4. SUBGRADE SOILS

4.1 Introduction

The subgrade constitutes the foundation material for the pavement structure as highway pavements ultimately rest on the native soil (subgrade). Hence the performance of the pavement is affected by the characteristics of the subgrade. And one of the major functions of a highway pavement is to reduce the stresses transmitted to the subgrade to a level which the soil will accept without significant deformation.

Soil is also used as construction materials for highway construction (fill, capping layer, subbase, etc.).

Hence it is important to study the characteristics and engineering properties of soils for highway engineers. However, given the inherent variability in soil nature which may change in type or condition within a few meters distance, and the nature of highway projects (which are not compact like other type of civil engineering projects (buildings, dams, --)) detail testing and assessment may not be possible, and hence one has to be content with average assessment of soil conditions over a long length of road.

It is important to note here that the basic characteristics of soils and the engineering properties of the soils depend among others on the geological processes and mechanics of soil formation i.e. origin & formation of soils - parent material; mode of deposition/transport (residual or transported soil); mechanism of transport – ice (glacial soils), water (alluvial soils), wind (aeoline soils), gravity (colluvial soils)); climate; topography; time/age; vegetation, etc.

Hence a comprehensive study and analysis of soils will also involve a study of soils on a parent material basis, which relies on the geological concepts of rock and soil formation as well as a good grasp of the basic principles of soil mechanics.

4.2 Overview of Soil Survey and Site Investigation

In the evaluation of an area for construction of road including structures, or as a source of construction materials, the soil condition must be investigated before any detailed designs are made. A soil survey forms an essential part of the preliminary engineering survey for a road and its purpose is to furnish the design engineer with all required information regarding the soil and ground water condition so that a rational and economical design can be obtained.
The information most often required from soil investigation include depth, thickness, and properties of each soil layer (the characteristics of the soil profile), location of groundwater table, availability of suitable local construction materials, etc.

Information obtained from the soil survey enables to make decision on one or more of the following design requirements:

- stability of the proposed location, both horizontally and vertically, and thereby selection of roadway alignment;
- suitability of local materials for use as a construction materials for embankments and pavement layers;
- subsurface and surface drainage requirements;
- need for treatment of subgrade and type of treatment required;
- thickness of pavement required; and
- design of foundations for bridges and other structures.

In general, a soil investigation work involves the following steps:

- Desk study & Site Reconnaissance - (Feasibility Stage Investigations)
- Ground investigation (field sampling and testing) and Laboratory testing - (Detail Investigations)
- Reporting - preparation of factual and interpretative report/reports pertinent for engineering design.

**Desk study:** refers to work taken up prior to commencing the the ground investigation. It is the first step in any soil survey work and involves the collection and review of existing information on the general soil characteristics of the area in which the highway is to be located.

Relevant data to be collected and studied/analyzed include:

- general features such as geological formations, climate (temperature and rainfall) and topography of the area.
- Specific data from previous soil investigation works for road design/construction purposes or other purposes in the project area.

During the desk study, one can obtain and investigate relevant information/data from such sources as given below:
• General data - geological and pedological (soil) maps, land use maps, topographical maps and aerial photos, and climate maps, meteorological data (temperature and rainfall data);
• Specific data – test pit/borehole logs and laboratory test results of previous investigations on adjacent roads (or roads in the general project area), construction histories of such roads, other subsurface investigation data (example – mineralogical explorations, water well drilling logs, or investigations for any other civil engineering structures in the general project area)

Data and information obtained from the desk study:

• will provide preliminary information about the depth and characteristics of the soil profile, as the soil profile in a particular local generally depend on the climate, topography, origin (parent material) and mode of formation (geological process), time and vegetation cover.
• will enable delineation of approximate limits of geological formations in the project area (a map showing such limits can also be prepared).
• can also furnish indications of potential material sources.

While conducting a desk study, it is essential to gather as much information as possible, as it would save much time later and largely improves the planning and quality of the investigation.

Site Reconnaissance - The data inferred above is normally supplemented with field reconnaissance survey to assess or visually inspect actual soil condition and any problems to be anticipated (may involve collection of few samples and field/laboratory testing).

This is a walk over survey of the site involving visual inspection of alignment soils and other pertinent geotechnical, topographic and hydrologic/hydraulic features. It aims at:

• verifying the assumptions made regarding the limits of geological formations,
• confirm the indications relative to sources of materials identified during the office review,
• visual inspection of the general and particular features including vegetation cover, identifying in general terms potential problem areas like embankments on compressible soils, expansive soils, deep or potentially unstable cuts, major rock excavations, slope instability, marshy ground, springs or seepage, ponds or streams, etc,

The information gained from the desk study and reconnaissance will be used in the successive screening and final selection of the most suitable route alignment. Also it will be used to plan type and extent of subsequent detail field/laboratory investigation required.
**Detail Investigation (Ground investigation and laboratory testing)**

Detail soil survey is carried out on the selected route (either in one stage or in stages, example: Preliminary soil survey and Final soil survey). This will involve obtaining and investigating enough soil samples along the selected route, carrying out field tests as required, transporting the sampled soils to a central laboratory and laboratory testing.

These will involve visual assessment, soil sampling and field and laboratory testing.

- Visual assessment on type and extent of soil along the route (soil extension survey)
- Identification of stretches with rock outcrops, potentially unsuitable (problematic) soil formations (expansive soils, swamps, etc.)
- Assessment of drainage patterns and potential drainage problems
- Assessment of potential slope stability hazards
- Assessment of foundation conditions for bridges or other structures, embankments, etc.
- Identifying and investigation of potential suitable sources of materials for use in the road construction works – fill, capping, subbase, etc.

The results of the investigation will be used to characterize the different soil types along the road and map their boundaries (homogeneous sections), and derive pertinent design parameters required – deformation parameters (E), shear strength parameters (C, ), consolidation parameters (C_c, C_v, m_v), empirical design parameters (CBR), etc.

The investigation has to be carried out at sufficient intervals and to depths below which ground conditions cease to affect the works.

The field investigation and sampling are carried out by the following methods:

- **Test pits or trenches:** suitable for shallow depths only; enables disturbed/undisturbed soil sampling, and direct inspection and register of soil profiles.
- **Hand augers:** suitable for shallow depths only, enable inspection of disturbed and mixed samples of soil (mostly for visual inspection purposes).
- **Boring test holes and sampling with drill rigs:** using borehole rigs for advancing boreholes and obtaining disturbed/ undisturbed samples, allow greater depths of investigation.
- **Geophysical methods (Seismic or electrical)**
The routine tests normally carried out on subgrade soils include:

- **Soil classification and index tests**: gradation, Atterberg limits, moisture content;
- **Compaction and Strength tests**: compaction test (standard or modified), CBR test
- **Field tests (on existing road)**: field density and moisture content, DCP test
- **Other tests (not so common)**: Modulus of deformation (Resilient Modulus – E), plate bearing test (field test).

**Reporting:**

The process and findings of the investigations/survey works are presented in one report or series of reports.

The procedures followed, the detail plan/program of investigation and actual investigations carried out, the types and procedures (standards) followed in conducting field/laboratory tests, the actual test results obtained, analysis and interpretation of field observations/assessments & test data and final recommendations for design (design parameters and recommendations) are compiled and presented in Factual reports and interpretative (Engineering) Reports (Soils and Materials Report, Engineering Report, Geotechnical Report, etc.). The reporting shall include among others a summery of the test program, a general description of the soil conditions, a detailed analysis of each type of soil found, and recommendations for design (as required). A copy of the test-hole logs and the soil profile is also included.

### 4.2.1 Depth of Investigation

Depth of investigation should normally extend to the level below which ground conditions cease to affect the works. In this respect, it will be important to predefined the required depth of investigation (or design depth) and carry out investigations accordingly at least to this depth.

The design depth is defined as the depth from the finished road level to the depth that the load bearing strength of the soil no longer has an effect on the pavement’s performance in relation to traffic loading. Properties of soil below the design depth may indirectly affect pavement performance, but are generally unrelated to traffic loading. A preliminary vertical alignment may be required at the time of the soil survey in order to ensure that soil samples are actually taken at levels that fall within the design depth of the road.
The following table shows the depth of test pits for soil sampling as given in Tanzania Pavement Design Manual, 1999.

### Table 4.1: Design Depth (Tanzania Pavement Design Manual, 1999)

<table>
<thead>
<tr>
<th>Road Type</th>
<th>General requirements</th>
<th>Heavy traffic class roads *</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paved trunk roads</td>
<td>0.8</td>
<td>1.2</td>
</tr>
<tr>
<td>Other roads</td>
<td>0.6</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* Heavy traffic class roads are roads with proportion of traffic loading as a result of axles loaded to above 13 tonnes is > (greater than) 50% of the total design traffic loading (and CESAL > 0.2 x 10^6)

The Ethiopian Roads Authority Site Investigation manual (ERA, 2002) stipulates that for the purpose of taking representative samples, pits shall be dug mostly in anticipated cut areas (since these cuts will expose the sub-grade support of the future pavement and provide embankment materials), if possible down to at least 30 cm below the expected sub-grade level. Further, in the case of a new alignment, the depth of any pit should in no case be less than 1.5m unless rock or other material impossible to excavate by hand is encountered. The engineer in charge of planning the investigations should make every effort to locate the test pits (along the alignment as well as within the lateral extent of the anticipated excavation) in order to optimize the representatively of the material excavated from the test pit.

When required, investigations should be extended to below design depth to detect problems that need special considerations such as presence of problem soils, unfavorable subgrade conditions, and features associated with slope and embankments stability. If necessary, sub surface investigation is carried out using field or in-place testing techniques.

### 4.2.2 Sampling and Frequency

The frequency and spacing of the test pits should depend on sound engineering judgment and on the field conditions and be guided by a prior review of all possible documents as well as a preliminary visual survey of the entire road alignment and the results of the investigations during preliminary
design. Although it is neither possible nor desirable to specify rigid rules for spacing, it is necessary to set adequate average guidelines for homogeneity of design and reliability.

Table 4-2 & 4-3 give recommended sampling frequency and the corresponding tests. This frequencies may be altered depending on the variations in soil types along the alignment. The identification tests include Atterberg limits and gradation tests.

### Table 4-2: Sampling Frequency (ERA, Site Investigation Manual, 2001)

<table>
<thead>
<tr>
<th>Investigation Stage</th>
<th>Test Description</th>
<th>Frequency of Cumulative Sampling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feasibility/Preliminary</td>
<td>Identification</td>
<td>1 Km</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td>2 – 5 Km</td>
</tr>
<tr>
<td>Final</td>
<td>Identification</td>
<td>0.5Km</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td>1Km</td>
</tr>
</tbody>
</table>

### Table 4-3: Sampling Frequency (Tanzanian Pavement Design Manual, 1999)

<table>
<thead>
<tr>
<th>Type of Road</th>
<th>Test Description</th>
<th>Frequency of Cumulative Sampling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paved Trunk Roads</td>
<td>Identification</td>
<td>0.25Km</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td>0.5Km</td>
</tr>
<tr>
<td>Other Paved Roads</td>
<td>Identification</td>
<td>0.5Km</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td>1Km</td>
</tr>
<tr>
<td>Gravel Roads</td>
<td>Identification</td>
<td>0.5Km</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td>2Km</td>
</tr>
</tbody>
</table>

### 4.3 Laboratory Tests

The subgrade (the foundation material) must possess sufficient strength and stiffness to provide adequate support for the pavement structure and associated traffic load, without shear failure or excessive deformation. Owing to the fact that the nature of stresses imposed on pavements (transient loads of short duration) and their relatively lower magnitude, shear strength of the soil is not normally anticipated to be a critical factor in pavement thickness design. The elastic properties (E, ) of the subgrade are the major foundation parameters needed (assuming normal traffic loading regimes). However, these are relatively complex properties to measure and bearing in mind the variability of soils
within relatively short distances, it may not be economically feasible to use them directly to evaluate subgrade properties. Hence, if pavements are to be designed empirically based on past performance/experience records or fundamentally using the elastic theory (analytical methods), relatively simple test procedures are required, the result of which can be related by experiment to the structural properties.

The shear strength parameters – cohesion, c & angle of internal friction $\phi$- are normally required in the stability analysis of road embankments and deep cuts. Also consolidation parameters $C_v$, $m_v$, $C_c$ will be required for settlement analysis of high embankment on soft clayey soils.

General desirable properties of a subgrade soil (or any foundation material) include: Stability – good strength and stiffness under adverse loading and climatic (moisture) conditions, incompressibility, good drainage properties, ease of compaction, volume stability (no/minimum shrink / swell characteristics with change in moisture content).

In this section some common basic soil identification and classification tests routinely carried out on subgrade soils and some empirical tests specifically developed for subgrade evaluation for pavement design purposes will be discussed.

While conducting the soil testing, reference is frequently made to the standard methods of testing such as British Standard (BS), American Society for Testing and Materials (ASTM), and American Association of State Highway and Transportation Officials (AASHTO). It should be noted that when actually performing tests it is of the utmost importance that the specified standards be followed precisely, as small differences in the testing procedure may have a noticeable influence on the test result obtained.

4.3.1 Gradation Test (AASHTO T88)

The type of soil can be described in terms of the particle sizes present. The particles in soil may range from granular fractions (boulders and cobbles - >75mm in size, Gravels 75 – 4.75mm in size; Sand – 4.75 – 0.075mm) to fine fractions which are too small to measure directly (silt – 0.075mm – 0.002mm & clay - < 0.002mm and colloids - <0.001mm in size). Gradation test is conducted in order to obtain the maximum size and the grain size distribution of particles in the soil. It is expressed in terms of particles by weight finer than specified sizes. Gradation test (sieve Analysis) is carried out for soil particles larger than 0.075(0.063)mm. Sedimentation tests (Hydrometer test) is conducted for smaller (finer) particles.
Sedimentation tests are relatively complex (for site laboratories) and the properties of the silt and clay fraction for road projects are generally assessed by plasticity tests.

Depending on the sample at hand, different types of gradation tests may be carried out: Dry Sieve Analysis (for pure coarse or granular materials with out fines), Wet sieve analysis with/without sedimentation test on the fine fractions (for mixture of coarse and fine grained soils) and hydrometer analysis (for fine grained soils).

The gradation of soils influences many properties of the soil such as density/compactibility, strength/stability/deformability, voids content, permeability, etc. Also particle shape, mineral composition and degree of compaction have an effect on the above properties. Soil classification/characterization and descriptions based on results of gradation tests are described at the end of this chapter.

4.3.2 Atterberg Limits

Soils containing clay exhibit a property called plasticity. Plasticity is the ability of a material to be moulded (irreversibly deformed) without fracturing. This behavior is unique to clays and arises due to the electrochemical behavior of clay minerals.

The stiffness or consistency of fine grained soils depends on their moisture content, and varies with variations in the amount of moisture present. Depending on its moisture content, a soil can exist in one of the following states: viscous liquid, plastic solid, semi solid and solid. Atterberg in 1911 proposed a series of tests, mostly empirical, for the determination of the consistency properties/states of fine grained soils. Atterberg limits define the moisture contents at which the soil changes from one state to another. These include the liquid limit (LL), the plastic limit (PL), shrinkage limit (SL). They are determined by tests carried out on the fine soil fraction passing the 425μm (No. 40) sieve.

**Liquid limit** (AASHTO T89) may be defined as the minimum water content at which the soil will start to flow under the application of a standard shearing force (dynamic loading).

**Plastic limit** (AASHTO T90) – measure of toughness – the moisture content at which the soil begins to fracture when rolled into a 3mm diameter thread.
Shrinkage limit (9AASHTO T92) is the maximum moisture content after which further reduction in water content does not cause reduction in volume. It is the lowest water content at which a clayey soil can occur in a saturated state.

Plasticity index (PI=LL-PL) is the numerical difference between the liquid and plastic limits. Thus, it indicates the range of moisture content over which the soil remains deformable (in plastic state).

\[ LI = \frac{w_n - PL}{PI} \]  

(4.1)

where, \( w_n \) is the natural moisture content of the soil.

Consistency limits and the plasticity index are used in the identification and classification of soils. Generally, soils having high values of liquid limit and plasticity index are poor as subgrades/engineering materials. Both the liquid limit and plastic limit depend on the type and amount of clay in the soils. In soils having same values of liquid limit, but with different values of plasticity index; it is generally found that rate of volume change and dry strength increases and permeability decreases with increase in plasticity index. On the other hand, in soils having same values of plasticity index but different values of liquid limit, it is seen that compressibility and permeability increase, and dry strength decreases with increase in liquid limit. Soils that cannot be rolled to a thread at any water content are termed as Non-Plastic (NP).
4.3.3 Compaction Test

Compaction is the process by which air is excluded from a soil mass to bring the particles closer together and thus increase its density (dry density). The state of compaction of a soil is appropriately expressed in terms of the dry density ($d$) which is a measure of the state of packing of soil particles.

In-situ soils (foundation soils) in highway construction or other structures, and imported soils used in embankments, subbases, bases in roads or other types of construction projects are placed in layers and compacted to a higher density. Increasing the density of a soil improves its strength, lowers its permeability, and reduces deformability (settlement, volume change). Compaction is achieved in the field by using hand-operated tampers, sheep-foot rollers, rubber-tired rollers, or other types of roller. The maximum density achieved because of compaction with rollers, and other types of compaction equipment is measured in the field and compared with the maximum dry density of the soil previously determined in laboratory compaction tests. This is the most common method of quality control at construction sites.

If a loose soil is compacted by the application of a fixed amount of energy, then the dry density achieved is related to the moisture content. The moisture-density relationship of soils was first studied by Procter, and the test is sometimes known as Procter test. The dry density that can be obtained by compaction varies with the moisture content, type of soil being compacted, and the compaction effort. The relationship between moisture content and dry density for practically all soils takes the form shown in Error! Reference source not found..

It can be seen from this relationship that for a given compactive effort, the dry density of a soil will vary with its water content. At low moisture content, the soil is dry and stiff and friction between adjacent particles prevents/limits relative movement between particles to assume denser configuration. As water is added, larger films of water form around the particles, causing lubrication effects and facilitating relative movements between particles to assume denser configuration (high density of soil mass). Thus, the density increase and the air content decreases as the moisture increases. At some moisture content, the soil attains the maximum practical degree of saturation ($S<100\%$). The degree of saturation, $S$, cannot be increase further due to entrapped air in the void spaces and around the particles. Hence any further addition of water will result in the voids being overfilled with water causing separation of particles and reduction of density (the additional water taking the space of the solid particles). The moisture content at which maximum dry density is obtained is known as optimum moisture content (OMC). At moisture content higher than the OMC, the air and water in the soil mass
tend to keep particles apart and prevent compaction. The dry density at higher moisture contents than OMC, thus, decreases and the total voids increase.

\[ \gamma_d = \frac{\gamma_a G_s}{1 + w G_s} \]  

(4.2)

where, \( \gamma_d \) = Dry density of soil,  
\( \gamma_a \) = unit weight (density) of water,  
\( G_s \) = specific gravity of soil particles, and  
\( w \) = moisture content of the soil.

The distance between the zero-air void (ZAV) curve and the test moisture-density curve is an indication of the amount of air voids remaining in the soil at different moisture contents. The farther away a point on the moisture-density curve is from the ZAV curve, the more air voids remaining in the soil and the higher is the likelihood of expansion or swelling if the soil is exposed to water. Swelling of such soil can be reduced by compacting at higher moisture content.
Soil type and gradation heavily affect the density that can be achieved by compaction. Granular, well-graded soils generally have fairly high maximum densities at lower optimum moisture contents, while clayey soils have lower densities and higher OMC. The edge-to-side bonds between clay particles resist compactive efforts preventing attainment of denser structure. With granular soils, the more well-graded soils have spaces between large particles that are filled with smaller particles when compacted, leading to a higher density than with uniform or poorly graded soils. Typical moisture-density curves for different types of soils are shown in Figure Error! No text of specified style in document.-2. Note that a line joining the peak points of the density curves would be approximately parallel to the ZAV curve. This is due to the fact that most soils at their maximum density still contain about 2-3% air.

![Figure Error! No text of specified style in document.-2: Compaction curves for different types soils](image)

The OMC and the maximum dry density that can be attained on a given soil also depend on the compactive effort used as shown in Figure Error! No text of specified style in document.-3. Compactive effort is a measure of the mechanical energy imposed on the soil mass during compaction (energy per unit volume). For a given soil, increase in compactive effort generally results in an increase in dry density and a decrease in optimum moisture content.
**Laboratory compaction test**: is a standard method of compaction using a standard amount of compactive effort to produce a soil density against which site density values can be compared. The original test involved compacting the soil in three approximately equal layers in a standard mould, using a 2.5kg hammer falling through a height of 305mm (standard compaction test). However, with the advent of heavier compaction equipment, greater densities were now achievable in the field. A modified version of the test was developed to allow the application of greater compactive effort (and achieve greater density) – i.e. compacting the soil of the same height in five approximately equal layers using a 4.5kg hammer falling through 457mm height (modified or heavy compaction test).

The soil sample is first air dried and sieved (usually through the 4.75-mm (No.4) sieve or 19mm sieve), mixed thoroughly with water and then compacted in layers. The mass of the compacted sample is measured (W), and a small sample taken to measure the corresponding moisture content (w). More water is then added to the soil, and the procedure repeated until the dry density obtained decreases. Comparison of standard and modified compaction tests is given in the following table;

**Table 4.4**: Standard and Modified compaction tests

<table>
<thead>
<tr>
<th>Items</th>
<th>Standard Compaction Test (AASHTO T99)</th>
<th>Modified (Heavy) Compaction Test (AASHTO T180)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of mould (mm)</td>
<td>101.6/152.4</td>
<td>101.6/152.4</td>
</tr>
<tr>
<td>Height of sample (mm)</td>
<td>117</td>
<td>117</td>
</tr>
<tr>
<td>Number of lifts (layers)</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>Number of blows per lift</td>
<td>25/56</td>
<td>25/56</td>
</tr>
<tr>
<td>-------------------------</td>
<td>-----------</td>
<td>-----------</td>
</tr>
<tr>
<td>Weight of Hammer</td>
<td>2.5kg</td>
<td>4.5kg</td>
</tr>
<tr>
<td>Diameter of end face of hammer (mm)</td>
<td>51</td>
<td>51</td>
</tr>
<tr>
<td>Free fall height (mm)</td>
<td>305</td>
<td>457</td>
</tr>
<tr>
<td>Net volume of mould (cm³)</td>
<td>944/2124</td>
<td>944/2124</td>
</tr>
</tbody>
</table>

Note: larger diameter mould (152.5mm) is used for gravelly soils (soils with a significant amount of gravel).

The bulk density of the soil for each trial is obtained by dividing the weight of the soil by the total volume ($\gamma_b=W/V$). The dry density of the soil is determined by:

$$\gamma_d = \frac{\gamma_m}{1 + w}$$  \hspace{1cm} (4.3)

Where $\gamma_b = \text{bulk unit weight}$, $w = \text{moisture content}$

**Field density test**: Since the compatibility of soils varies considerably, the construction requirements for roads are usually specified as a percentage of the maximum dry density found in a laboratory compaction test for each soil type encountered on the project. For example, a project specification might require that the soil be compacted to 95% of the maximum dry density found by the standard compaction test. Quality control of compaction on a construction project involves conducting standard field compaction tests on each soil type and constructed layer after compaction, and comparing the result with the laboratory maximum dry density value for the soil, to ascertain if the specifications have been met. If the maximum dry density from the test was 2000 kg/m³ at an optimum water content of 11%, the required field density would be 95% of 2000, or 1900 kg/m³. The moisture content of the soil should be as close as possible to 11%, which reduces the required compactive effort (for example, number of passes of the roller).

Field density tests are made using either destructive or nondestructive methods.

Destructive methods: the simplest is the core-cutter method. This method can be used only on cohesive soils free from coarse-grained material. It involves driving a hollow metal cylinder, which has a cutting edge, into the soil to remove an undistributed sample on which dry density and moisture content
determinations can be made. The other commonly used methods are the Sand Replacement method and Rubber Balloon method. In these methods, a sample of compacted material is dug out of a test hole in the soil layer whose density is being checked. The bulk mass of the soil removed is immediately weighed (making sure that it does not lose any moisture) and the sample transported to the laboratory for measuring the moisture content or the oven dried mass. The volume originally occupied by the sample (the test hole) is then measured. The two methods differ in the method used to measure the volume of the test hole. In the rubber balloon method, the volume is determined by forcing a liquid-filled balloon into the test hole. The rubber membrane allows the fluid to fill all the cavities in the test hole. The volume of fluid required to do this is read on a scale on the apparatus. In the sand replacement method (using a sand cone apparatus), the weight of a standard dry sand \( W_s \), of known unit weight, \( \gamma_s \), required to fill the test hole is measured. The volume of the test hole is then determined from the known unit weight of the sand as follows:

\[
V = \frac{W_s}{\gamma_s} \quad (4.4)
\]

The quick and nondestructive method of measuring the in situ density and moisture content of the compacted soil is the nuclear method. Using the nuclear equipment, the density is obtained by measuring the scatter of gamma radiation by the soil particles since the amount of rays is proportional to the bulk density of the soil. The moisture content is also obtained by measuring the scatter of neutrons emitted in the soil due to the presence of hydrogen atoms. The detector in the nuclear equipment measures the amount of rays and the neutrons that passes through the soil, and thus the density and the moisture content can be calculated.

### 4.3.4 California Bearing Ratio (CBR) Test

The CBR test was originally developed by the California Division of Highways in the 1930s, as part of a study of pavement failures. Its purpose was to provide an assessment of the relative stability of fine crushed rock base materials. The test has been modified since then and extended to subgrades. It is now widely used for evaluating the stability or strength of subgrade soil and other flexible pavement materials for pavement design throughout the world. The CBR values obtained from either laboratory tests or in-situ (field tests) have been correlated with flexible pavement thickness requirements for highways and airfields.

In this test, a plunger is made to penetrate the soil, which is compacted to the prevalent dry density and moisture content anticipated in the field (or to MDD and OMC as specified) in a standard mould (CBR
mould) at a specified rate of penetration. The resulting load-penetration curve is compared with that obtained for a standard crushed rock material, which is considered an excellent base course material. Depending upon the prevailing climatic conditions of the site, the compacted specimens are immersed in water for four days before the penetration test. The soaking process is to simulate the worst moisture condition of the soil that may occur in the field. During this period, the sample is loaded with a surcharge load that simulates the estimated weight of pavement layers over the material tested. Any swell due to soaking is also measured.

The load is applied by cylindrical metal plunger of 50 mm diameter, the standard penetration rate used is 1.27mm/minute and readings of the applied load are taken at appropriate intervals of penetration (0.5mm, 1.27mm(0.5")) up to a total penetration of usually not more than 7.5 mm-12.7mm.

Typical test results are illustrated in Error! Reference source not found.. It sometimes happens that the plunger is still not perfectly bedded in the specimen and, as a result of this and other factors, a load-penetration curve with a shape similar to that of curve for Test 2 in Error! Reference source not found. may be obtained instead of the more normal shaped curve illustrated by the curve for Test 1. When this happens the curve must be corrected by drawing a tangent at the point of greatest slope and then transposing the axis of load so that zero penetration is taken as the point where the tangent cuts the axis of penetration (as illustrated in Error! Reference source not found.).

The CBR is then determined by reading off from the curve the load that causes a penetration of 2.54 mm and dividing this value by the standard load (13.34kN) required to produce the same penetration in the standard crushed stone as

$$CBR = \frac{\text{Unit load for 2.54 mm penetration in test specimen}}{\text{Unit load for 2.54 mm penetration in standard crushed rock}} \times 100$$ (4.5)

Similarly, the CBR at 5.08 mm penetration is obtained by dividing the load causing a penetration of 5.08 mm with the standard load of 20kN required to produce the same penetration in standard crushed stone. The CBR corresponding to 2.54mm penetration is normally greater than that at 5.08mm pen., and is accepted as the CBR of the soil (provided that it is greater than that obtained at 5.08mm penetration). AASHTO T193 test procedure stipulates that, if the CBR at 5.08 mm pen. is greater than that at 2.54mm pen., the entire test should be repeated on a fresh sample. If the 5.08 mm pen. CBR in the repeat test is still greater, then it is accepted as the CBR of the soil.
Apparatus is also available to carry out in situ CBR tests in the field on exposed subgrades, subbases, and bases. Such tests can be useful in investigating pavement failures and also in examining existing roads in good condition. Accompanied by measurements of field densities and moisture conditions, such testing provides a useful means of building up knowledge of appropriate pavement design criteria for local soils under the locally prevailing climatic conditions.

**Design Subgrade CBR:** The strength of subgrade soils is dependent on the type of soil, density, and moisture content. Hence to determine the subgrade strength, which would be used for design of the road pavement structure, it is apparent to ascertain the density-moisture content-strength relationship specific to the subgrade soils encountered along the project road. The design CBR of the subgrade soil, therefore, should be evaluated at the moisture content and density representative to the subgrade condition during the service time of the pavement structure. For wet or moderate climatic zones and

---

**Figure 4.5: Typical Load – Penetration Curve**

[Diagram showing typical load-penetration curve with annotations on tests and penetration correction.]
where the ground water influences the subgrade moisture content, the CBR test is carried out after 4 days of soaking.

A road section for which a pavement design is undertaken should be subdivided into subgrade areas where the subgrade CBR can be reasonably expected to be uniform, i.e. without significant variations. Identification of sections deemed to have homogenous subgrade conditions is carried out by desk studies on the basis of geology, pedology, drainage conditions and topography, and considering soil categories which have fairly consistent geotechnical characteristics (e.g. grading, plasticity, CBR). Usually, the number of soil categories and the number of uniform subgrade areas will not exceed 4 or 5 for a given road project. The design subgrade CBR for homogenous section is usually taken as the 90 %-ile value of the CBR test results as shown in Figure Error! No text of specified style in document.-4.

![CBR values plotted in ascending order](Image)

**Figure** Error! No text of specified style in document.-4: Design CBR as the 90 %-ile value

### 4.3.5 Resilient Modulus Test

The resilient modulus has recently been accepted as the most representative test for soils and aggregates under highway loading conditions. Stress in pavements is due to repeated moving wheel loads, and hence this test simulates the soil under a series of load applications. The resilient modulus, $M_r$, is the elastic modulus obtained from repetitive load test that simulates the actual pavement loading. It is calculated as the ratio of the imposed repeated deviator stress ($\sigma_d$) to the recoverable axial strain $\varepsilon_r$.  

75
The above figure shows the straining of a specimen under a repeated load test. At the initial stage of load applications, there is considerable permanent deformation, as indicated by the plastic strain. As the number of repetitions increases, the plastic strain due to each load repetitions decreases. After 100 to 200 repetitions, the strain is practically all recoverable, as indicated by $\varepsilon_r$ in the figure. A triaxial device equipped for repetitive load condition is used to carry out resilient modulus test. The test may be conducted on all types of unbounded pavement materials ranging from cohesive to stabilized materials. However this test is not available in most cases and hence some recommendations are made to correlate the CBR values with the resilient modulus. The asphalt Institute recommends the following approximate relationships in their design method:

$$M_R \text{ (MPa)} = 10.35 \times \text{CBR value} \quad (4.6)$$

### 4.3.6 Other soil Tests

There are other tests likely to be used for soil surveys, design and control of construction depending on the site conditions encountered and structures to be constructed. These include field and laboratory tests such as those conducted to determine the shear strength, settlement, and permeability of soils. These include tests required for special investigations relating to deep cuts, embankments over soft and compressible soils, expansive soils and natural slopes (Refer to ERA, Site Investigation Manual, 2001). Procedures for these and those described above are defined in BS, ASTM, AASHTO and other equivalent standards.
4.4 Soil Classification for Highway Use

The purpose of soil classification system is to group soils with similar properties or attributes. As a means of obtaining general behavior, soils are systematically categorized on the basis of some common characteristics obtained from visual inspection:description and laboratory tests. Various soil classification systems are in use throughout the world in different areas of study. In highway engineering, soils are classified by conducting relatively simple tests on disturbed samples to serve as a means of identifying suitable materials and predicting the probable behavior when used as subgrade or subbase material. The two most important soil characteristics used in classifying soils are their grain size distribution and plasticity.

The most commonly used classification systems for highway purposes are the American Association of State Highway and Transportation Officials (AASHTO) Classification System and the Unified Soil Classification System (USCS). These classification systems only help engineers to predict how the soil will behave if used as a subgrade or subbase material, however, the information obtained should not be regarded as a substitute for the detailed investigation of the soil properties.

4.4.1 AASHTO Classification System

The AASHTO Classification System is based on the Public Roads Classification System that was developed from the results of extensive research conducted by the Bureau of Public Roads, now known as the Federal Highway Administration of the United States. Several revisions have been made to the system since it was first published. The system has been described by AASHTO as a means for determining the relative quality of soils for use in embankments, subgrades, subbases, and bases.

In this system of classification, soils are categorized into seven groups, A-1 through A-7, with several subgroups, as shown in Error! Not a valid bookmark self-reference. The classification of a given soil is based on its particle size distribution, LL, and PI. Soils are evaluated within each group by using an empirical formula to determine the group index (GI) of the soils, given as

\[
GI = (F - 35)[0.2 + 0.005(\text{LL} - 40)] + 0.01(F - 15)(\text{PI} - 10)
\]  

(4.7)

where, GI = group index

\[
F = \% \text{ of soil particles passing 0.075 mm (No. 200) sieve in whole number based on material passing 75 mm (3 in.) sieve,}
\]

\[
\text{LL} = \text{liquid limit expressed in whole number, and}
\]

\[
\text{PI} = \text{plasticity index expressed in whole number.}
\]
The GI is determined to the nearest whole number. A value of zero should be recorded when a negative value is obtained for the GI. Also, in determining the GI for A-2-6 and A-2-7 subgroups, the LL part is not used, that is, only the second term of the equation is used. Classifying soils under the AASHTO system is finding the correct group for the particle size distribution and atterberg limits of the soil from the classification. In this system of classification, soils are categorized into seven groups, A-1 through A-7, with several subgroups, as shown in Table 5. The classification of a given soil is based on its particle size distribution, LL, and PI. Soils are evaluated within each group by using an empirical formula to determine the group index (GI) of the soils, given as

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10)$$

where, GI = group index

- $F$ = % of soil particles passing 0.075 mm (No. 200) sieve in whole number based on material passing 75 mm (3 in.) sieve,
- $LL$ = liquid limit expressed in whole number, and
- $PI$ = plasticity index expressed in whole number.

Table 5: AASHTO soil classification system

<table>
<thead>
<tr>
<th>General Classification</th>
<th>Granular Materials (35 Percent or Less Passing 75 μm)</th>
<th>Silt-Clay Materials (More Than 35 Percent Passing 75 μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve analysis, percent passing:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00 mm (No. 30)</td>
<td>50 max</td>
<td>50 max</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>50 max</td>
<td>50 max</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>50 max</td>
<td>50 max</td>
</tr>
</tbody>
</table>

Characteristics of fraction passing 0.425 mm (No. 40)

- Liquid limit: 10 max 40 max 40 max 40 max 40 max 40 max 40 max 40 max 40 max 40 max 40 max 40 max 40 max
- Plasticity index: 6 max 10 max 10 max 10 max 10 max 10 max 10 max 10 max 10 max 10 max 10 max 10 max 10 max

Usual types of significant constituent materials

- Stone fragments, gravel, and sand, Silty or clayey gravel and sand, Silty soils, Clayey soils

General rating as subgrade: Excellent to Good, Fair to Poor

* Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30. (See Figure 1.)

The GI is determined to the nearest whole number. A value of zero should be recorded when a negative value is obtained for the GI. Also, in determining the GI for A-2-6 and A-2-7 subgroups, the LL part is not used, that is, only the second term of the equation is used. Classifying soils under the AASHTO system is finding the correct group for the particle size distribution and atterberg limits of the soil from the classification. In this system of classification, soils are categorized into seven groups, A-1 through A-7, with several subgroups, as shown in Table 5. The classification of a given soil is based on its particle size distribution, LL, and PI. Soils are evaluated within each group by using an empirical formula to determine the group index (GI) of the soils, given as

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10)$$

where, GI = group index

- $F$ = % of soil particles passing 0.075 mm (No. 200) sieve in whole number based on material passing 75 mm (3 in.) sieve,
- $LL$ = liquid limit expressed in whole number, and
- $PI$ = plasticity index expressed in whole number.

The group is then designated using the GI value. Granular soils fall into classes A-1 to A-3.

A-1 soils consist of well-graded granular materials, A-2 soils contain significant amounts of silts and clays, and A-3 soils are clean but poorly graded sands. A-4 soils cover non-plastic or moderately plastic soils, and A-5 contains similar material to Group A-4 but exhibits high LL. A-6 soils are typical plastic clays exhibiting high volume change between wet and dry states. Group A-7 covers plastic clays, having high values of LL and PI and show high volume change.
In general, according to the AASHTO system of classification, the suitability of a soil deposit for use in highway construction can be summarized as follows.

1. Soils classified as A-1-a, A-1-b, A-2-4, A-2-5, and A-3 can be used satisfactorily as subgrade or subbase material if properly drained. In addition, such soils must be properly compacted and covered with an adequate thickness of pavement for the surface load to be carried.

2. Materials classified as A-2-6, A-2-7, A-4, A-5, A-6, A-7-5, and A-7-6 will require a layer of subbase material if used as subgrade. If these are to be used as embankment materials, special attention must be given to the design of the embankment.

3. Generally, as the GI of a soil increases its value as subgrade material decreases. For example, a soil with a GI of 0 (an indication of a good subgrade material) will be better as a subgrade material than one with GI of 20 (an indication of a poor subgrade material).

4.4.2 Unified Soil Classification System (USCS)

Originally developed by Casagrande during World War II for use in airfield construction, USCS has been modified several times to obtain the current version. The fundamental premise used in the USCS system is that, the engineering properties of any coarse-grained soil depend on its particle size distribution, whereas those for a fine-grained soil depend on its plasticity. Thus, the system classifies coarse-grained soils on the basis of grain size characteristics and fine-grained soils according to plasticity characteristics.

In this system of classification, material that is retained in the 75 mm (3 in.) sieve is recorded, but only that which passes is used for the classification of the sample. Soils are designated by letter symbols with each letter having a particular meaning as defined as follows:

**Coarse-grained soils.** Soils with more than 50 percent of their particles being retained on the No. 200 sieve are classified as coarse-grained soils. The coarse-grained soils are subdivided into gravels (G) — soils having more than 50 percent of their particles larger than 4.75 mm (i.e., retained on No. 4 sieve) and sands (S) — those with more than 50 percent of their particles smaller than 4.75 mm (i.e., passed through No. 4 sieve). The gravels and sands are further divided into four subgroups, each based on grain size distribution and the nature of the fine particles in them as well graded (W), poorly graded (P), silty (M), or clayey (C). Gravels can be described as either well-graded gravel (GW), poorly graded...
gravel (GP), silty gravel (GM), or clayey gravels (GC), and sands can be described as well-graded sand (SW), poorly graded sand (SP), silty sand (SM), or clayey sand (SC).

A gravel or sandy soil is described as well graded or poorly graded, depending on the values of two shape parameters known as the coefficient of uniformity, $C_u$, and the coefficient of curvature, $C_c$ given as

\[
C_u = \frac{D_{60}}{D_{10}}
\]

and

\[
C_c = \frac{(D_{50})^2}{D_{10} \times D_{60}}
\]

where, $D_{60} =$ grain diameter at 60% passing

$D_{30} =$ grain diameter at 30% passing

$D_{10} =$ grain diameter at 10% passing

Accordingly, gravels are described as well graded if $C_u$ is above 4, and $C_c$ is between 1 and 3. Sands are also described as well graded if $C_u$ is above 6, and $C_c$ is between 1 and 3.

Moreover, coarse-grained soils with more than 12 percent fines (i.e., passes No. 200 sieve) are classified as silty or clayey depending on their LL plots. Those soils with plots below the "A" line (defined as below) or with a PI less than 4 are silty gravel (GM) or silty sand (SM), and those with plots above the "A" line with a PI greater than 7 are classified as clayey gravels (GC) or clayey sands (SC).

**Fine-grained soils.** Soils with less than 50 percent of their particles retained on the No. 200 sieve are classified as fine-grained soils. The fine-grained soils are subdivided into clays (C) or silt (M) based on a plasticity chart plotted PI versus LL of the soil from which a dividing line known as the "A" line separates the more clayey materials from the silty materials. The equation of the "A" line is

\[
PI = 0.73(LL - 20)
\]

Soils that fall below the "A" line are silty soils, whereas those with plots above the "A" line are clayey soils. Organic clays are an exception to this general rule since they plot below the "A" line. Organic clays, however, generally behave similarly to soils of lower plasticity. The organic, silty, and clayey soils are further divided into two groups, one having a relatively low LL (L) and the other having a relatively high LL (H). The dividing line between high LL soils and low LL soils is arbitrarily set at 50 percent. Fine-grained soils are, thus, further classified as either silt with low plasticity (ML), silt with high
plasticity (MH), clays with high plasticity (CH), clays with low plasticity (CL), or organic with high plasticity (OH).

Table 4-6: Unified Soil Classification System
<table>
<thead>
<tr>
<th>Major Divisions</th>
<th>Group Symbols</th>
<th>Typical Names</th>
<th>Laboratory Classification Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse-grained soils (more than half of material is coarser than No. 200 sieve size)</td>
<td>GW</td>
<td>Well-graded gravels, gravel-sand mixtures, little or no fines</td>
<td>$C_u = \frac{D_{50}}{D_{10}} &gt; 4$, (C_c = \frac{(D_{50})^2}{D_{10} \times D_{30}}) between 1 and 3, Not meeting all gradation requirements for GW.</td>
</tr>
<tr>
<td></td>
<td>GP</td>
<td>Poorly graded gravels, gravel-sand mixtures, little or no fines</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td>Silty gravels, gravel-sand-silt mixtures</td>
<td>Atterberg limits below “A” line with P.I. less than 4, Above “A” line with P.I. greater than 7.</td>
</tr>
<tr>
<td></td>
<td>GC</td>
<td>Clayey gravels, gravel-sand-clay mixtures</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SW</td>
<td>Well-graded sands, gravelly sands, little or no fines</td>
<td>Atterberg limits above “A” line with P.I. less than 4, Atterberg limits above “A” line with P.I. greater than 7.</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>Poorly graded sands, gravelly sands, little or no fines</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Silty sands, sand-silt mixtures</td>
<td>Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.</td>
</tr>
<tr>
<td></td>
<td>SC</td>
<td>Clayey sands, sand-clay mixtures</td>
<td></td>
</tr>
<tr>
<td>Fine-grained soils (less than half of material is finer than No. 200 sieve size)</td>
<td>ML</td>
<td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity</td>
<td>Plasticity Chart</td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
<td></td>
</tr>
<tr>
<td></td>
<td>OL</td>
<td>Organic silts and organic silty clays of low plasticity</td>
<td>CH</td>
</tr>
<tr>
<td>Silt and clay soils (liquid limit less than 50)</td>
<td>MH</td>
<td>Inorganic silts, micaeous or diatomaceous fine sandy or silty soils, plastic silts</td>
<td>OH and MH</td>
</tr>
<tr>
<td></td>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>Organic clays of medium to high plasticity, organic silts</td>
<td>CL-ML</td>
</tr>
<tr>
<td>Highly organic soils</td>
<td>Pr</td>
<td>Peat and other highly organic soils</td>
<td>ML and OL</td>
</tr>
</tbody>
</table>

*Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less, the suffix u used when L.L. is greater than 28.

Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-SC, well-graded gravel-sand mixture with clay binder.