wetting front can be approximated as one half of the air entry value (Bouwer, 1969).

Brakensiek (1977) found that average suction can be related to the water entry suction which is approximately equal to one half of the air entry value (bubbling pressure). Brakensiek and Onstad (1977) utilized the.infiltration data to test a fitting procedure for the Green and Ampt infiltration equation parameters. They concluded that variation in the fillable porosity and effective conductivity parameters has a major influence on infiltration and runoff amounts and rates whereas the effective capillary pressure is the least sensitive parameter. Though, the equation parameters have physical significance and can be computed from soil properties, it is more advantageous to determine these parameter from field measurement by fitting measured infiltration data. It will further enhance applicability of Green-Ampt equation for describing infiltration under varied initial boundary and soil profile conditions.

(iii) Richards Equation

Richards (1931) derived the most theoretically based equation for unsaturated media by combining the Darcy's Law and the principle of conservation of mass for the soil which may be written as:

$$Q_s = - K \, \delta H/\delta s \qquad (20)$$

$$\delta\theta/\delta t = - \nabla q \qquad (21)$$

where, q_s is the flux or volume of water moving through the soil in the S-direction per unit area per unit time and $\delta H/\delta s$ is the hydraulic gradient in the S-direction, K is the hydraulic conductivity which depends on both properties of fluid and the porous medium. H is the total potential head and may be considered equal to the hydraulic head which is the sum of the pressure head, h, and the distance above the datum plane or the elevation-head, Z. Taking datum plane at the soil surface.

$$H = h - Z \qquad (22)$$

where, Z is the distance measured positive downwards from the surface. For partially saturated soil condition where the water content varies with both time and position, the equation for flux may be written as,

$$q = \; = - K(\theta) \; \frac{\delta H}{\delta s} \qquad (23)$$

Since the relation $\theta = \theta$ (h) is a property of the soil, we may write K = K(h) and Equation becomes

$$q = \; = -K(h) \frac{\delta H}{\delta s} \tag{24}$$

where, q is the flux vector, t is time and θ is the volumetric soil water content. For flux in the vertical Z direction only, equation may be written as,

$$\frac{\delta\theta}{\delta t} = -\frac{\delta q_z}{\delta_z} \tag{25}$$

Combining these equations, the Richards equation in the vertical direction may be written as,

$$C\ (h) \frac{\delta h}{\delta t} = \frac{\delta}{\delta z} [\, K(h)\ \delta h/\delta z \,] - \frac{\delta K}{\delta z} \tag{26}$$

where, C(H) is the soil water capacity and may be obtained from the soil water characteristic as,

$$C(h) = \frac{d\theta}{dh} \tag{27}$$

The pressure head based equation assumes that there is no resistance to soil air movement and the air pressure remains constant throughout the profile which is often not the case.

In terms of water content θ, Richards equation may be written as,

$$\frac{\delta\theta}{\delta t} = \frac{\delta}{\delta z} \left[D(\theta) \frac{\delta\theta}{\delta z} \right] - \frac{\delta K}{\delta z} \tag{28}$$

where, D (θ) = K(h) dh/dθ and is known as soil water diffusivity.

Equations 27 and 28 are known as h-based and θ-based equations, respectively. The θ-based equation contains D(θ) and K(θ) and the h-based equation contains C(h) and K(h) as the parameters. For unsaturated soils, these parameters are related by D = K/C. The main problem in solving Richards equation subject to boundary conditions pertinent to infiltration is that for most of the soils, all these three parameters behave non-linearly with either water content or pressure head. Where both saturated and unsaturated flow conditions exist, h-based equation provides a valid solution, whereas under completely unsaturated condition, θ-based equation can be used with a dual advantage because changes in both. θ and D are typically an order of magnitude less than corresponding changes in h and C. Moreover for homogenous soil, only θ-based equation can be used. Under saturated conditions, θ-based equation, however, does not hold good as D (θ) tends to infinity.

(iv) Numerical solutions of Richards equation

Under non - uniform initial water contents, time dependent boundary conditions, heterogeneous and anisotropic porous media with hysteresis in soil properties, numerical solution of the governing equations provide the only answer. Staple (1966) used explicit scheme for solving Richards equation for infiltration and redistribution problems. Rubin (1967), Amerman (1976) and Freeze (1971) developed numerical procedures for solving Richards equation for two and three dimensional flow. Freeze (1969) summarized the finite difference solutions for the Richards equations. Whisler and Klute (1965) used an interactive numerical procedure to solve h-based equation subject to a non-uniform initial water content. The method is, however, time consuming and the nature of convergence or divergence of such a scheme appears to be dependent upon the type of equation used, the finite grid dimensions and the local shape of the θ-h-K relationship (Smith and Woolhiser, 1971). Molz and Remson (1970) and Havercamp *et. al*. (1971) developed. Implicit finite difference formulations and solutions of Richards equation using the Predictor- Corrector method.

The application of finite element technique to infiltration problems, especially for complex flow domains is of recent origin but is gaining recognition. Hayhoe (1978), while comparing the finite difference schemes and finite element technique concluded that Galerkin scheme with the linear elements or the finite difference scheme with the modified averaging procedure is preferable to other schemes. Singh and Kumar (1985) solved Richards equation with a sink term using finite element technique for studying the water balance of an aquifer-soil system in the presence of crops. The Richards equation with a sink term has also been solved numerically by Nimah and Hanks (1973), Hillel (1977), Feddes *et.al.* (1976) and Slack et. al. (1977).

6.0 SOIL EROSION

Soil erosion is a two phase process involving the detachment of individual soil particles from the soil mass, transporting it from one place to another (by the action of any one of the agents of erosion, viz.; water, wind, ice or gravity, water and wind being the two major agents) and its deposition. When sufficient energy is not available to transport a particle, a third phase known as deposition occurs. In general, finer soil particles get eroded more easily than coarse particles (silt is more easily eroded than sand). Hence soil erosion is defined as a process of detachment, transportation and deposition of soil particles (sediment). It is evident that sediment is the end product of soil erosion process. Sediment is, therefore, defined as any fragmented material, which is transported or deposited by water, ice, air or any other natural agent. From this, it is inferred that sedimentation is also the process of detachment, transportation and deposition of eroded soil particles. Erosion and sedimentation are major soil conservation problems. Erosion is almost universally recognized as a serious threat to man's well being. Erosion reduces the productivity of crop land by removing and washing away of plant nutrients and organic matter. Sediment degrades water quality and may carry soil and absorbed polluting chemicals. Deposition in irrigation canals, stream channels, reservoirs, estuaries, harbors and other water conveyance structures reduces the capacity of these structures and requires removal, which is costly.

6.1 Factors Influencing Erosion

For planning strategies for soil erosion control, it is, therefore, necessary to have a thorough knowledge of factors influencing erosion. The major factors influencing erosion are climate, topography, soil, vegetation and biotic activities.

6.1.1 Climate

Climatic factors affecting the erosion are rainfall characteristics, temperature, wind, humidity and solar radiation. Of these, rainfall is one of the most powerful factor causing runoff and soil loss. Rainfall characteristics such as amount, duration, intensity and frequency greatly influence the runoff and erosion. A large total rainfall of low intensity may not produce any soil loss. Similarly, rainfall of high intensity for short duration may not produce sufficient runoff to cause erosion.

6.1.2 Topography

Topographical features that influence erosion are degree of slope, length of slope, micro-topography, aspect of slope, size and shape or configuration of a watershed. On flat slopes, erosion is usually not a problem. On steep slopes, erosion is a very serious problem. As the land slope increases from mild to steep, the depression storage decreases and amount and rate of runoff increases.

6.1.3 Soils

The response of soil erosion process is complex and is greatly influenced by four soil characteristics, viz; texture, organic matter content, structure and permeability. Although each characteristic is discussed individually as below, the four characteristics influence erosion in an integrated manner.

a) Soil texture: It refers to the sizes and proportion of the particles making up a particular soil. Sand, silt and clay are the three major fractions of soil particles. The related percentage of these particles in soil determines the soil type. Soils which are high in sand content are considered as coarse textured. Because water readily infiltrates into the sandy soils, the runoff and consequently erosion potential is relatively low. These soils are more easily detachable and less easily transportable than clayey soils. Soils with high content of silt and clay are fine textured or heavy. Clay because of its stickiness binds soil particles together and is not easily detachable, but has low infiltration rate that leads to high runoff and increases erosion. Silt soils on the other hand because of their easy detachability and easy transportability are most erodible.

b) Organic matter: It consists of plant and animal litter in various stages of decomposition. Organic matter improves soil structure and increases permeability, water holding capacity and soil fertility. Increase in permeability and water holding capacity of soil reduces runoff and consequently erosion potential.

c) Soil structure: It refers to the arrangement of particles into the aggregates. A granular structure is most desirable one. Soil structure affects the soils ability to adsorb water. When the soil surface is compacted or crusted, water leads to runoff rather than infiltration. Erosion hazard increases with increased runoff. Loose granular soils adsorb and retain water, which reduces runoff and encourages plant growth.

d) Soil permeability: This indicates the ability of soil to allow air and water to move through the soil. Soil texture, structure and organic matter all contribute to permeability. Soils with high permeability produce less runoff at a lower rate than soils with low permeability which minimizes erosion potential.

6.1.4 Vegetation

All types of vegetation ranging from grass to forests, even crop stubble and residue in agricultural fields acts as a cover over the soil surface and provides it a much needed protection against the impact of falling rain drops. Cover reduces the effect of raindrops thereby preventing the detachment of soil particles. Vegetation and vegetation debris creates an obstruction in the path of surface water flow, resulting in an increased infiltration of runoff into the soil profile. This increases the moisture content of the soil and also reduces the runoff magnitude and soil erosion. Canopy cover of trees, shrubs and grasses and also of agricultural crops intercept significant quantities of rainfall, a part of which reaches the ground as stem flow and leaf drop. The part of rainfall which is retained on the canopy and later gets evaporated is called interception. The water loss due to interception varies with type, density and height of vegetation. This part of rainfall does not contribute to runoff. Roots of plant especially in a multi-layered forest hold the soil upto different depths making it difficult for running water to detach and transport the soil. Vegetation especially that of grasses and herbs acts as a fine sieve holding and reducing sediment load of surface runoff. Thus, a major portion of detached soil particles can be held in the same area, provided there exists a good soil cover.

6.2 Equations for Estimating Soil Erosion

Most commonly used equations are the USLE (Wischmeier and Smith, 1978), the Onstad – Foster modification of the USLE (Onstad and Foster, 1975), the MUSLE (Williams, 1975), two modifications in MUSLE, MUSLE structure that accepts in four coefficients and RULSE (Renard et al., 1991). These equations are identical except for their energy components which is a primary factor for detachment of soil particles. The USLE depends strictly upon rainfall as an indicator of erosive energy. The MUSLE and its modifications use only runoff variables to simulate erosion and sediment yield. Runoff variables in MUSLE increase the prediction accuracy, eliminate the need for delivery ratio (used in USLE to estimate sediment yield), and enables the equation to give single storm estimates of sediment yields. The USLE gives only annual estimates. The Onstad – Foster equations contains a combination of the USLE and MUSLE energy factors. Thus, the water erosion models can be expressed as:

$$Y = \chi * K * LS * C* P* F \quad (29)$$

$$\chi = EI \quad \text{for USLE} \quad (30)$$

$$\chi = 0.646EI + 0.45\,(Q.\,q_p)^{0.33} \quad \text{for Onstad Foster} \quad (31)$$

$$\chi = 1.586\,(Q.q_r)^{0.56}\,A^{0.12} \quad \text{for MUSLE} \quad (32)$$

$$\chi = 2.5\,(Q.q_p)^{0.5} \quad \text{for MUST} \quad (33)$$

$$\chi = 0.79\,(Q.q_p)^{0.65}\,A^{0.009} \quad \text{for MUSS} \quad (34)$$

$$\chi = b_1\,Q^{b}{}_2\;q_p{}^{b}{}_3\,A^{b}{}_4 \quad \text{for MUSI} \quad (35)$$

where, Y is the sediment yield in t ha^{-1}, χ is the rainfall erosivity factor in different equations, K is soil erodibility, LS is the slope length and steepness factor, C is the crop management factor, P is the erosion control practice factor, F is the coarse fragment factor, Q is the runoff volume in mm, q_p is the peak runoff rate in mm h^{-1}, A is the watershed area in ha, MUST is a new equation theoretically developed from sediment concentration bases, MUSS is an equation developed by fitting small watershed data, and MUSI allows user input of four coefficients.

RUSLE (Revised Universal Soil Loss Equation) was released in the early 1990s (Renard et al. 1991). RUSLE is land-use independent and applies to any land use condition. The USLE is an index based, empirically derived model. RUSLE is a hybrid model that combines index and process-based equations. Both models compute average annual rate of soil erosion. In USLE, average annual factor values are multiplied, however, RUSLE multiplies the factor values for each day to estimate daily erosion values, which are summed for average annual erosion. This difference results in as much as 20% difference in average annual erosion values between RUSLE and the USLE. RUSLE uses basic variables rather than the *RKLSCP* factors to compute erosion. RUSLE is mathematically superior to USLE. Also, RUSLE is much more powerful than the USLE and uses better relationships to compute factor values.

RUSLE uses a subfactor method to compute C factors, which are the ratios of soil loss at any given time in a cover-management sequence to soil loss from the unit plot. Soil loss ratios vary with time as canopy, ground cover, soil biomass and consolidation change. A "C" factor value in RUSLE is an average soil loss ratio weighted according to the distribution of "χ" during the year. The subfactors used to compute a soil loss ratio value are canopy, surface cover, surface roughness, and prior land use. Both time-variant (cropping/rotation scenario) and time-invariant (average annual values) values of C factor can be used. The time-variant option is used when plant and/or soil conditions change enough to significantly affect erosion during the year, during a rotation cycle, or over an extended period. This option is typically applied to croplands; rangelands where cover changes significantly during the year such as from grazing, burning, or herbicide application; sites regenerating following soil-disturbing activities on forest lands; and recovery following construction or earth moving activities. The time-invariant option is used where constant conditions can be assumed, which is often the case on range and pasture land.

Typical user inputs required for time-invariant C factor estimation are: effective root mass in top 10 cm of soil (kg/ha), percent canopy, average fall height (m), surface roughness value, percent ground cover (rock + litter, excluding plant basal cover), and surface cover function which represents the relative effectiveness of surface cover for reducing soil loss. The choice of surface cover function is based on the ratio of rill/interrill erosion under bare soil conditions. RUSLE requires additional data for its application to field conditions and estimated soil losses are sensitive to changes in input variables to differing degrees.

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