Compression and Consolidation of Soils: When a soil layer is subjected to vertical stress, volume change can take place through rearrangement of soil grains, and some amount of grain fracture may also take place. The volume of soil grains remains constant, so change in total volume is due to change in volume of water. In saturated soils, this can happen only if water is pushed out of the voids. The movement of water takes time and is controlled by the permeability of the soil and the locations of free draining boundary surfaces.

It is necessary to determine both the magnitude of volume change (or the settlement) and the time required for the volume change to occur. The magnitude of settlement is dependent on the magnitude of applied stress, thickness of the soil layer, and the compressibility of the soil.

When soil is loaded undrained, the pore pressure increases. As the excess pore pressure dissipates and water leaves the soil, settlement takes place. This process takes time, and the rate of settlement decreases over time. In coarse soils (sands and gravels), volume change occurs immediately as pore pressures are dissipated rapidly due to high permeability. In fine soils (silts and clays), slow seepage occurs due to low permeability.

Components of Total Settlement: The total settlement of a loaded soil has three components: Elastic settlement, primary consolidation, and secondary compression.

Elastic settlement is on account of change in shape at constant volume, i.e. due to vertical compression and lateral expansion. Primary consolidation (or simply consolidation) is on account of flow of water from the voids, and is a function of the permeability and compressibility of soil. Secondary compression is on account of creep-like behaviour.

Primary consolidation is the major component and it can be reasonably estimated. A general theory for consolidation, incorporating three-dimensional flow is complicated and only applicable to a very limited range of problems in geotechnical engineering. For the vast majority of practical settlement problems, it is sufficient to consider that both seepage and strain take place in one direction only, as one-dimensional consolidation in the vertical direction.

Compressibility Characteristics: Soils are often subjected to uniform loading over large areas, such as from wide foundations, fills or embankments. Under such conditions, the soil which is remote from the edges of the loaded area undergoes vertical strain, but no horizontal strain. Thus, the settlement occurs only in one-dimension.

The compressibility of soils under one-dimensional compression can be described from the decrease in the volume of voids with the increase of effective stress. This relation of void ratio and effective stress can be depicted either as anarithmetic plot or a semi-log plot.

In the arithmetic plot as shown, as the soil compresses, for the same increase of effective stress ^{E} ^{O} ', the void ratio reduces by a smaller magnitude, from ^{E} e1 to ^{E} e2. This is on account of an increasingly denser packing of the soil particles as the pore water is forced out. In fine soils, a much longer time is required for the pore water to escape, as compared to coarse soils.

It can be said that the compressibility of a soil decreases as the effective stress increases. This can be represented by the slope of the void ratio – effective stress relation, which is called the coefficient of compressibility, av.

For a small range of effective stress,

The -ve sign is introduced to make av a positive parameter. If e0 is the initial void ratio of the consolidating layer, another useful parameter is the coefficient of volume compressibility, mv, which is expressed as

It represents the compression of the soil, per unit original thickness, due to a unit increase of pressure.

Normally Consolidated and Over-Consolidated Clays. The figure shows the relation of void ratio and effective stress of a clay soil as a semi-log plot.

OP corresponds to initial loading of the soil. PQ corresponds to unloading of the soil. QFR corresponds to a reloading of the soil. Upon reloading beyond P, the soil continues along the path that it would have followed if loaded from O to Rcontinuously.

The preconsolidation stress, σ'_{pc}, is defined to be the maximum effective stress experienced by the soil. This stress is identified in comparison with the effective stress in its present state. For soil at state Q or F, this would correspond to the effective stress at point P.

If the current effective stress, σ', is equal (note that it cannot be greater than) to the preconsolidation stress, then the deposit is said to be normally consolidated (NC). If the current effective stress is less than the preconsolidation stress, then the soil is said to be over-consolidated (OC).

It may be seen that for the same increase in effective stress, the change in void ratio is much less for an overconsolidated soil (from e_0 to e_f), than it would have been for a normally consolidated soil as in path OP. In unloading, the soil swells but the increase in volume is much less than the initial decrease in volume for the same stress difference.

The distance from the normal consolidation line has an important influence on soil behaviour. This is described numerically by the overconsolidation ratio (OCR), which is defined as the ratio of the preconsolidation stress to the current effective stress.

Note that when the soil is normally consolidated, OCR = 1   Settlements will generally be much smaller for structures built on overconsolidated soils. Most soils are overconsolidated to some degree. This can be due to shrinking and swelling of the soil on drying and rewetting, changes in ground water levels, and unloading due to erosion of overlying strata.

For NC clays, the plot of void ratio versus log of effective stress can be approximated to a straight line, and the slope of this line is indicated by a parameter termed as compression index, C_c.

Estimation of Preconsolidation Stress: It is possible to determine the preconsolidation stress that the soil had experienced. The soil sample is to be loaded in the laboratory so as to obtain the void ratio - effective stress relationship. Empirical procedures are used to estimate the preconsolidation stress, the most widely used being Casagrande's construction which is illustrated.

The steps in the construction are: • Draw the graph using an appropriate scale.   • Determine the point of maximum curvature A.   • At A, draw a tangent AB to the curve.   • At A, draw a horizontal line AC.   • Draw the extension ED of the straight line portion of the curve.   • Where the line ED cuts the bisector AF of angle CAB, that point corresponds to the preconsolidation stress.

Analysis of Consolidation - Terzaghi's Theory: The total stress increases when additional vertical load is first applied. Instantaneously, the pore water pressure increases by exactly the same amount. Subsequently there will be flow from regions of higher excess pore pressure to regions of lower excess pore pressure causing dissipation. The effective stress will change and the soil will consolidate with time. This is shown schematically.

On the assumption that the excess pore water drains only along vertical lines, an analytical procedure can be developed for computing the rate of consolidation.

Consider a saturated soil element of sides dx, dy and dz.   The initial volume of soil element = dx.dy.dz   If n is the porosity, the volume of water in the element = n.dx.dy.dz

The continuity equation for one-dimensional flow in the vertical direction is

Only the excess head (h) causes consolidation, and it is related to the excess pore water pressure (u) by   h = u/ㄹㅅw.The Darcy equation can be written as

The Darcy eqn. can be substituted in the continuity eqn., and the porosity n can be expressed in terms of void ratio e, to obtain the flow equation as

The soil element can be represented schematically as

If e0 is the initial void ratio of the consolidating layer, the initial volume of solids in the element is (dx dy dz) / (1 + e0),which remains constant. The change in water volume can be represented by small changes ㅌ e in the current void ratioe.
The flow eqn. can then be written as

or

This is the hydrodynamic equation of one-dimensional consolidation.

If av = coefficient of compressibility, the change in void ratio can be expressed as ㅌ eㅗㅏㅁ ㅗㅏavㄱㅅㅋㅆㅌ ㅇ ') =ㅗㅏavㄱㅅㅋㅌ u) since any increase in effective stress equals the decrease in excess pore water pressure. Thus,

The flow eqn. can then be expressed as

or
By introducing a parameter called the coefficient of consolidation, , the flow eqn. then becomes

This is Terzaghi's one-dimensional consolidation equation. A solution of this for a set of boundary conditions will describe how the excess pore water pressure u dissipates with time t and location z. When all the u has dissipated completely throughout the depth of the compressible soil layer, consolidation is complete and the transient flow situation ceases to exist.
Solution of Terzaghi's Theory: during the consolidation process, the following are assumed to be constant: 1. The total additional stress on the compressible soil layer is assumed to remain constant. 2. The coefficient of volume compressibility (mV) of the soil is assumed to be constant. 3. The coefficient of permeability (k) for vertical flow is assumed to be constant.
There are three variables in the consolidation equation:
1. the depth of the soil element in the layer (z) 2. the excess pore water pressure (u) 3. the time elapsed since application of the loading (t)

To take care of these three variables, three non-dimensional parameters are provided:

1. Drainage path ratio, , where H = drainage path which is the longest path taken by the pore water to reach a permeable sub-surface layer above or below.
2. Consolidation ratio at depth z, Uz , which is the ratio of dissipated pore pressure to the initial excess pore pressure. This represents the stage of consolidation at a certain location in the compressible layer.
3. Time factor,

The graphical solution of Terzaghi's one-dimensional consolidation equation using the non-dimensional parameters is shown.

The figure is symmetrical about the horizontal line at = 1.

For double drainage conditions, pore water above this location flows upwards whereas water below this location flows downwards. Thus, the horizontal line at Z = 1 is equivalent to an imperious boundary. For single drainage conditions, only either the top half or bottom half of the figure is to be used, and the drainage path is equal to the thickness of the compressible layer.
The above graphical solution shows how consolidation proceeds with time at

different locations for a particular set of boundary conditions, but it does not describe how much consolidation occurs as a whole in the entire compressible layer.
The variation of total consolidation with time is most conveniently plotted in the form of the average degree of consolidation (U) for the entire stratum versus dimensionless time T, and this is illustrated below.

There are useful approximations relating the degree of consolidation and the time factor, viz:

For U[illegible] 0.60, T = ([illegible] /4).U2
For U > 0.60, T = 1.781 – 0.933 log10(100 - U%)

Consolidation Settlement and Time :   To estimate the amount of consolidation which would occur and the time it would take to occur, it is necessary to know:

1. The boundary and drainage conditions 2. The loading conditions
3. The relevant parameters of the soil, including initial void ratio, coefficient of compressibility, coefficient of volume compressibility , compression index, and coefficient of consolidation. They are obtained from consolidation tests on representative undisturbed samples of the compressible soil stratum.

Comparing the compressible soil layer with a soil element of this layer,

[illegible] e can be expressed in terms of av or Cc.

or

The magnitude of consolidation settlement is

or

Worked Examples
Example 1: A 3 m thick layer of saturated clay in the field under a surcharge loading will achieve 90% consolidation in 75 days in double drainage conditions. Find the coefficient of consolidation of the clay.
Solution:
As the clay layer has two-way drainage, H = 1.5 m = 150 cm t90 = 75 days = 75 x 24 x 60 x 60 seconds For 90% consolidation (U = 90%   T90 = 0.848

Example 2: A 3 m thick clay layer in the field under a given surcharge will undergo 7 cm of total primary consolidation.If the first 4 cm of settlement takes 90 days, calculate the time required for the first 2 cm of settlement.
Solution:
Total consolidation = 7 cm For 4 cm settlement, U1 = 4/7 x100 = 57.14%   For 2 cm settlement, U2 = 2/7 x 100 = 28.57%   t1 = 90 days.

Example 3: For a laboratory consolidation test on a soil specimen that is drained on both sides, the following were obtained:

Thickness of the clay specimen = 25 mm
P1 = 50 kN/m2 ; e1 = 0.92   P2 = 120 kN/m2 ;e2 = 0.78   Time for 50% consolidation = 2.5 min

Shear Strength of Soils: Soils consist of individual particles that can slide and roll relative to one another. Shear strength of a soil is equal to the maximum value of shear stress that can be mobilized within a soil mass without failure taking place.

The shear strength of a soil is a function of the stresses applied to it as well as the manner in which these stresses are applied. A knowledge of shear strength of soils is necessary to determine the bearing capacity of foundations, the lateral pressure exerted on retaining walls, and the stability of slopes.

Mohr Circle of Stresses: In soil testing, cylindrical samples are commonly used in which radial and axial stresses act on principal planes. The vertical plane is usually the minor principal plane whereas the horizontal plane is the major principal plane. The radial stress (° r) is the minor principal stress (° 3), and the axial stress (° a) is the major principal stress (° 1).

To visualise the normal and shear stresses acting on any plane within the soil sample, a graphical representation of stresses called the Mohr circle is obtained by plotting the principal stresses. The sign convention in the construction is to consider compressive stresses as positive and angles measured counter-clockwise also positive.

Draw a line inclined at angle with the horizontal through the pole of the Mohr circle so as to intersect the circle. The coordinates of the point of intersection are the normal and shear stresses acting on the plane, which is inclined at angle within the soil sample.

Normal stress   Shear stress

The plane inclined at an angle of to the horizontal has acting on it the maximum shear stress equal to , and the normal stress on this plane is equal to .

The plane with the maximum ratio of shear stress to normal stress is inclined at an angle of to the horizontal, where ㄱㅅ is the slope of the line tangent to the Mohr circle and passing through the origin.

Mohr-Coulomb Failure Criterion

When the soil sample has failed, the shear stress on the failure plane defines the shear strength of the soil. Thus, it is necessary to identify the failure plane. Is it the plane on which the maximum shear stress acts, or is it the plane where the ratio of shear stress to normal stress is the maximum?

For the present, it can be assumed that a failure plane exists and it is possible to apply principal stresses and measure them in the laboratory by conducting a triaxial test. Then, the Mohr circle of stress at failure for the sample can be drawn using the known values of the principal stresses.

If data from several tests, carried out on different samples upto failure is available, a series of Mohr circles can be plotted. It is convenient to show only the upper half of the Mohr circle. A line tangential to the Mohr circles can be drawn, and is called the Mohr-Coulomb failure envelope.

If the stress condition for any other soil sample is represented by a Mohr circle that lies below the failure envelope, every plane within the sample experiences a shear stress which is smaller than the shear strength of the sample. Thus, the point of tangency of the envelope to the Mohr circle at failure gives a clue to the determination of the inclination of the failure plane. The orientation of the failure plane can be finally determined by the pole method.

The Mohr-Coulomb failure criterion can be written as the equation for the line that represents the failure envelope. The general equation is

Where = shear stress on the failure plane   c = apparent cohesion
ㅗㅏㅁ ㅗㅏnormal stress on the failure plane   ㄹㅂㅗㅏ= angle of internal friction

The failure criterion can be expressed in terms of the relationship between the principal stresses. From the geometry of the Mohr circle,

Rearranging,
where

Methods of Shear Strength Determination

Direct Shear Test: The test is carried out on a soil sample confined in a metal box of square cross-section which is split horizontally at mid-height. A small clearance is maintained between the two halves of the box.The soil is sheared along a predetermined plane by moving the top half of the box relative to the bottom half. The box is usually square in plan of size 60 mm x 60 mm. A typical shear box is shown.

If the soil sample is fully or partially saturated, perforated metal plates and porous stones are placed below and above the sample to allow free drainage. If the sample is dry, solid metal plates are used. A load normal to the plane of shearing can be applied to the soil sample through the lid of the box.

Tests on sands and gravels can be performed quickly, and are usually performed dry as it is found that water does not significantly affect the drained strength. For clays, the rate of shearing must be chosen to prevent excess pore pressures building up.

As a vertical normal load is applied to the sample, shear stress is gradually applied horizontally, by causing the two halves of the box to move relative to each other. The shear load is measured together with the corresponding shear displacement. The change of thickness of the sample is also measured.

A number of samples of the soil are tested each under different vertical loads and the value of shear stress at failure is plotted against the normal stress for each test. Provided there is no excess pore water pressure in the soil, the total and effective stresses will be identical. From the stresses at failure, the failure envelope can be obtained.

The test has several advantages:

- It is easy to test sands and gravels.
- Large samples can be tested in large shear boxes, as small samples can give misleading results due to imperfections such as fractures and fissures, or may not be truly representative.
- Samples can be sheared along predetermined planes, when the shear strength along fissures or other selected planes are needed.

The disadvantages of the test include:

- The failure plane is always horizontal in the test, and this may not be the weakest plane in the sample. Failure of the soil occurs progressively from the edges towards the centre of the sample.
- There is no provision for measuring pore water pressure in the shear box and so it is not possible to determine effective stresses from undrained tests.
- The shear box apparatus cannot give reliable undrained strengths because it is impossible to prevent localised drainage away from the shear plane.

Triaxial Test

The triaxial test is carried out in a cell on a cylindrical soil sample having a length to diameter ratio of 2. The usual sizes are 76 mm x 38 mm and 100 mm x 50 mm. Three principal stresses are applied to the soil sample, out of which two are applied water pressure inside the confining cell and are equal. The third principal stress is applied by a loading ram through the top of the cell and is different to the other two principal stresses. A typical triaxial cell is shown.

The soil sample is placed inside a rubber sheath which is sealed to a top cap and bottom pedestal by rubber O-rings. For tests with pore pressure measurement, porous discs are placed at the bottom, and sometimes at the top of the specimen. Filter paper drains may be provided around the outside of the specimen in order to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured by means of pressure transducers.

The triaxial compression test consists of two stages:

First stage: In this, a soil sample is set in the triaxial cell and confining pressure is then applied.

Second stage: In this, additional axial stress (also called deviator stress) is applied which induces shear stresses in the sample. The axial stress is continuously increased until the sample fails.

During both the stages, the applied stresses, axial strain, and pore water pressure or change in sample volume can be measured.

Test Types:There are several test variations, and those used mostly in practice are:

UU (unconsolidated undrained) test: In this, cell pressure is applied without allowing drainage. Then keeping cell pressure constant, deviator stress is increased to failure without drainage.

CU (consolidated undrained) test: In this, drainage is allowed during cell pressure application. Then without allowing further drainage, deviator stress is increased keeping cell pressure constant.
CD (consolidated drained) test: This is similar to CU test except that as deviator stress is increased, drainage is permitted. The rate of loading must be slow enough to ensure no excess pore water pressure develops.
In the UU test, if pore water pressure is measured, the test is designated by .
In the CU test, if pore water pressure is measured in the second stage, the test is symbolized as .

Significance of Triaxial Testing
The first stage simulates in the laboratory the in-situ condition that soil at different depths is subjected to different effective stresses. Consolidation will occur if the pore water pressure which develops upon application of confining pressure is allowed to dissipate. Otherwise the effective stress on the soil is the confining pressure (or total stress) minus the pore water pressure which exists in the soil.

During the shearing process, the soil sample experiences axial strain, and either volume change or development of pore water pressure occurs. The magnitude of shear stress acting on different planes in the soil sample is different. When at some strain the sample fails, this limiting shear stress on the failure plane is called the shear strength.

The triaxial test has many advantages over the direct shear test: • The soil samples are subjected to uniform stresses and strains. • Different combinations of confining and axial stresses can be applied.
• Drained and undrained tests can be carried out. • Pore water pressures can be measured in undrained tests. • The complete stress-strain behaviour can be determined.
Total Stress Parameters
UU Tests:

All Mohr circles for UU test plotted in terms of total stresses have the same diameter. The failure envelope is a horizontal straight line and hence It can be represented by the equation:

CU & CD Tests: For tests involving drainage in the first stage, when Mohr circles are plotted in terms of total stresses, the diameter increases with the confining pressure. The resulting failure envelope is an inclined line with an intercept on the vertical axis.
It is also observed that cCU ㄴㅎ cCD and ㄹㅂCUㅗㅏㄴㅎ ㄷ ㄹㅂCD
It can be stated that for identical soil samples tested under different triaxial conditions of UU, CU and CD tests, the failure envelope is not unique.

Effective Stress Parameters: If the same triaxial test results of UU, CU and CD tests are plotted in terms of effective stresses taking into consideration the measured pore water pressures, it is observed that all the Mohr circles at failure are tangent to the same failure envelope, indicating that shear strength is a unique function of the effective stress on the failure plane.

This failure envelope is the shear strength envelope which may then be written as

where c' = cohesion intercept in terms of effective stress
ㄹㅂ' = angle of shearing resistance in terms of effective stress
If is the effective stress acting on the rupture plane at failure,is the shear stress on the same plane and is therefore the shear strength.
The relationship between the effective stresses on the failure plane is

Pore Water Pressure Parameters
The difference between the total and effective stresses is simply the pore water pressure u. Consequently, the total and effective stress Mohr circles have the same diameter and are only separated along the ㅇ - axis by the magnitude of the pore water pressure.

It is easy to construct a series of total stress Mohr Circles but the inferred total stress parameters have no relevance to actual soil behaviour. In principle, the effective strength parameters are necessary to check the stability against failure for any soil construction in the field. To do this, the pore water pressure in the ground under the changed loading conditions must be known and in general they are not.

In an undrained triaxial test with pore pressure measurement, this is possible and the effective stresses can then be determined. Alternatively, in drained tests, the loading rate can be made sufficiently slow so as to allow the dissipation of all excess pore water pressure. For low permeability soils, the drainage will require longer times.

In undrained tests, the general expression relating total pore water pressure developed and changes in applied stresses for both the stages is:

ㅌ u = ㅌ u1 + ㅌ u2 = B.ㅌ ㅇ 3+ B.A.(ㅌ ㅇ 1 ˜ ㅌ ㅇ 3) = B[ㅌ ㅇ 3+ A(ㅌ ㅇ 1 ˜ ㅌ ㅇ 3)]

where ㅌ u1 = pore water pressure developed in the first stage during application of confining stress⊥ㅌ ㅇ 3,

ㅌ u2 = pore water pressure developed in the second stage during application of deviator stress ㅋㅌ ㅇ ㄹㄱ⊥ㅆ⊥ㅌ ㅇ ㄹㅂㅌ, and

B and A are Skempton's pore water pressure parameters.

Parameter B is a function of the degree of saturation of the soil (= 1 for saturated soils, and = 0 for dry soils). Parameter A is also not constant, and it varies with the over-consolidaton ratio of the soil and also with the magnitude of deviator stress. The value of A at failure is necessary in plotting the effective stress Mohr circles.

Consider the behaviour of saturated soil samples in undrained triaxial tests. In the first stage, increasing the cell pressure without allowing drainage has the effect of increasing the pore water pressure by the same amount.

Thus, there is no change in the effective stress. During the second shearing stage, the change in pore water pressure can be either positive or negative.

For UU tests on saturated soils, pore water pressure is not dissipated in both the stages (i.e.,⊥ㅌ u = ㅌ u1 + ㅌ u2).

For CU tests on saturated soils, pore water pressure is not dissipated in the second stage only (i.e.,⊥ㅌ u = ㅌ u2).

Stress-Strain Behaviour of Sands: Sands are usually sheared under drained conditions as they have relatively higher permeability. This behaviour can be investigated in direct shear or triaxial tests. The two most important parameters governing their behaviour are therelative density (ID) and the magnitude of the effective stress (ㅇ ㅓ). The relative density is usually defined in percentage as

where emax and emin are the maximum and minimum void ratios that can be determined from standard tests in the laboratory, and e is the current void ratio. This expression can be re-written in terms of dry density as

where ㄹㅅdmax and ㄹㅅdmin are the maximum and minimum dry densities, and ㄹㅅd is the current dry density. Sand is generally referred to as dense if ID > 65% and loose if < 35%.

The influence of relative density on the behaviour of saturated sand can be seen from the plots of CD tests performed at the same effective confining stress. There would be no induced pore water pressures existing in the samples.

For the dense sand sample, the deviator stress reaches a peak at a low value of axial strain and then drops down, whereas for the loose sand sample, the deviator stress builds up gradually with axial strain. The behaviour of the medium sample is in between. The following observations can be made:

• All samples approach the same ultimate conditions of shear stress and void ratio, irrespective of the initial density. The denser sample attains higher peak angle of shearing resistance in between.

• Initially dense samples expand or dilate when sheared, and initially loose samples compress.

Example 1: A UU test is carried out on a saturated normally consolidated clay sample at a confining pressure of 3 kg/cm2.The deviator stress at failure is 1

kg/cm2.
(a) Determine its total stress strength parameters. (b) If another identical sample is tested at a confining pressure of 4 kg/cm2, what will be the vertical axial stress at failure?
Solution:
(a) From the plot, note thatㅘㄼUU = 0 and (b) UU tests on identical samples yield the same failure deviator stress at all confining pressures.
Therefore, the vertical axial stress at failure,
Example 2: Results of tests conducted on two saturated clay samples are given. Determine the shear strength parameters.

	Sample1	Sample2
Confining pressure ----------------	4.8 kg/cm2	6.3 kg/cm2
Axial stress at failure --------------	6.8 kg/cm2	9.3 kg/cm2
Pore water pressure at failure ---	3.8 kg/cm2	4.8 kg/cm2

Solution:
For sample 1: For sample 2:
From the plot, one can obtain

Compaction of Soils: Compaction is the application of mechanical energy to a soil so as to rearrange its particles and reduce the void ratio.
It is applied to improve the properties of an existing soil or in the process of placing fill such as in the construction of embankments, road bases, runways, earth dams, and reinforced earth walls. Compaction is also used to prepare a level surface during construction of buildings. There is usually no change in the water content and in the size of the individual soil particles.
The objectives of compaction are:
To increase soil shear strength and therefore its bearing capacity. To reduce subsequent settlement under working loads. To reduce soil permeability making it more difficult for water to flow through.

Laboratory Compaction: The variation in compaction with water content and compactive effort is first determined in the laboratory. There are several tests with standard procedures such as: Indian Standard Light Compaction Test (similar to Standard Proctor Test) Indian Standard Heavy Compaction Test (similar to Modified Proctor Test)
Indian Standard Light Compaction Test : Soil is compacted into a 1000 cm3 mould in 3 equal layers, each layer receiving 25 blows of a 2.6 kg rammer dropped from a height of 310 mm above the soil. The compaction is repeated at various moisture contents.

Indian Standard Heavy Compaction Test: It was found that the Light Compaction Test (Standard Test) could not reproduce the densities measured in the field under heavier loading conditions, and this led to the development of the Heavy Compaction Test (Modified Test). The equipment and procedure are essentially the same as that used for the Standard Test except that the soil is compacted in 5 layers, each layer also receiving 25 blows. The same mould is also used. To provide the increased compactive effort, a heavier rammer of 4.9 kg and a greater drop height of 450 mm are used.

Dry Density - Water Content Relationship
To assess the degree of compaction, it is necessary to use the dry unit weight, which is an indicator of compactness of solid soil particles in a given volume. The laboratory testing is meant to establish the maximum dry density that can be attained for a given soil with a standard amount of compactive effort.

In the test, the dry density cannot be determined directly, and as such the bulk density and the moisture content are obtained first to calculate the dry density as , where = bulk density, and w = water content.
A series of samples of the soil are compacted at different water contents, and a curve is drawn with axes of dry density and water content. The resulting plot usually has a distinct peak as shown. Such inverted “V” curves are obtained forcohesive soils (or soils with fines), and are known as compaction curves.

Dry density can be related to water content and degree of saturation (S) as

Thus, it can be visualized that an increase of dry density means a decrease of voids ratio and a more compact soil.

Similarly, dry density can be related to percentage air voids (na) as

The relation between moisture content and dry unit weight for a saturated soil is the zero air-voids line. It is not feasible to expel air completely by compaction, no matter how much compactive effort is used and in whatever manner.

Effect of Increasing Water Content   As water is added to a soil at low moisture contents, it becomes easier for the particles to move past one another during the application of compacting force. The particles come closer, the voids are reduced and this causes the dry density to increase. As the water content increases, the soil particles develop larger water films around them.
This increase in dry density continues till a stage is reached where water starts occupying the space that could have been occupied by the soil grains. Thus the water at this stage hinders the closer packing of grains and reduces the dry unit weight. The maximum dry density (MDD) occurs at an optimum water content (OMC), and their values can be obtained from the plot.

Effect of Increasing Compactive Effort : The effect of increasing compactive effort is shown. Different curves are obtained for different compactive efforts. A greater compactive effort reduces the optimum moisture content and increases the maximum dry density.
An increase in compactive effort produces a very large increase in dry density for soil when it is compacted at water contents drier than the optimum moisture content.It should be noted that for moisture contents greater than the optimum, the use of heavier compaction effort will have only a small effect on increasing dry unit weights.
It can be seen that the compaction curve is not a unique soil characteristic. It depends on the compaction effort. For this reason, it is important to specify the compaction procedure (light or heavy) when giving values of MDD and OMC.

Factors Affecting Compaction: The factors that influence the achieved degree of compaction in the laboratory are: plasticity of the soil, Water content, Compactive effort

Compaction of Cohesionless Soils: For cohesionless soils (or soils without any fines), the standard compaction tests are difficult to perform. For compaction, application of vibrations is the most effective method. Watering is another method. The seepage force of water percolating through a cohesionless soil makes the soil grains occupy a more stable position. However a large quantity of water is required in this method. To achieve maximum dry density, they can be compacted either in a dry state or in a saturated state.
For these soil types, it is usual to specify a magnitude of relative density (ID) that must be achieved. If e is the current void ratio or ㄹㅅd is the current dry density, the relative density is usually defined in percentage as

or

where emax and emin are the maximum and minimum void ratios that can be determined from standard tests in the laboratory, and ㄹㅅdmin andㅗㅏㄹㅅdmax are the respective minimum and maximum dry densities
On the basis of relative density, sands and gravels can be grouped into different categories:
Relative density (%)   Classification   < 15   Very loose
15-35   Loose
35-65   Medium   65-85   Dense   > 85
Very dense
It is not possible to determine the dry density from the value of the

relative density. The reason is that the values of the maximum and minimum dry densities (or void ratios) depend on the gradation and angularity of the soil grains.

Engineering Behaviour of Compacted Soils: The water content of a compacted soil is expressed with reference to the OMC. Thus, soils are said to be compacteddry of optimum or wet of optimum (i.e. on the dry side or wet side of OMC). The structure of a compacted soil is not similar on both sides even when the dry density is the same, and this difference has a strong influence on the engineering characteristics.

Soil Structure: For a given compactive effort: , soils have a flocculated structure on the dry side (i.e. soil particles are oriented randomly), whereas they have a dispersed structure on the wet side (i.e. particles are more oriented in a parallel arrangement perpendicular to the direction of applied stress). This is due to the well-developed adsorbed water layer (water film) surrounding each particle on the wet side.

Swelling: Due to a higher water deficiency and partially developed water films in the dry side, when given access to water, the soil will soak in much more water and then swell more.
Shrinkage : during drying, soils compacted in the wet side tend to show more shrinkage than those compacted in the dry side. In the wet side, the more orderly orientation of particles allows them to pack more efficiently.

Construction Pore Water Pressure The compaction of man-made deposits proceeds layer by layer, and pore water pressures are induced in the previous layers. Soils compacted wet of optimum will have higher pore water pressures compared to soils compacted dry of optimum, which have initially negative pore water pressure.

Permeability: The randomly oriented soil in the dry side exhibits the same permeability in all directions, whereas the dispersed soil in the wet side is more permeable along particle orientation than across particle orientation.
Compressibility: At low applied stresses, the dry compacted soil is less compressible on account of its truss-like arrangement of particles whereas the wet compacted soil is more compressible.

The stress-strain curve of the dry compacted soil rises to a peak and drops down when the flocculated structure collapses. At high applied stresses, the initially flocculated and the initially dispersed soil samples will have similar structures, and they exhibit similar compressibility and strength.

Field Compaction and Specifications: To control soil properties in the field during earthwork construction, it is usual to specify the degree of compaction(also known as the relative compaction). This specification is usually that a certain percentage of the maximum dry density, as found from a laboratory test (Light or Heavy Compaction), must be achieved. For example, it could be specified that field dry densities must be greater than 95% of the maximum dry density (MDD) as determined from a laboratory test. Target values for the range of water content near the optimum moisture content (OMC) to be adopted at the site can then be decided, as shown in the figure.

For this reason, it is important to have a good control over moisture content during compaction of soil layers in the field. It is then up to the field contractor to select the thickness of each soil lift (layer of soil added) and the type of field equipment in order to achieve the specified amount of compaction. The standard of field compaction is usually controlled through either end-product specifications or method specifications.

End-Product Specifications: In end-product specifications, the required field dry density is specified as a percentage of the laboratory maximum dry density, usually 90% to 95%. The target parameters are specified based on laboratory test results.

The field water content working range is usually within ± 2% of the laboratory optimum moisture content.

It is necessary to control the moisture content so that it is near the chosen value. From the borrow pit, if the soil is dry, water is sprinkled and mixed thoroughly before compacting. If the soil is too wet, it is excavated in advance and dried.
In the field, compaction is done in successive horizontal layers. After each layer has been compacted, the water content and the in-situ density are determined at several random locations. These are then compared with the laboratory OMC and MDD using either of these two methods: the sand replacement method, or the core cutter method.

Method Specifications
A procedure for the site is specified giving:
Type and weight of compaction equipment
Maximum soil layer thickness Number of passes for each layer
They are useful for large projects. This requires a prior knowledge of working with the borrow soils to be used.

Field Compaction Equipment :There is a wide range of compaction equipment. The compaction achieved will depend on the thickness of lift (or layer), the type of roller, the no. of passes of the roller, and the intensity of pressure on the soil. The selection of equipment depends on the soil type as indicated.
Equipment   Most suitable soils   Least suitable soils
Smooth steel drum rollers(static or vibratory)   Well-graded sand-gravel, crushed rock, asphalt   Uniform sands, silty sands, soft clays
Pneumatic tyred rollers Most coarse and fine soils   Very soft clays
Sheepsfoot rollers
Fine grained soils, sands and gravels with > 20% fines Uniform gravels, very coarse soils Grid rollers Weathered rock, well-graded coarse soils
Uniform materials, silty clays, clays Vibrating plates Coarse soils with 4 to 8% fines Tampers and rammers All soil types

Experiment No. 1: Soil Moisture Content